TOWARDS SUSTAINABLE AND ROBUST ON-SITE DOMESTIC WASTEWATER TREATMENT FOR ALL CITIZENS

Shaaban Mrisho Mgana

Towards sustainable and robust on-site domestic wastewater treatment for all citizens

Shaaban Mrisho Mgana

Promotoren:

Prof. Dr. ir. G. Lettinga
hoogleraar in de anaërobe zuiveringstechnologie en hergebruik van afvalstoffen

Co-promotor:

Dr. W.T.M. Sanders
Post –doc onderzoeker bij de sectie Milieutechnologie van het departement agrotechnologie en voedingswetenschappen

Prof. Tolly S.A. Mbwette
Dar es salaam Universiteit, Tanzania

Samenstelling promotiecommissie:

Dr. ir. G. Zeeman	Wageningen Universiteit
Prof. dr. H.J.Gijzen	UNESCO-IHE, Delft/Wageningen Universiteit
Prof. Dr. J. Iza	University of the Basque, Bilbao, Spain
Prof. ir. J.H.J.M. Van der Graaf	Technische Universiteit Delft

Towards sustainable and robust on-site domestic wastewater treatment for all citizens

Shaaban Mrisho Mgana

Proefschrift

ter verkrijging van de graad van doctor
op gezag van de rector magnificus
van Wageningen Universiteit,
prof. dr. ir. L. Speelman,
in het openbaar te verdedigen
op Woensdag 8 Januari 2003
des namiddags te vier uur in de aula

CIP-DATA KONINKLIJKE BIBLIOTHEEK, DEN HAAG

Author: Shaaban Mrisho Mgana
Title: Towards sustainable and robust community on-site domestic wastewater treatment
ISBN:90-5808-764-6
Publication year: 2003
Subject headings: domestic wastewater/anaerobic sludge stabilisation /septic tank/single-step UASB reactor/ two-step UASB reactor
Thesis Wageningen University, Wageningen, The Netherlands- with references – with summary, discussion, conclusion and recommendations in English and Dutch

In memory of my parents,
mzee Mrisho Mgana
and
Mwamvua binti Msafiri

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

In most developing countries commonly practiced domestic wastewater treatment systems predominantly constitute anaerobic treatment process. The anaerobic treatment units mostly installed are on-site at residential dwellings.

However the commonly installed units, viz., traditional pit latrines and septic tanks are in fact 'low-rate' anaerobic pre-treatment units and are associated with inefficiency, poor maintenance and groundwater pollution. Moreover since most poor communities, who constitute the majority in the developing countries' populations could afford these types of anaerobic pre-treatment units, their numbers have also grown to unmanageable proportions. Consequently the demand for effective but low cost wastewater treatment facilities for developing countries is indisputably great.

On the basis of already available technical information concerning the Upflow Anaerobic Sludge Bed (UASB) reactor performance a wastewater treatment system based on the UASB reactor can lead to a compact, effective and low cost community on-site pre-treatment unit for tropical wastewaters. However the performance of these systems in an actual community on-site situation has so far not been investigated. This thesis therefore investigates the performance and feasibility of using the UASB reactor for the pre-treatment of wastewater under the conditions that arise at community level in tropical regions, viz. highly varying organic and hydraulic loads, but low variation in temperature.

On-site pilot scale UASB reactors were configured and operated in parallel at community level for the purpose of acquiring performance data. All the reactors were operated in gravity flow mode at ambient tropical temperature of 25 - 34 ºC. The wastewater in the study area – community level – was highly biodegradable with an average ratio COD:BOD_5 of 1.52 at a standard deviation of 0.13. The wastewater characteristic was highly variable. A 1277-day monitoring duration of the wastewater grab samples has shown that the values of organic loads (with standard deviation in brackets), in terms of COD_{tot}, and COD_{ss}, were 529.4 (544.6) and 264.4 (448.4) mg/L respectively.

A conventional pilot single-step community on-site UASB reactor (volume: 1.5 m^3, height: 1.7 m) was operated over three and a half years at an average hydraulic retention time of 6.2 (4.92). The performance data obtained via regular monitoring of the treatment unit showed a declining removal efficiency over time with respect to COD_{total}, which likely can be attributed to the increasing rate at which biogas was produced along with the growth of sludge bed and the presence of floating sludge. As a result the removal of dispersed sludge particles becomes poorer, which likely is reinforced by the possible 'less' optimal dimensions and design of the Gas-Solids-Separator (GSS) device. The average removal efficiency on COD_{tot} basis was 64 percent. However a study of a parallel pilot two-step community on-site UASB reactor configuration gave more promising results. The two-step UASB pre-treatment

unit in this research refers to two UASB reactors connected in series, viz. a first 2m high 1.8m^3 UASB reactor put in front of a second 1m high 0.852 m^3 UASB reactor. The second-step UASB reactor is a recipient of effluent including washouts from the first-step UASB reactor The average organic loads of the wastewater imposed to the system with respect to COD fractions COD_{tot}, COD_{ss}, COD_{col} and COD_{sol} were 537.2 (165.3), 189.9 (109.5), 127.4 (75) and 223.4 (108.8) respectively. The investigations were conducted over a period of 630 days. The overall removal efficiency obtained on the basis of the distinguished COD fractions was far better than for the individual reactors, i.e efficiencies for COD_{tot}, COD_{ss}, COD_{col} and COD_{sol}, were respectively 68.7 (16.7), 51.2 (41.8), 62.1(38.2), and 71.8 (30.5) %. The imposed overall HRT was 7.4 (1.6) (i.e. 5 + 2.4) hours. The advantages of the two-step UASB reactor configuration include 1) the distinct higher overall removal efficiency of the anaerobic pre-treatment system 2) the higher sludge age 3) the higher reliability of the anaerobic pre-treatment process 4) the two reactors can separately be operated in case of technical problems.

ACKNOWLEDGEMENT

I am grateful and appreciate Prof. Gatze Lettinga's identification of my participation in the "Sandwich Construction" of the University of Wageningen. I have been fortunate to be a student of Prof. Lettinga, my 'promotor' and on this account I would like to express my appreciation of his integrity, vision and vivid imagination that I experienced in various facets of this research throughout the research period. For example, I was always amazed by his deep comprehension of the very difficult research conditions I was persevering in Tanzania and always he acted very judiciously. His support was beyond the call of duty of a "promotor". That was very encouraging and enabled me to continue with the research. While in The Netherlands, the few occasions that I met Dora Lettinga left me with the impression of a responsible hospitable family.

I am also very grateful to Wageningen University for the support extended to me within the framework of the "Sandwich Construction". The former Ardhi Institute (currently, University College of Lands and Architectural Studies – 'UCLAS') being the home institute in Tanzania was instrumental in supporting the research undertaking locally. On the later, I would like to sincerely thank Prof. Saburi John and Haruna Masebu, who were then the Principal and Director of Studies of Ardhi Institute respectively, for the interest and commitment they have shown at the initiation of the research. At this juncture I am expressing my admiration to the current Principal of UCLAS Prof. Idris Kikula that within a short stay at UCLAS after his appointment as Principal, he was able to immediately comprehend the potential this research has in alleviating the insanitary conditions prevailing in community settlements in Tanzania and the encouragement extended. May I also take this opportunity to thank my local promotor, Prof. Tolly Mbwette for the enthusiasm he had for this research as well as my appreciation of his responsible administrative guidance and the comments he made for the thesis.

My very sincere gratitude goes to Wendy Sanders from whom I received the most important support during the writing of this thesis that experienced serious time constrains. I thank her for the patience, critic, very constructive demands and a demonstration of a high degree of responsibility as a promotor. Above all, incredibly, she managed to do all this in the midst of taking care as a mother of her two very young children. My appreciation of the encouragement I received and the comments made thereafter on this research goes to Dr. G. Zeeman. Grateful to Joost van Buuren for the regular exchange of research ideas and the efforts demonstrated in looking for supportive research funds.

During my short stay in The Netherlands, I received lots of helping hands from within and without the Wageningen University community that were responsible for an effective and conducive working environment. Some of those whom I could recall their names are: Bram Klapwijk, Look Hulshoff Pol, Jules van Lier, Wim Rulkens, Adriaan Mels, Katarzyna Kujawa-Roeleved, Hardy Timmink, Heleen Vos and Liesbeth Kesaulya-Monster, Rob Roersma, Bert Willemsen, Sjoerd Hobman, Bas Buys, Hellen Elissen, Vinnie de Wilde, Katje Grolle, Peter van der Steen, Tarek Ellmitwalli, Maha Halalsheh, Hamed Elmashad, Ahmed Tawfik, Nedal Mahmoud, Titia de Mes. Thanks to Gerry and Joop Rikken for their hospitality while residing in their house during the start-up of the programme.

Many thanks are due to Addo Ndimbo for the effective participation and unfaltering maintenance of the units during the research period. Best regards to Richard Faber for the commitment, enthusiasm and effective participation in this research while in Tanzania as part of his MSc programme. I wish to express my admiration to my colleague Stephen Mbuligwe for the resolute enthusiasm to this research and dynamic encouragement that I received throughout the conduct of this research. In parallel I wish to extend my sincere thanks to all staff and students of environmental engineering department of UCLAS who contributed in one way or another to the realisation of this research.

Thanks to my elder brother, Salum Mrisho Mgana to whom much of what my life realises in adulthood is based on a childhood discipline that he was responsible for. Finally my very special thanks are meant to my wife Donatha Balthalzar Mgana for the excellent responsibility and affectionate to our family while I was busy with the research either in the country or abroad. To my children Anza, Adam and Azima I wish I spent more time with you, I hope I will in future!

CONTENTS

CONTENTS

1 GENERAL INTRODUCTION

Scope of this thesis – On-site wastewater treatment processes at individual residential dwelling or at community level has often been overlooked in anticipation that one day all domestic wastewater will be drained off through centralised sewerage systems and treated off-site in a central wastewater treatment plant. But the reality is, that is not always the case particularly in the developing world. Meanwhile 'low rate' anaerobic treatment units have become wide spread, normally with little attention paid to their performance after construction and as a result public health and the environment have become victims of false hope. On the basis of already available technical information concerning the Upflow Anaerobic Sludge Bed (UASB) reactor performance it was decided that a wastewater treatment system based on the UASB reactor can lead to a compact, effective and low cost community on-site pre-treatment unit for tropical wastewaters. Although there is already a lot of knowledge available on UASB reactors the performance of these systems in an actual community on-site situation has so far not been investigated. Therefore the scope of this thesis is to investigate the performance and feasibility of using the UASB reactor for the pre-treatment of wastewater under the conditions that arise at community level in tropical regions, viz. highly varying organic and hydraulic loads, but low variation in temperature.

1.0 Background

The desire of providing sanitation to majority of the peoples of developing world through conventional approach that involve central sewerage system has practically failed. The failure is witnessed in all human settlement areas: urban, peri-urban and rural. The major reason for failure is that the conventional sewerage systems that are normally accompanied with centralised wastewater treatment plants are certainly far too expensive and complex for poor countries (Zeeman, *et al*., 2001). Nevertheless western consultants and contractors to implement western expensive concepts of sanitation attempt these systems. This failure has led to wide spread use of onsite sanitation systems viz. pit latrines and septic tanks in the case of Tanzania.

About 90 percent of the population in Tanzania is served by on-site sanitation systems: pit latrines (80%) and septic tanks (10%). The biological treatment process-taking place in these on-site systems is anaerobic digestion. These traditional collection + treatment systems in fact belong to the category "low rate" anaerobic treatment units. They are often associated with inefficiency and groundwater pollution. Performance data of a typical example of these systems in the city of Dar es salaam, the commercial metropolis of Tanzania where there is documentation on possible pollution loads these systems might discharge to groundwater and surface water is shown in Table 1.1.

Table 1.1: Pollution loads from Pit latrines and septic tanks to water resources in Dar es salaam city

Type of Pollution Load	Pollution loads quantities discharged from pit latrines and septic tanks to identified water resources (kg/day)			
	Pit latrines	Septic tanks	Pit latrines	Septic tanks
	Surface water resources		Groundwater resources	
BOD	15,282	3,275	15,282	7,641
COD	16,131	3,457	16,131	8,068
Suspended Solids	25,470	5,458	6,116	3,832
Dissolved Solids	45,280	9,830	97,857	61,128
Total N	2,236	479	4,829	3,018
Total P	425	91	915	572

Source:UNEP, GPA COORDINATION OFFICE, THE HAGUE, Institute of Marine Sciences University of Dar es salaam (2002)

The sanitation situation in all urban areas of Tanzania is comparable to that of Dar es salaam city and is worse when compared to rural areas. Access to safe water is still a problem in Tanzania. The overall population in Tanzania that had access to safe water in the period 1988 – 1993 was about 50 percent. The population in urban and rural areas that had access to safe water in the same period 1988-1993 was 67 and 46 percent respectively (UNEP *et al.*, 2002). Of major concern is the immense urban development that has occurred in urban areas placing heavy demands on social services including waste disposal services. Urbanisation population trends for Tanzania are on the increase. For example the percentage of urban population for the years 1960, 1994 and 2000 were 4, 24, and 28 respectively (UNEP *et al.*, 2002). The outcome of this high population growth in major urban areas is a proliferation of unplanned and un-serviced areas. Under these circumstances the sanitation situation in urban areas get worsened particularly in congested unplanned settlements. The pit latrines and septic tanks become too numerous to manage, apart from the question whether there are available good guidelines for operation and maintenance of these systems and/or these guidelines are followed. Consequently they pose an increasing environmental pollution problem.

Typical features of existing urban sewerage systems in Tanzania -the case of Dar es salaam, the commercial metropolis of Tanzania

The city of Dar es salaam is estimated to have 75 percent of her residents living in squatter areas and 65 percent of new housing is being built in these areas (UNEP *et al.*, 2002). Presently most of the residential areas in Dar es salaam are served by onsite disposal systems in the form of pit latrines or septic tanks with soak pits. About 10 percent of the current population in Dar es salaam is served by sewerage system (Elmcrest group, *et al*., 2000). The sewerage system in Dar es salaam is operated and maintained by a semi-autonomous organisation, the Dar es salaam Water and Sanitation Authority (DAWASA). The system is based on a separate system (i.e. excluding storm water) with a combination of gravity and pumped flows, comprising of 15 pumping stations, a total sewer length of 169 km of 100 –1,000 mm diameter covering a total area of 1,717 hectares, with access for servicing via over 2,967 manholes (Elmcrest group, *et al*., 2000). The Dar es salaam sewerage system is a collection of small independent drainage areas rather than a fully integrated network. Apart from the city centre, other sewered areas are mainly institutional: airport, military barracks and the University of Dar es salaam. A total of 9-sets of waste stabilization ponds are used in treating sewage from these independent drainage area based sewerage systems. The 9-sets of waste stabilization ponds have a total area of 23.2 hectares and overall volume of 304,376 m^3 with a total design hydraulic retention time of 160 days (Elmcrest group, *et al*., 2000). The sewerage system serving the city centre disposes of the sewage untreated via a 1-km long, 1,000 mm diameter asbestos-cement sea outfall. The sea outfall was constructed in 1956 and rehabilitated during the 1986-90 rehabilitation programme (Elmcrest group, *et al*., 2000). Some of the oldest sewers of the sewerage system were laid between 1948 and 1952 and the latest in 1975. Operation maintenance has always been minimal and as a result most of the system has effectively stopped functioning (Elmcrest group, *et al*., 2000). Previous full rehabilitation of the system was undertaken in the period between 1986 and 1990. However, the situation has now generally reverted back to that of 1986 (Elmcrest group, *et al*., 2000) and rehabilitation of the system is again required.

The lesson learned here from this brief presentation of the state of the sewerage system in Dar es salaam metropolis is that regular high rehabilitation investments in terms of huge replacement capital cost for worn out parts/facilities and high operational costs, mainly through the employment of 15 sewage pumping stations is needed in serving only 10 percent of the city residents. Had proper wastewater treatment technology and a more decentralized wastewater management system option been adopted then many smaller communities in the city of Dar es salaam metropolis would have been provided with quality wastewater treatment in a most cost-effective manner.

The UASB reactor

Modern high rate, consequently compact, anaerobic sewage pre-treatment units can offer a feasible solution in situations the like of Dar es salaam metropolis. These systems can be an appropriate option while upgrading the worsening sanitation situation in congested unplanned settlements when installed onsite at community level. What make a high rate compact anaerobic treatment unit an attractive sanitation solution are the typical social-economic characteristics of such areas and what the high rate anaerobic treatment unit can offer to alleviate them. These areas are normally poor, they do not have enough space for installation of conventional – so called low-cost - sanitation technologies that would demand large areas of land (like oxidation ponds), and safe water accessibility by the population is still a problem. These social-economic characteristics qualify for the installation of high rate compact anaerobic wastewater treatment systems.

The Upflow Anaerobic Sludge Bed (UASB) reactor was developed in the 1970s by Lettinga and his group at the University of Wageningen in the Netherlands (Haandel and Lettinga, 1994). The UASB reactor presently is by far the most widely used high rate system for sewage pre-treatment. Several full-scale plants have been put into operation and many more are presently under construction (Haandel and Lettinga, 1994)

Modern high-rate anaerobic treatment systems for a number of reasons represents an attractive option for domestic sewage treatment, because of their low construction costs, small land requirements, low excess sludge production, their plain operation and maintenance, and the fact that they generate energy in form of biogas (Haandel and Lettinga, 1994; Zeeeman e*t al.*, 2001; Elmitwalli e*t al*., 2002,). Installation of the

Table 1.2: Design and operational characteristics of a number of full-scale UASB reactors for domestic wastewater (Wiegant, 2001)

No	T (°C)	Volume (m^3)	Height (m)	HRT (h)	V_{up} (m/h)	FID[1] (m)	E_{COD} (%)	E_{BOD} (%)	E_{TSS} (%)	Reference[2]
1	±25	64	4.0	6	0.67	-	65	79	76	1
2[3]	20-30	1,2000	4.5	6	0.75	1.93	74	75	75	2
3	18-30	120		5-15	0.3-0.9	-	±60	±70	±70	3
4	-	6,600	4.0	5.2	0.77	1.70	-	-	-	4
5	16-23	67.5	-	>100	<0.15	1.41	80	74	87	5
6	22-25	160	4.0	7.2	0.56	2.00	67	84	77	6
7	18-26	3,380	4.5	6	0.77	2.00	63	66	73	7
8	24-30	12,000	4.5	6	0.75	2.00	54	52	65	8
9	26-29	11,2000	4.5	6	0.75	2.00	61	48	51	8

[1]FID: feed inlet distance

[2]References(1):Maaskant *et al.*,1991; (2):Draaijer *et al.*, 1992; (3):Vieira and Garcia 1992; (4):Schellinkhout and Collazos, 1992; (5):Vieira *et al.*, 1994; (6): van Haandel and Lettinga, 1994; (7): van Starkenburg *et al.*, 2001; (8): Wiegant *et al.*, 2001

[3]Wastewater partially contaminated with effluent from tanneries, leading to high influent concentrations

high-rate compact anaerobic treatment unit on-site at community level also leads to huge cost reductions in the construction and maintenance of the sewer network including the saving made on the absence of pump stations (Zeeman e*t al.*, 2001).

Monroy *et al*., (2000) reports that the anaerobic treatment of wastewater in Mexico started in 1987 and within one decade, 31 anaerobic reactors, mainly UASB, treating domestic sewage have been installed (Elmitwalli, 2000). The performance of full-scale UASB reactors treating domestic sewage is shown in Table1.2.

The results presented in Table1.2 show that a maximum COD removal can be expected at very low upflow velocities: the highly under loaded plant no.5 achieves removal efficiencies for COD, BOD and TSS of 80, 74 and 87 percent, respectively, at an upflow velocity below 0.15 m/h. On the other hand, the application of very high upflow velocities, of over 1.0 to 1.5 m/h, seems not to have been tested, although the inclusion of a polishing, or sedimentation step after a UASB reactor may justify this approach (Wiegant, 2001).

1.1 Community onsite UASB reactors for pre-treating domestic wastewater

Following the failure of providing centralised sewerage systems modelled in the western industrialised world approach, the demand for effective but low cost wastewater treatment facilities for developing countries like Tanzania is indisputably great. Based on practical experience in project execution in waste and sludge collection in African cities by 'WASTE Advisers on Urban Environment and Development', the scales of for on-site sanitation were clearly identified Rijnsburger (1996): -The *housing unit,* shared by a household or family (i.e. 10-40 people), sharing a housing or concession. This is the scale for on-site treatment, where you find latrine pits, and sometimes septic tanks. -The *housing block*, where 4-10 households (i.e. 40-200 people) as neighbours share an on-site treatment facility, almost in principle a septic tank

The neighbourhood scale is less easy to define in terms of numbers, but has more to do with the entity of the geographical delimitation (Rijnsburger, 1996). The neighbourhood is defined as where a social issue and joint interests are the basis for a *community* effort to improve conditions, either in the upgrading of the on-site facilities, or in the organisation of wastewater collection and the population involved is in the range of 100-2000 people (Rijnsburger 1996). In this research the community on-site wastewater treatment is therefore considered to serve a population in the range of 40-2000 people, viz., from a population size for housing block to that of a neighbourhood. Considering the inhabitant settlements distribution pattern viz. scattered high population densities with low income levels especially in unplanned urban and peri-urban areas, where the majority of the population live, the community on-site UASB reactors treating domestic wastewater, could easily be effectively scaled so as to prescribe to existing socio-economic levels of the would be beneficiaries in terms of ability and willingness to pay for the service.

A recent survey in Pernambuco State, Brazil, has shown that the use of UASB reactors for small domestic wastewater treatment plants is a very feasible option (Florencio, *et al*. 2001). Lack of financial resources for the sewage collection and construction of centralised wastewater treatment plants, the conventional sewerage system plan for the Recife Metropolitan Region (RMR) in Pernambuco State, Brazil could not be implemented and instead decentralised UASB reactors were opted for implementation (Florencio, *et al*. 2001). By the year 2020, 3.3 million inhabitants representing 91 percent of the total population would be served by 55 decentralised sewerage systems, each having a UASB reactor (Florencio, *et al*. 2001). The performance data of three existing UASB reactors in Brazil, in terms of COD removal efficiency was an average of about 80 percent (Kato, *et al*. 2001). The three monitored UASB reactors had the following details: 25 m^3 reactor situated in a small resort hotel (Gravatá City) serving about 500 people, a 340 m^3 reactor situated in a housing project of 5000 inhabitants (Praia Grande, Jaboatão City); and an 810 m^3 reactor situated in a low-income neighbourhood (Mangueira, Recife City) of 18,000 inhabitants. All the reactors each had a hydraulic retention time between 8-9 hours and the net height of 5 m. The reactors developed active biomass that had a maximum methanogenic activity of up to 0.215 gCOD/gVSS.day. Regular maintenance for the reactors included cleansing and removal of grit and scum layer in the reactors (Kato, *et al*. 2001). The decentralised sewerage system with community onsite treatment of sewage has many advantages over centralised sewerage systems with centralised wastewater treatment plants. The advantages include flexibility and gradual implementation of the sewers and treatment plants as the resources become available in contrast to the centralised system in which normally enormous capital is needed so that the whole system has to be built at once (Florencio, *et al*. 2001).

1.2 Conclusion

From what has been presented in this chapter and on the basis of already available technical information concerning the UASB performance, the treatment process of domestic wastewater using the UASB reactor can possibly be stabilised to a simple operational form and come up with a proper community onsite pre-treatment unit that is compact, effective and low cost for tropical wastewaters, particularly for Tanzania. Therefore pilot scale UASB reactors have been constructed for the purpose of gathering performance data in order to confirm the feasibility at local conditions.

1.3 Outline of this thesis

This thesis presents the results obtained during the monitoring operations of constructed pilot scale, community onsite UASB reactor configurations as well as the results of a monitoring operation of a full-scale onsite community septic tank. All reactors monitored were operated in parallel at ambient temperature ranging 25-34 ºC, though the start up was at different times during the research period. Chapter 2 presents the results of the wastewater characteristics in terms of hydraulic and organic loads variations. Comparison is made between centralised and community level wastewater discharge characteristics. Chapter 3 looks at the concept of sludge stability and its characteristics. Generation of stable sludge is a requirement for a successful wastewater treatment process. Stability analysis has been carried out for sludge samples withdrawn from reactors UASB, septic tank and pit latrine. Chapter 4 follows up the performance of a full-scale three-compartment community onsite septic tank that was constructed in 1982. Septic tanks are the most known and commonly applied method for community onsite anaerobic pre-treatment of domestic wastewater. Understanding the limits of treatment performance of the septic tank provides a measure of comparison with other potential wastewater treatment technologies such as the UASB reactors. Chapter 5 presents the results of investigation made on the performance of a pilot single-step community onsite UASB reactor. Relationships of gas production, sludge bed growth, scum layer formation and deformation, as well as the overall performance of the UASB reactor are all correlated simultaneously. Encountered operational problems are identified and possible explanations are given. Chapter 6 presents results of the pilot two-step UASB reactor configuration. The two-step UASB reactor configuration constitutes two UASB reactors in series viz. the first- and second-step UASB reactor. The second-step UASB reactor is fed with the effluent from the first-step UASB reactor. This reactor configuration tries to provide a solution to some of the problems encountered while operating the single-step UASB reactor in Chapter 5. The summaries of the results of the investigations in this thesis and the implications of the need for the effective community onsite UASB reactors treating domestic wastewaters in tropical regions is discussed in Chapter 7.

1.4 References:

Elmcrest group and MMK *Project Services LTD (2000).Proposed Dar es salaam water supplu and sanitation project*. Draft design report.

Elmitwalli, A.T., (2000) Anaerobic treatment of domestic sewage at low temperature. *PhD thesis*, Department of Environmental Technology, Wageningen University, The Netherlands

Elmitwalli Tarik A., Kim L. Oahn, Grietje Zeeman and Gatze Lettinga (2002). Treatment of domestic sewage in a two-step anaerobic filter/anaerobic hybrid system at low temperature.*Water Res* **36**:2225-2232

Florencio, L. and Kato, M. T. (2001).*Poster*: Perspectives of anaerobic treatment for domestic sewage in Recife Metropolitan Region. *Anaerobic digestion for sustainable development: Papers of the farewell seminar of Prof. Dr. ir. Gatze Lettinga, March 29-30,2001, Wageningen, The Netherlands*.

Haandel A. C. van and Lettinga G. (1994). *Anaerobic sewage treatment-A practical guide for regions with a hot climate.* John Wiley & Sons Ltd. England.

IHE, Wageningen Agricultural University (1991). Upflow Anaerobic Sludge Blanket (UASB): Low Cost Sanitation Research Project in Bandung/Indonesia. *Final Report*

Kato, M. T. and Florencio, L. (2001). *Poster*: The use of UASB reactors in small sewage treatment plants. *Anaerobic digestion for sustainable development: Papers of the farewell seminar of Prof. Dr. ir. Gatze Lettinga, March 29-30,2001, Wageningen, The Netherlands*.

Lettinga, G., Man, A.d., Last, A. R. M., v. d., Wiegant, W., Knippenberg, K., Frijns, J. and Buuren, J. C. L. v. (1993) Anaerobic treatment of domestic sewage and wastewater. *Wat. Sci. Tech*., **27**(9), 63-73.

Monroy O., Fatma G., Meraz M., Montoya L., and Macarie H. (2000). Anaerobic digestion for wastewater treatment in Mexico: State of the technology. *Water Res*. **34**(6), 743-750.

Rijnsburger J. (1996). Scale of wastewater. *Workshop Proceedings Sustainable Municipal Wastewater Treatment Systems. Leusden, 12 – 14 November 1996. The Netherlands. Edition January 1998*

UNEP, GPA COORDINATION OFFICE, THE HAGUE, Institute of Marine Sciences University of Dar es salaam (2002). *The regional annex to recommended framework for decision-making*

Wiegant, W.M. (2001).Experiencies and potential of anaerobic wastewater treatment in tropical regions. *Water Sci. Technol*. **44**(8), 107-113

Zeeman, G. and Lettinga, G. (2001). Objectives of DESAR-Decentralise Sanitation and Reuse. *30.Abwassertechnisches Seminar & 13. DECHEMA-Fachgespräch Umweltschutz*

Zeeman, G. and Sanders, W. (2001) Potential of anaerobic digestion of complex waste(water) *Water Sci. Tech.* **40**(8), 115-122

2 WASTEWATER CHARACTERISTICS AND FLOWS

Abstract – The major constituents of the wastewater at community level are derived from the domestic sources compared to the other categories such as industrial, commercial, storm water in case of combined sewers and groundwater infiltration. The quantity and strength of domestic wastewater depends on the size and the socio-economic behaviour of the population constituting the community. In this research pollution concentration distribution of the wastewater over a day at community level was found to be highly variable with an hourly average of 564 mgCOD/L and standard deviation of 230 whereas less variations were found while conducting a similar monitoring exercise of wastewater quality over a day for centralised municipal wastewater discharges. The centralised municipal wastewater discharges had an hourly average of 470 mgCOD_{tot}/L with a standard deviation of 98. The average ratio COD: BOD_5 found for the wastewater at the research area was 1.52 with standard deviation of 0.13. The COD_{tot} hourly average of the wastewater at community level was found to correspond numerically to the COD_{tot} average of 529 mg/L that was obtained on sampling days over a long-term monitoring period (1277 days). In the long-term period the extreme values i.e. the minimum and maximum, wastewater COD_{tot} were observed to be 46.4 and 4109 mg/L respectively. The wastewater with such high variability of pollution concentrations can best be pre-treated by modern high rate anaerobic systems such as Upflow Anaerobic Sludge Bed (UASB) reactors.

Key words – wastewater characteristics, anaerobic treatment, sanitation, DESAR.

Nomenclature-		
	UASB	upflow anaerobic sludge blanket
	COD_{tot}	chemical oxygen demand-total (mg/L)
	CODss	chemical oxygen demand-total suspended solids (mg/L)
	COD_{col}	chemical oxygen demand-colloidal (mg/L)
	COD_{sol}	chemical oxygen demand-soluble (mg/L)
	SS	suspended solids (g/L)
	BOD	Biochemical oxygen demand
	DESAR	Decentralised sanitation and reuse
	l/s	Litres per second

2.0 Introduction

In general wastewater is characterised in terms of its physical, chemical, and biological composition. However the most important constituents of these categories of characteristics are those of undesirable properties and usually are the ones liable for removal in a wastewater treatment plant. The objectives of sewage treatment include the removal of suspended solids and organic material (Haandel, *et, al.,* 1994). The wastewater generation in a community may constitute wastewater generated by: domestic, industrial, commercial, storm water in case of combined sewers and groundwater infiltration. This research is addressing the wastewater generation at community level and the feasibility of treatment on site. The major constituents of the wastewater at community level are derived from the domestic sources compared to the other mentioned categories.

The domestic sewage is composed of toilet wastewater (black water) and household wastewater, i.e. sullage from the kitchen and bathroom (Haandel, *et, al.,* 1994). The quantity and strength of domestic wastewater depends on the size and the socio-economic behaviour of the population constituting the community. These factors influence the design of the treatment plant, particularly the size of the plant. In Dar es salaam, Tanzania where this research was undertaken the average water consumption in the year 2000 was estimated at about 60 litres per capita per day (l.c.d) (Elmcrest group, *et, al.,* 2000). Less water consumed means less wastewater discharged and less volume of the wastewater to be treated in such communities.

This research aims at identifying existing most important constituents of wastewater characteristics in a research area that are of undesirable properties and amenable for removal in an anaerobic pre-treatment unit such as Upflow Anaerobic Sludge Bed (UASB) reactor which is a modern high rate anaerobic treatment system.

2.1 Materials and methods

Research area

The source of wastewater that had to be treated was from the cafeteria, toilets of some students' dormitories and partly from the university staff residential houses belonging to the University College of Lands and Architectural studies (UCLAS), Dar es salaam, Tanzania. The monitoring location for the wastewater was located below the ground level in order to facilitate natural gravity flow mode for the wastewater. The wastewater hydraulic flow rates and characteristic parameters were measured just before the wastewater entered the treatment units i.e., UASB reactors.

Wastewater characteristics

To characterize the wastewater two different approaches were used. In the first approach the wastewater composition and flow was monitored over a period of 24 hours. The degree of flow rate variation dictated the time interval for sampling, however on the average at about every 25 minutes a grab sample was taken and the flow rate was measured. In addition to the separate grab samples also a composite sample was made. For this composite sample the amount of each individual sample that was added to the total mixture was proportional to the wastewater flow at the time the sample was taken. Samples were analysed for total COD.

In the second approach grab samples were taken twice to thrice a week at 9 a.m. for a period of about three and a half years. All samples were analysed for total COD (COD_{tot}), Suspended COD (COD_{SS}), Colloidal COD (COD_{col}), Soluble COD (COD_{sol}), BOD, and temperature.

The COD was assessed using the micro-method described by Jirka and Carter (1975). The 4.4 µm folded paper-filtered (Schleicher & Schuell 595½) was used for assessment of COD_{fil} and the COD_{sol} using 0.45 µm membrane-filtered samples (Schleicher & Schuell ME 25). The COD_{ss} and COD_{col} were calculated by differences between CODtot and COD_{fil}, COD_{fil} and COD_{sol} respectively. All COD analyses were done at least in duplicate

The BOD was assessed using BODTrak apparatus model: 26197-01 AMR: 1.0 manufactured by HACH company. The measured wastewater sample volumes were placed in BOD bottles of the apparatus. The pre-prepared BOD nutrient buffer pillows also from Hach Company were added and mixed with the wastewater samples in the BOD bottles. The apparatus uses automatic magnetic stirring mechanism that continuously agitates the wastewater samples in the respective bottles so as to enhance oxygen transfer. During the test the BODTrak apparatus uses sensitive sensors that continuously monitor pressure changes due to oxygen uptake by the sample. The instrument converts measurements directly to mgBOD/L. CO_2 produced by the oxidation of organic matter is removed from the BOD bottle by the lithium hydroxide crystals placed in a sealed cup inside each bottle. This ensured that pressure measurements accurately reflected oxygen consumption. The apparatus operates in a closed-end system independently of barometric changes.

Biodegradability assay

One of the most important characteristics of the wastewater is its anaerobic biodegradability. This biodegradability was assessed on day 365 of the research period. The biodegradability test was performed in 1000 ml flaks. To four of these flasks of 550 ml of wastewater and 5 g TS/L of sludge from a UASB reactor (Chapter 6) was added. Subsequently the flasks were filled up to a volume of 800 ml using distilled water. To another four of these flasks only 5 g TS/L was added after which the flasks were filled to 800 ml with distilled water. Latter flaks severed as blanks to account for the gas production from the sludge. The contents of all flasks were flushed with N_2 and they were incubated at 25 ºC. During the incubation the methane production was monitored by liquid displacement method. In which case the displaced liquid was a 15% NaOH solution. The COD from the wastewater sample that was converted to CH_4 was calculated by assuming that 1 ml CH_4 displaces 1 ml NaOH solution and that at 25ºC 1.0 g COD produces 394 ml moist CH_4 (382 ml dry CH_4). Besides that the CODfil and volatile fatty acid contents of the flasks were regularly monitored. The total volatile fatty acids (VFA) concentration was determined by using the titration method on distillates of the samples. The procedure followed is described in the Standard Methods (APHA, *et al.*, 1992). The CODfil was analysed as described above.

2.2 Results and discussion

2.2.1 Wastewater COD & BOD

Fig. 2.1 shows the variation of the COD_{tot} and hydraulic load that was followed over 24 hours on day 537 during the conduct of the research. As found by other researchers, Haandel, *et al*., (1994), the pollution load, COD_{tot}, concentration in Fig. 2.0 is increasing at increasing flow rate and vice versa. The minimum and maximum COD_{tot} observed were 222.5 and 1310 mgCOD_{tot}/L respectively. Table 2.1 shows a comparison between the COD_{tot} followed over a day (i.e. 24h) and the long term COD_{tot} that was followed during the whole research period at sampling times. When the value obtained from the 24-hour volume proportional sample is compared to the value obtained from the average of the samples taken at 9 a.m. it can be noted that the former values are higher than the actual daily influent averages. However, the daily samples taken at 9 a.m. are more suitable to evaluate the performance of the wastewater treatment unit, as this is the period of the day with the highest loading rate (Figure 2.1). From Table 2.1 the minimum and maximum measured COD_{tot} over a monitoring day are very different to the minimum and maximum COD_{tot} measured on long-term basis. Taking into account the observed big disparity in the extreme values for the minimum and maximum COD_{tot} for the two scenarios of monitoring durations, a wastewater treatment plant at community level is expected to experience more severe extreme values than what can be observed during a monitoring exercise over a day.

Table 2.1:Comparison between the COD_{tot} concentrations monitored over a day (24h) and those monitored on long-term basis. Standard deviations are presented in brackets

Monitoring duration	Characteristic	Average	Minimum	Maximum
Over a day monitoring (24h) at research site. (Grab samples were taken at about every 25 minutes)	COD_{tot} (mg/L)	564.35 (230.4)	222.5	1310
	24h Composite sample COD_{tot} (mg/L)	410.9 (26.5)		
1277-day monitoring duration at research site. (Grab samples were taken twice to thrice in a week at about 9 a.m. and analysed in duplicate at the laboratory)	COD_{tot} (mg/L)	529.4 (544.6)	46.4	4108.8
	CODss (mg/L)	264.4 (448.4)	2.5	3840.7
	BOD5 (mg/L)	430.9 (146.6)	146	734
	COD: BOD5	1.52 (0.13)	1.18	2.53
	Temperature (°C)	28.1 (1.58)	24.6	34.30

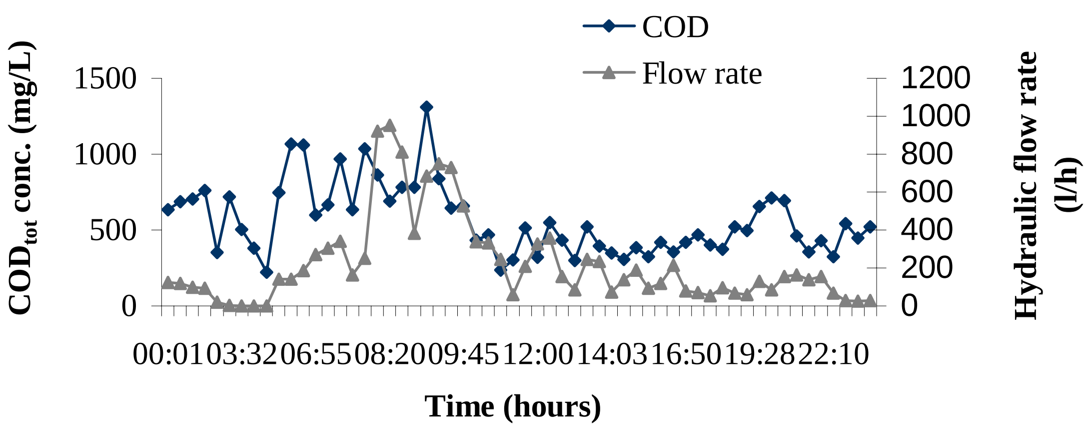


Figure 2.1:Variation of the CODtot concentration and hydraulic load over a day on day 537 of the research period

Figure 2.3 shows the long-term variations of the wastewater COD_{tot} and distinguished COD fractions observed at community level in the research area. The concentrations of COD_{tot} and COD fractions are highly variable at community level.

Biodegradability of the wastewater

The average ratio COD: BOD_5 found for the wastewater at the research area is 1.52 (0.13), (Table 2.0). This ratio is within the range of the typical untreated domestic wastewater that varies from 1.25-2.5 (Metcalf and Eddy, 1991). However since the ratio is close to the lower bound of 1.25, it indicates that the wastewater in the research area is highly biodegradable, which is expected of the wastewater quality at community level.

Results of an anaerobic biodegradability assay are depicted in Figure 2.2. The initial COD_{tot} in the biodegradability assay was 370 mg COD/L. The initial amount of COD_{fil} in the assay was 332 mg COD/L. The results in Figure 2.2 show that during the period of 36 days all of the filtered COD was removed, 238 mg COD/L was converted to methane. The rest of the removed filtered COD was used for biomass generation and probably flocculated to particles larger than 4.4 µm. The latter was also observed by Elmitwalli *et al*. (2001). The biodegradability of the wastewater based on the amount of total COD that was converted to methane amounted 64%. The COD:BOD_5 ratio indicated an aerobic biodegradability of 66%, which compares well to the anaerobic biodegradability.

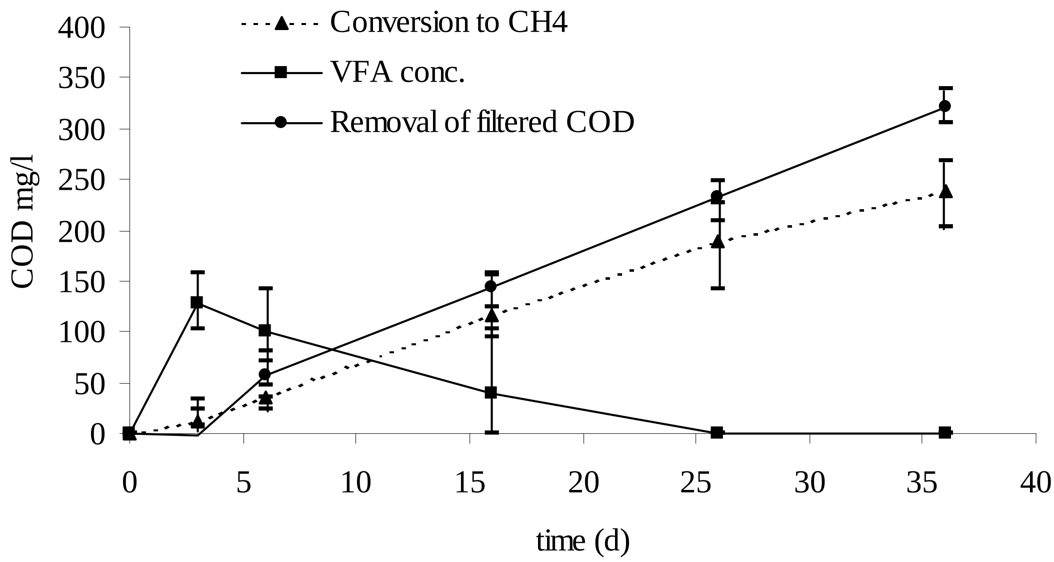


Figure 2.2: Fate of the wastewater COD during the anaerobic biodegradability assay. The initial COD_{tot} for the wastewater was 370 mg COD/l.

2.2.2 Comparison of observed wastewater characteristics at community and municipal levels

Figure 2.4 shows those 24h variations of COD_{tot} of the municipal wastewater and the accompanied hydraulic loads. The COD_{tot} variations of municipal wastewater are less extreme having a standard deviation of 98 (Table 2.2); while at the community level the variations are more extreme with the standard deviation of 230.4 (Table 2.1). Table 2.2 presents the assessed value of the selected wastewater characteristics,

COD_{tot}, and temperature monitored over a day for raw wastewater discharged from the main sewerage system located at ‘Screen House’ Dar es salaam metropolis, Tanzania. COD_{tot} at municipal level shown in Table 2.2 are more diluted and of less extreme in the lower and upper bounds compared to community level wastewater pollution concentration levels shown in Table 2.1, both wastewater discharges were monitored over a day period. This means that the treatment of wastewater at community level is expected to handle more concentrated wastewater than at central municipal wastewater treatment plant. Unlike at community level where the pollution loads, COD_{tot}, concentration were observed to increase at increasing hydraulic flow loads and vice versa (Fig. 2.1), the situation is quite the opposite at the discharge point ‘Screen House’ of the centralised municipal sewerage system for Dar es salaam metropolis, Tanzania. Fig. 2.4 shows that the pollution concentrations are not in phase with the increasing or decreasing hydraulic flow loads. Possible explanation for the observed increase and decrease in COD_{tot} concentration with respect to decrease and increase of hydraulic load might be infiltration of groundwater and that the central sewerage system appears to receive a lot of water from daily commercial and trade activities that do not generate high pollution loads as compared to activities at domestic level. Such relatively less polluted wastewaters emanating from groundwater infiltration and trade and/or commercial discharges do dilute wastewater more in the sewerage system. This finding confirms the observations made by other researchers that at decentralised treatment of domestic sewage total COD concentration is expected to be high in comparison with wastewater at central treatment plant as a result of minimal diluted wastewater arising from groundwater infiltration and / or rainwater, and the shorter retention time in the sewer (Zeeman, *et al.*, 1999, 2001). Reijnders, (2001), points out that centralised sanitation systems take in a variety of waste streams whereas Decentralised Sanitation and Reuse (DESAR) systems may be specialised, as for this case the wastewater at community level is more special in the sense that it is mostly of domestic origin.

Table 2.2: Observed selected characteristics of raw wastewater discharges monitored at ‘Screen House’ location over a day from the main sewerage system of Dar es salaam metropolis, Tanzania - (January 17, 2000). Standard deviations are presented in brackets.

Characteristic	24h-average	Minimum	Maximum
CODtot (mg/L)	470.3 (98.0)	289.0	656
Temperature (^{o}C)	29.8 (0.20)	29.0	30.0

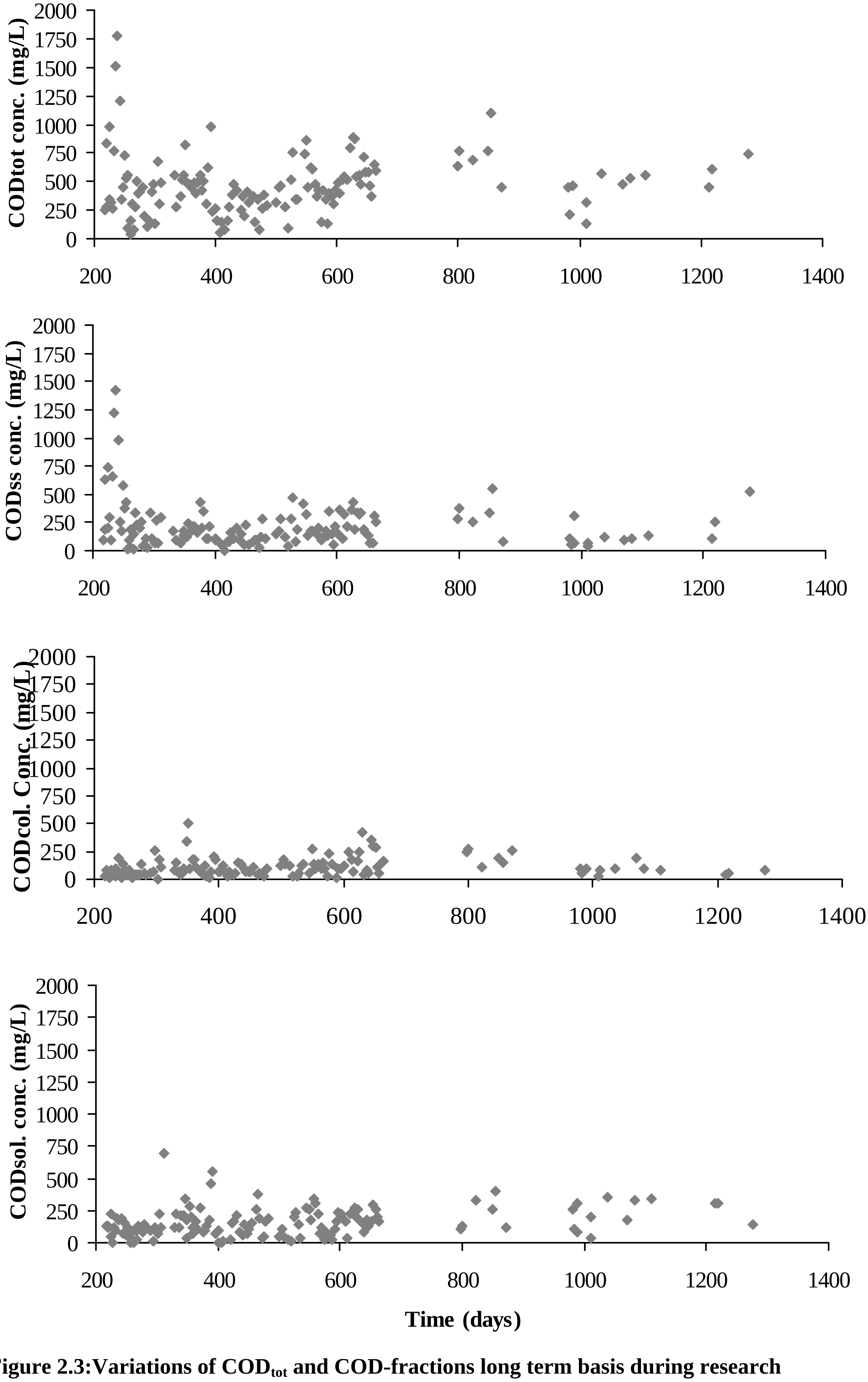


Figure 2.3:Variations of COD_{tot} and COD-fractions long term basis during research

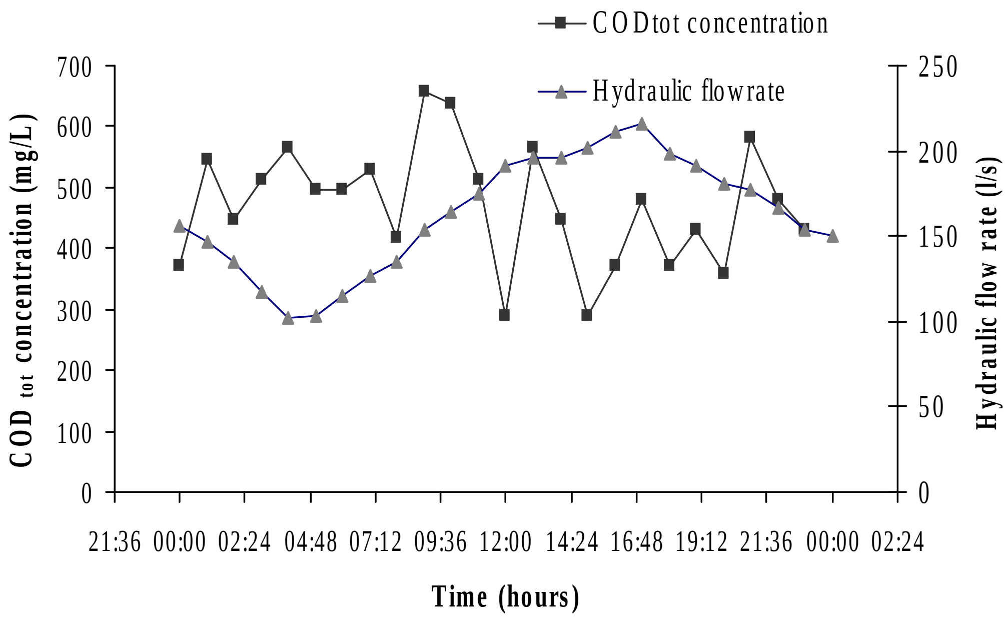


Figure 2.4: Observed fluctuations of COD_{tot} and hydraulic flow rate of raw wastewater at 'Screen House' discharged from main sewerage system for Dar es salaam metropolis, Tanzania – January 17, 2000

2.3 General discussion and conclusion

The results of this research reveal that the wastewater at community level in the research area, mainly of domestic origin is highly biodegradable with a COD/BOD_5 ratio of 1.52. The variation of COD_{tot} over a day at community level is high and has very extreme values between the minimum and maximum compared to similar observations made on the concentrations of centralised municipal wastewater. These variations constitute the typical characteristics of wastewater at community level. Small system flow rates and wastewater characteristics differ significantly from those of large systems (Metcalf & Eddy, 1991). The wastewater treatment plant to be installed on site at community level should be able to withstand these great variations in constituent concentrations and hydraulic loads without adverse effects on the effluent quality. The suspended solids and organic (biodegradable) material being the main constituents of sewage at community level can be highly reduced in concentration using modern high rate anaerobic pre-treatment units such as conventional Upflow Anaerobic Sludge Bed (UASB) reactors. Compared to clarigesters, the UASB reactors perform better with regard to the removal of COD and BOD and at least equally with respect to TSS (Wiegant, 2001). Apart from a satisfactory treatment efficiency in terms of COD, BOD and TSS both for black water, as well as for combined black and gray water, the anaerobic reactor in an onsite system also will provide a high stabilization of solids, while in specific designs a sufficiently high sludge hold-up can be realised to allow discharge of excess sludge only once every three to four years (Lettinga, 1996). The advantages of anaerobic over aerobic treatment systems that can be installed at community level are shown in Table 2.3. The anaerobic wastewater treatment systems do not need energy to operate and they can withstand interruption of substrate supply for a longer period compared to aerobic systems (Haandel & Lettinga 1994). Since UASB reactors are high rate anaerobic systems even at peak times of substrate supply they should be able to perform satisfactorily. However at very high upflow velocities, over 1.0 to 1.5 m/h (i.e. mostly happening at peak times in case of gravity flow operation mode) inclusion of a polishing step after a UASB reactor may be justifiable (Wiegant, 2001).

Table 2.3:Comparison of some aspects of anaerobic and aerobic treatment systems

Parameter	Aerobic system	Anaerobic system
Energy required (W/kgCOD d)	20-30	-35*
Sludge production (kgVSS/kgCOD)	0.2-0.3	0.05-0.15
Nature of excess sludge	Unstable	Stable
Interruption substrate supply	<2 weeks	Several months

(Source Haandel, *et al*., 1994)

*Anaerobic systems have the potential for energy production

2.4 References:

American Public Health Association (APHA); American water and works Association; Water Pollution and Control Federation, *Standard methods for the Examination of Water and Wastewater* (1992).

Elmcrest group and MMK *Project Services LTD (2000).Proposed Dar es salaam water supplu and sanitation project*. Draft design report.

Elmitwalli T. A., Soellner J., de Keizer A., Zeeman G., Bruning H. and Lettinga, G. (2001). Biodegradability and change of physical characteristics of particles during anaerobic digestion of domestic sewage.*Water Res* **35**(5):1311-7

Haandel A. C. van and Lettinga G. (1994). *Anaerobic sewage treatment-A practical guide for regions with a hot climate*. John Wiley & Sons Ltd. England.

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

Metcalf and Eddy (1991). *Wastewater Engineering:Treatment, Disposal, and Reuse.* 3rd edn. Revised by Tchobanoglous, G. and Burton, F.L., McGraw Hill, Inc.

Reijnders R. (2001). The environmental impact of decentralised compared to centralised treatment concepts. In:*Decentralised Sanitation and Reuse,* eds.Lens, P., Zeeman, G. and Lettinga, G. IWA Publishing, London.

Wiegant, W.M. (2001). Experiences and potentials of anaerobic wastewater treatment in tropical regions. *Wat. Sci. Tech.,* **44**(8), 107-113

Zeeman, G. and Lettinga, G. (1999) The role of anaerobic digestion of domestic sewage in closing the water and nutrient cycle at community level. *Water Sci. Tech.* **39**(5), 190-91

Zeeman, G. and Sanders, W. (2001) Potential of anaerobic digestion of complex waste (water) *Water Sci. Tech.* **40**(8), 115-122

3 INTERPRETATION OF ANAEROBIC SLUDGE STABILISATION ASSAYS FOR COMPARATIVE STABILITY ASSESSMENT

Abstract –Stability of sludge is expressed as the amount of methane that is produced per gram VSS (i.e. mlCH_4/gVSS) that was initially incubated. Batch reactors coupled with "serum bottle liquid displacement system" have always been used while determining the stability of anaerobic sludge assay. However while evaluating literature a notice has been made that different approaches have been used in determining the stability of anaerobic sludge assays. These include: 1) with or without addition of nutrients and trace elements, 2) different incubation periods of sludge sample assays and 3) different incubation temperatures. These differences have made interpretation of results of anaerobic sludge stabilisation assays difficult to compare since they have not been conducted on a common platform. The proposed method for interpretation of anaerobic sludge stabilisation assays is based on linearisation of cumulative methane production curve. The methane production is derived from digestion of degradable components and decay of biomass all of which have been part of the organic volatile solids of the sludge. The processes underlying the digestion of degradable components and decay of biomass follow first order rates. This means that linearisation of the cumulative methane production curve is only possible after the degradable component of the sludge has been exhausted and that what remains is the decay of the biomass alone. From the linearised portion, the first order constant (kd), i.e. in this case representing only the decay of biomass alone, can be obtained. After knowing the decay constant (kd) of the biomass, then the sludge stability assay components viz., degradable organics (Xdegr), biomass (Xbm) and inert organics (Xi) can be retrieved through model simulation for the sludge stabilisation process using Aquasim 2.0 (Reichert, 1998). With this approach interpretation of sludge stability assays of anaerobic sludge from different origins can be compared. When this method was applied to sludge stability assays for sludge samples withdrawn from UASB reactor, septic tank and pit latrine the results showed that the sludge from the UASB and septic tank were fully stabilised whereas the sludge from the pit latrine still contained a considerable amount of degradable organics.

Key words **–** stability assay, hydrolysis, decay, biomass, inerts, biodegradable, model

Nomenclature-	Xbm	Suspended solids present as biomass
	Xdegr	Biodegradable suspended solids
	Xi	Inert suspended solids
	VFA	Volatile fatty acids
	S_{CH4}	Methane
	Hydrolysis	Process in which biodegradable suspended solids are converted to soluble organics components
	Decay	Process which incorporates death, lysis and hydrolysis of biomass

3.0 Introduction

3.0.1 Background

All conventional wastewater treatment processes produce large quantities of wastewater material in the form of dilute solids mixtures known as sludge. The composition, solids content and stability of the sludge is a function of the characteristics of the raw wastewater flow and the treatment process background that generated the sludge. In this regard, primary sludge produced during the treatment of municipal wastewater consists primarily of solid particles of predominately organic nature, whereas secondary sludge consists mainly of excess biomass generated as a result of organic removal in the biological process (Benefield, *et al*. 1980)

Anaerobic digestion of sludge is one of the technologies available for sludge stabilisation. The objectives of sludge stabilisation are to, i) reduce pathogens ii) eliminate offensive odours, iii) reduce liquid volume (moisture content) of sludge and iv) inhibit, reduce, or eliminate potential for putrefaction. This implies that qualities of a more stable sludge are that it has: less pathogens, less offensive odours, less moisture content (i.e. well dewatered) and most important of all the sludge must have least potential for putrefaction. The success in achieving these objectives is related to the effects of the stabilisation operation or process on the volatile or organic fraction of the sludge (Metcalf & Eddy, 1991, Benefield, *et al*. 1980). That is a stable sludge is the sludge that has least accumulated organic fraction in a non-stabilised form.

The success in achieving these qualities for the sludge is related to the effects of stabilisation operation or process on the volatile or organic content of the sludge. Biological reduction of volatile content of the sludge is one of the means that eliminates nuisance-odour conditions and the potential for putrefaction. As the duration of experiment run has a great impact on the calculated value for the stability of the sludge this chapter aims at presenting a method that interprets the anaerobic sludge stabilisation assays in terms of composition of anaerobic sludge.

3.0.2 Experimental set-ups for the assessment of stability of sludge

In general the anaerobic stability test is done by incubation of a sludge sample over a long period of time during which the methane production from the sample is followed. After termination of the experiment the stability of the sludge is expressed as the amount of methane that is produced per gram VSS that was initially incubated.

The stability of the sludge then should give an indication of the accumulated organic fraction (non-stabilised) of the sludge. It should be pointed out here that this approach only refers to conversion of organic matter and not to pathogen removal.

When evaluating literature, the evaluation clearly showed that different approaches have been used in determining the stability of anaerobic sludge. Table 3.1 shows the main differences in the approaches that have been used in literature.

Table 3.1: Experimental set-up of stability test reported in literature

Authors	Inoculum	Nutrients/ Trace elements	Duration of experiment	Temp
Mahmoud (2002)	Yes	Yes	44-95 days	25°C, 35°C
Haandel v. and Lettinga (1994)	Yes	No	One month	25 ±2°C
Halasheh (2002)	Yes	Yes	Approx. 70 days	33°C
Lettinga, *et al.*, (1991)	Yes	No	100 days	Ambient

In all of the reported approaches the methane production was measured via the "serum bottle liquid displacement system" Figure 3.1 shows a schematic diagram of the experimental set-up, (method described by Lettinga *et al.*, 1991). The displaced liquid is a strong solution of NaOH 15 g/L. As the biogas passes through a high pH solution of NaOH, the CO_2 of the biogas is converted to carbonate and absorbed into the liquid. Only the methane gas passed through the solution and an equivalent volume is pushed out of the top serum bottle. The displaced liquid is measured in a graduated cylinder as a volume. The less the volume of methane gas produced per unit weight of sludge the more stable the sludge is.

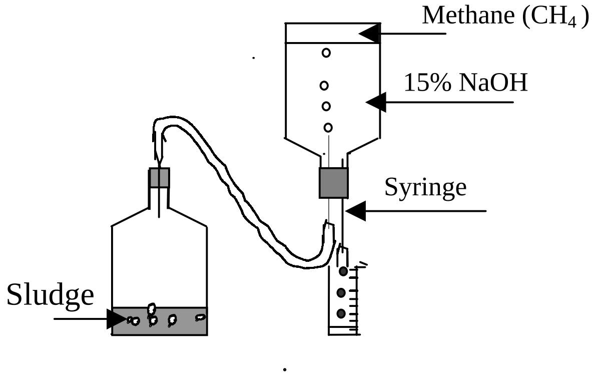


Figure 3.1: A schematic diagram of batch reactors used in determining sludge stability by "serum bottle liquid displacement system"

The stability of sludge is expressed in terms of amount of methane gas produced during the entire period of the stability test (usually for 30 days) per gram of Volatile Suspended Solids (i.e. ml CH_4/g VSS).

From Table 3.1 it can be observed that some experiments are performed with addition of nutrient and trace elements. However, this is not necessary as the aim of the stability test is not to obtain maximum growth rate but to see the behaviour of the sludge when it is disposed of. The experiment should therefore be carried out with the existing nutrients and trace elements of the sludge.

In the different approaches reported in literature the stability experiments were terminated after different periods of experimental run. Some experiments were carried for 100 days and others only for one month. Guidelines or procedures for termination of a stability experiments are not clear. Some authors suggest continuing the experiment until the biogas production has reached it endogenic level of about 2 ml CH_4/g VSS/day (Haskoning, 1989). At the endogenic level all the degradable components that were initially present in the sludge are degraded and the only source of biogas production comes from the decay of biomass. However, the suggested endogenic gas production of 2 ml CH_4/g VSS/day is only arbitrary as it depends very much on the ratio between biomass and organic inerts in the sludge. As the duration of experiment run has a great impact on the calculated value for the stability of the sludge this chapter aims at interpreting the stability assays in terms of composition of anaerobic sludge.

3.0.3 Proposed method for interpretation of sludge stability assays

As discussed in section 3.0.2 it is very difficult to compare results of stability assays reported in literature because the results very much depend on the time, which the sludge is allowed to stabilise. In this section a method for interpretation of stability assays is presented which will give a better insight in the composition of the anaerobic sludge. Moreover which will allow better comparison of sludge from different origins.

In Figure 3.2 a schematic representation of the (simplified) biological processes during the anaerobic stabilization of sludge is given. The organic volatile solids in the sludge initially constitute biomass, degradable components and inert organic components. During the digestion the degradable components are converted to biogas and new biomass. The biomass decays and the biodegradable components in the cell material is converted to volatile fatty acids. Part of the cell material will contribute to the fraction organic inerts.

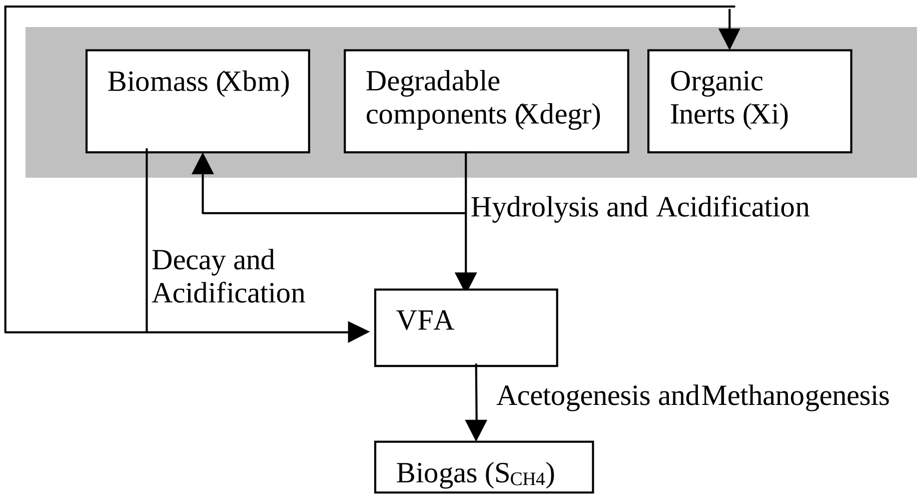


Figure 3.2: Schematic representation of the biological processes during the stabilization of anaerobic sludge

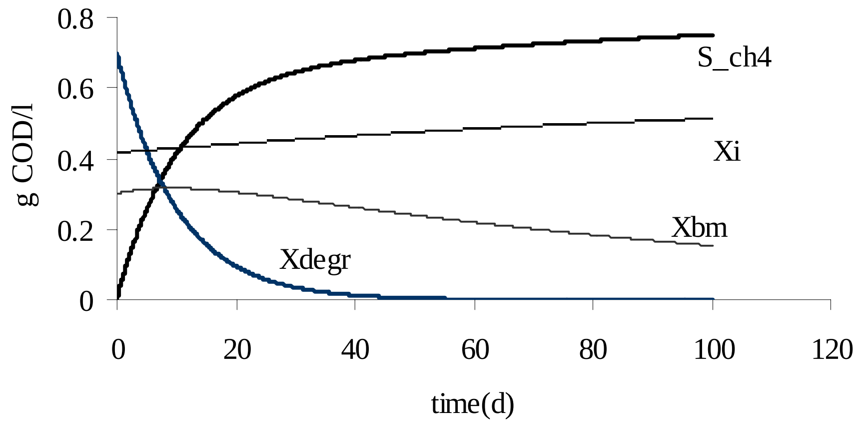
For interpretation of the stability assays the process scheme in Figure 3.2 is put into a mathematical model. The matrix of this model with the processes, stochiometry and rate are presented in Table 3.2. In this table it can be seen that only two processes are modelled, hydrolysis of degradable components and decay of biomass. These are considered to be the two processes which can be rate limiting during a stability assay. Both of them are assumed to have first order rates, but the first order decay constant is always smaller than the hydrolysis constant. Based on the literature review of hydrolysis constants done by Batstone *et al.*, (2002) it is assumed that value of the first order hydrolysis constant for the biodegradable solids in the anaerobic sludge will be lower than 1.0. Furthermore the model assumes that 10% of the hydrolysed COD will be converted to biomass, the rest will proceed to methane. Upon decay 50% of the biomass will contribute to the fraction inert organic organics, 50% will proceed to methane. These values are arbitrary as the reported values in literature vary widely (Batstone *et al.*, 2002).

Table 3.2: Stochoimetry and rates of the processes during stability assays

Process/Components	Xdegr	Xbm	Xi	S_{CH4}	Rate*
Hydrolysis	-1	0.1		0.9	kh.Xdegr
Decay		-1	0.5	0.5	kd.Xbm

*With kd<kh and kh< 1.0 1/d.

The model in Table 3.2 was used to simulate the course of the different sludge components and the production of methane during an anaerobic stability assay (Figure 3.3, upper part). For this simulation it was assumed that the sludge sample initially constituted 1.42 g COD (1 g VSS/L) in total with 0.7, 0.3 and 0.42 g COD/L of degradable organics (Xdegr), biomass (Xbm) and inert organics (Xi), respectively. The hydrolysis and decay constants were 0.1 d^{-1} and 0.01 d^{-1}, respectively. The simulation was done using Aquasim 2.0 (Reichert, 1998). In Figure 3.3 it can be observed that Xdegr is completely reduced within 45 days and this initially results in a small increase of Xbm. After 100 days of stabilisation the sludge sample consist of 0.15 g COD/L of biomass and 0.51 gCOD/L of inert organics.



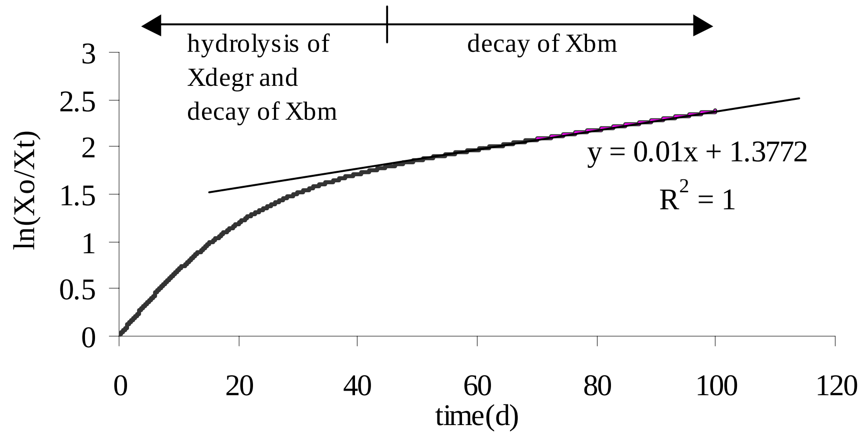


Figure 3.3: Course of Xdegr, Xbm, Xi and S_{CH4} during the simulation of the anaerobic stabilisation of 1.42 g COD (1 g VSS) of sludge (upper part) and the linear representation of the cumulative methane production in the same experiment (lower part). With Xo = total degraded Xdegr + Xbm in experiment, Xt = remaining degradable Xdegr + Xbm at any time during experiment.

Because a model has generated the results in Figure 3.3 the components of the sludge sample can be followed throughout the stability assay. In a ‘real life’ laboratory experiment of course only the production of methane from the sample can be followed. However, the model in Table 3.2 can in that case be used to retrieve much information from the curve of the cumulative methane production. This can be done by linearising the methane production curve as presented in Figure 3.3 (lower part). If the stabilisation of the sludge sample had progressing according to a single first order process (e.g. only decay) the line in Figure 3.3 (lower part) would have been completely straight. However, in the first part of the stabilisation assay two first order processes take place (hydrolysis and decay) moreover the hydrolysis process contributes extra biomass to the decay process. Therefore the first part of the line (until approx. day 44) is difficult to interpret. In the last part of the stabilisation assay only decay takes place resulting in a straight line from which the first order decay constant can easily be obtained. If the decay constant (kd) of the biomass is known kh and the Xdegr, Xbm at the start of the stability assay can be estimated by a simulation software (e.g. Aquasim (Reichert, 1998)) using the model in Table 3.2. In section 3.1 and 3.2 the proposed method for interpretation of sludge stability assays is illustrated by stability assays of anaerobic sludge from different origin.

3.1 Materials and methods

Origin of the sludges

The sludge samples for the tests were withdrawn from the third compartment of three compartment full-scale septic tank treating mixed sewage located at the university in Dar-es-Salaam, Tanzania. Specifications of the septic tank are found in Chapter 4. The second sample was taken from the bottom of the UASB reactor discussed in Chapter 5. The third sample was sludge taken from the bottom of a full scale Pit Latrine in the community. Sludge samples were incubated in serum flaks at average temperature of 25 ºC. Volume and sludge concentrations are shown in Table 3.3. No inoculum was added. The methane production was followed using the serum bottle liquid displacement method as described above.

Table 3.3:Composition of batch experiments for stability tests

Batch constituent	UASB	Pit latrine	Septic tank
Sludge volume (ml)	400	400	400
VSS (g/batch)	9	20	13
COD (g/l)	32	71	46

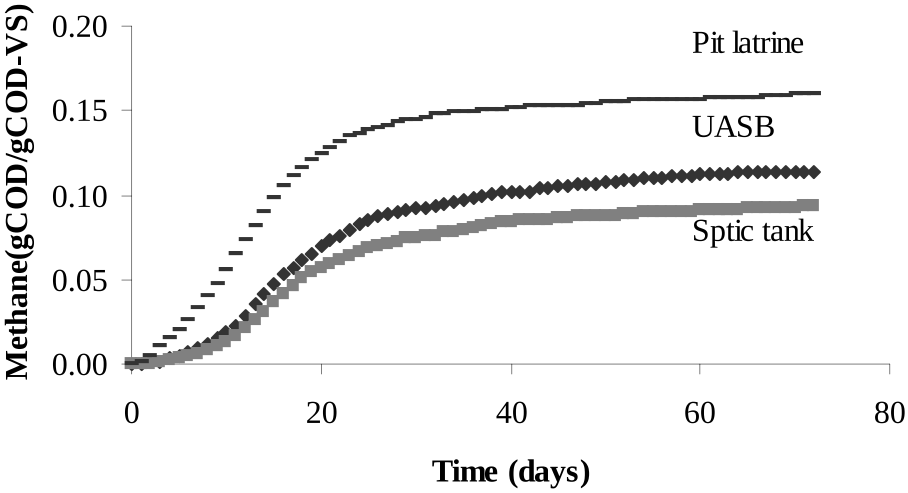


Figure 3.4: Cumulative methane production in the stability assays conducted with sludge from the Pit latrine, UASB and Septic tank.

3.2 Results and discussion

Figure 3.4 shows the cumulative methane production in the stability assays conducted with sludge from the Pit latrine, UASB and Septic tank. The values for the first order decay rate (kd) were obtained from the linearized cumulative methane production (Figure 3.5). These were 0.055, 0.047 and 0.024 d^{-1} for the UASB, Septic tank and Pit latrine sludge, respectively. The kd values were subsequently used to estimate the other parameters of the stabilisation model (Table 3.2). The estimation of kh and the concentration of biomass and degradable components in the original sludge samples were done by fitting the stabilisation model to the respective cumulative gas productions (Figure 3.6) using Aquasim 2.0 (Reichert, 1989). The amount of inert organics was calculated with equation 1. The estimated kh and the percentages of biomass, degradable and inert organics of the respective sludge are presented in Figure 3.7.

$$COD_{inert} = COD_{total} - COD_{biomass} - COD_{degradable} \qquad (1)$$

From Figure 3.7 it can be observed that the sludge from the septic tank and UASB reactor don't contain any degradable components and all the methane that was produced during the stability assay originated from the decay of biomass. The results showed that the sludge from the UASB and septic tank contained 21 and 18% biomass, respectively. No degradable solids were detected in these sludges indicating that the sludge from the UASB and septic tank were fully stabilised and all the methane that was produced during the stability assay originated from the decay of biomass. The lower percentage of biomass in the septic tank sludge as compared to the UASB sludge can be attributed to the higher loading rate of the UASB reactor as compared to the septic tank system. The Pit Latrine sludge did not contain a significant amount of biomass. The amount of degradable components in this sludge constituted 17% of the sludge. However the hydrolysis constant calculated for these degradable solids is almost identical to the decay constant of the biomass in the septic tank and UASB sludge. Indicating that these were very slowly degradable solids possibly pieces of clothes or otherwise used by the latrine users to clean themselves. The decay rate that was calculated for this sludge is to be assigned to the biomass that is produced during the stability assay. For the calculations assumptions were made with respect to the amount of inert organics produced from the decay of biomass and the production of biomass from the digestion of degradable organics. These assumptions affect the outcome of the calculations meaning that the percentages of biomass and degradables in the sludge are not necessarily the true existing values. However, because all samples go through the same procedure the comparison of the samples is valid.

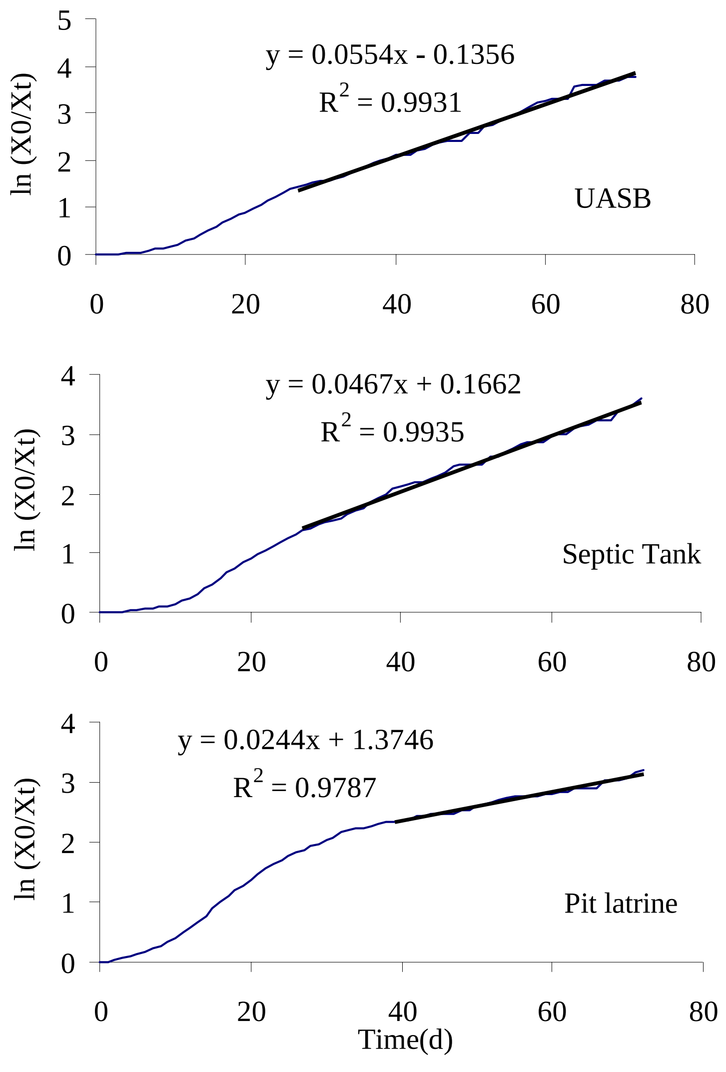


Figure 3.5: The linearized cumulative methane production of the UASB, Septic tank and Pit latrine sludge

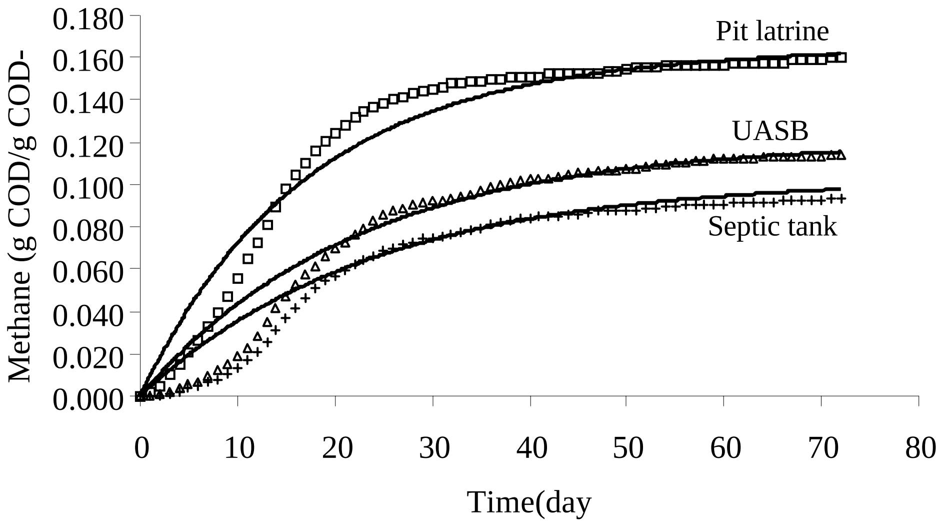


Figure 3.6: Results of the fit of the stabilisation model to the methane production of the sludges.

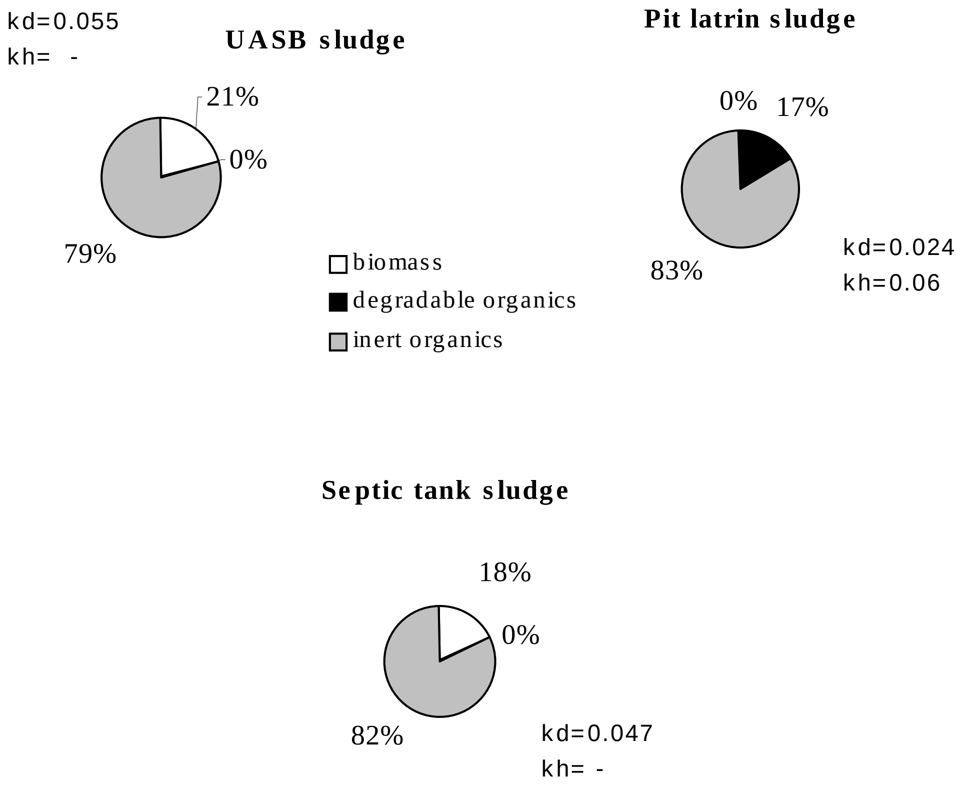


Figure 3.7: Composition of the sludge samples as estimated with the stabilisation model.

3.3 Conclusion

The method for interpretation of sludge stability assays proposed in this chapter provides means of comparing sludge from different reactor systems in terms of sludge composition. The results of the calculations obtained when applying the method to the sludge samples from different origins, viz., sludge withdrawn from UASB reactor, septic tank and pit latrine, showed that the sludge from the UASB and septic tank were fully stabilised whereas the sludge from the pit latrine still contained a considerable amount of degradable inerts.

3.4 References

Batstone D. J., Angelidaki I., Kalyuznhyi S.V., Pavlostathis S.G., Rozzi A., Sanders W.T.M., Siegrist H., Vavilin V.A. (2002). Anaerobic digestion model no.1, Scientific and Technical Report No. 13, IWA publishing, London

Benefield Larry D., Randall Clifford W. (1980) Biological Process Design for Wastewater Treatment. Prentice-Hall, Inc., Book Company. pp.457-458.

Haandel A. C. van and Lettinga G. (1994). Anaerobic sewage treatment-A practical guide for regions with a hot climate. John Wiley & Sons Ltd. England.

Halalsheh, Maha, M. (2002).Anaerobic Pre-treatment of strong sewage. A proper solution for Jordan. *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen.

Haskoning, (1989). Royal Dutch Consulting Engineers and Architects. Anaerobic treatment of domestic wastewater under tropical conditions-final report.

Lettinga G., Knippenberg K. van, Veensta S., Wiegant W. (1991) Upflow Anaerobic Sludge Blanket (UASB): Low cost sanitation research project in Bandung/Indonesia. Final Report. Wageningen Agricultural University, February 1991

Mahmoud, Nidal, J. (2002) *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen

Metcalf and Eddy, Inc. (1991) Wastewater Engineering Treatment/Disposal/Reuse, McGraw-Hill Book Company, pp. 810-81.

Reichert, P. (1998), Aquasim 2.0 a Computer program for the identification and simulation of aquatic systems, Swiss Federal Institute for Environmental Science and Technology (EAWAG), September 1998.

4 PERFORMANCE OF A THREE-COMPARTMENT SEPTIC TANK (CAPACITY: 56.3 M^3; HEIGHT: 1.93 M) TREATING TROPICAL DOMESTIC WASTEWATER

Abstract – A full-scale community onsite septic tank constituting three-compartments was monitored for its performance for a duration of 400 days. The septic tank was constructed in 1982 and treats wastewater from one of the hostel flats accommodating students at a University College of Lands and Architectural studies (UCLAS), Tanzania. The wastewater originates from the Hostel toilets, washing stalls and some apartment kitchens. The effective volume of the septic tank is 56.3 m^3. The average influent organic pollution of wastewater for COD_{tot} and COD-fractions, viz. COD_{ss}, COD_{col} and COD_{sol} were respectively (standard deviations in brackets) 471 (311), 176 (291), 101 (85) and 204 (131). The performance of the full scale septic tank during the monitoring period was poor. The removal efficiency for COD_{tot}, COD_{ss}, COD_{col} and COD_{sol} were respectively 29, 29, 16 and 16 percent. Washouts of COD fractions were often observed. The average hydraulic retention time (HRT) observed during the periods of taking samples was 5.6 (8) days. The observed poor performance of the septic tank despite the high HRT is mainly attributed to possible high accumulation of sludge in the compartments leading to hydraulic scouring of the settled solids. Adhering to recommended desludging interval of septic tank compartments ensure best performance in accordance to design specifications. However the fact still remains that with septic tanks the theoretical HRT becomes shorter with time of the septic tank operation, when the accumulated sludge occupies more and more septic tank space and there is also poor contact between influent substrate and the accumulated biomass in the compartments. These short comings together with the relatively high demand of land area and construction costs makes the septic tank installation at community level less attractive compared to other wastewater pre-treatment options such as community onsite UASB treatment units.

Key words – septic tank, domestic wastewater, anaerobic treatment, sanitation.

Nomenclature –

UASB	upflow anaerobic sludge blanket
COD_{tot}	chemical oxygen demand-total (mg/L)
CODss	chemical oxygen demand-total suspended solids (mg/L)
COD_{col}	chemical oxygen demand-colloidal (mg/L)
COD_{sol}	chemical oxygen demand-soluble (mg/L)
SS	suspended solids (g/L)
BOD	Biochemical oxygen demand
DESAR	Decentralised sanitation and reuse

4.0 Introduction

Cameron in England developed septic tank around the change of the century (Haandel and Lettinga, 1994). In the septic tank sewage flows through in the upper part while settleable solids settle out and are retained in the bottom of the tank. The settled solids are degraded by anaerobic sludge (Haandel and Lettinga, 1994). The efficiency of the septic tank depends to a great extent on the retention time. Other factors that affect the performance of the septic tanks are: ambient temperature, the nature of the influent wastewater, the organic matter content in the wastewater and the position of the inlet and outlet in the septic tank (Polprasert, *et al*., 1982).

Septic tanks are the most widely used community on-site wastewater treatment units in Tanzania. However for individual residential dwellings septic tanks are second to pit latrines the most used treatment facilities for domestic wastewater. Septic tanks are also widely used in other parts of the world. Viraraghavan, (1976) reported that about 50,000,000 Americans and 4,000,000 Canadians use septic tank systems for household sewage disposal. Septic tanks provide more than two million Australians with a means of treating domestic wastewater (Geary, 1994). Despite the design shortcomings septic tanks have been perceived as low-cost or water-borne waste anaerobic treatment plant for management of sewage of individual household or collection of households at community level.

The objective of this research is to examine the treatment efficiency and performance limitations of a community full-scale septic tank for the purpose of determining its suitability as a wastewater treatment option at community level. The treatment efficiency of a three-compartment, 56.3-m^3 effective volume community on-site full-scale septic tank constructed at the University College of Lands and Architectural Studies (UCLAS), Tanzania has been monitored for 400 days for this purpose.

4.1 Material and methods

4.1.1 Full-scale septic tank

Shape and size

A full-scale, three-compartment, septic tank Figure 4 was monitored for 400 days. The septic tank is of sand-cement brick structure. It receives wastewater from students residential hostel of about 224 people. The wastewater flow was by gravity since the treatment unit was located on down gradient of the hostel. The influent wastewater grab samples were taken from a short open channel just before entering a Tee-inlet to first chamber of the septic tank. The effluent grab samples were taken from an open Polyvinyl chloride (PVC) pipe connected to Tee-outlet leading the effluent out of the septic tank's third compartment. The sludge grab samples were withdrawn from septic tank compartments using a sampler made of plastic bottle cut open on one end and connected to a 2.5 m long handle. The sludge samples were scooped from the bottom in small amounts at a time from several locations and poured into a plastic bottle ready for analysis in the laboratory.

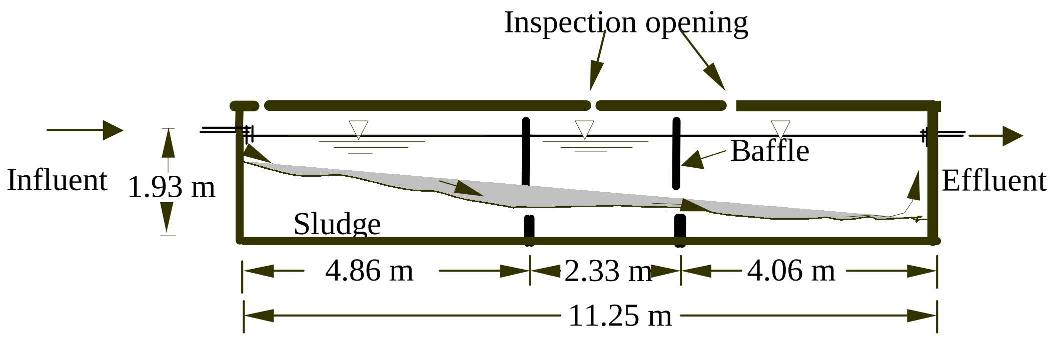


Figure 4.1: Schematic diagram of a Full-scale three-compartment septic tank (Effective volume: 56.3m^3;water width: 2.59 m, depth: 1.93 m, Length: 11.25 m)

Wastewater characteristics

The wastewater fed to the septic tank originated from a different hostel from that described in Chapter 2. However both wastewater streams are of domestic origin. The average characteristics of raw wastewater in terms of COD_{tot}, COD_{SS}, COD_{col} and COD_{sol} influent to the three-compartment septic tank were (with standard deviations in brackets) as 470.8 (311.1), 176.5 (289.1), 101.4 (84.6) and 204.5 (131.3) mg/L respectively.

Sampling and analyses

Sampling of influent and effluent was made (almost) regularly twice to thrice in a week for a period of 400 days. One grab sample was taken usually at about 10 a.m. on every sampling day and analysed in duplicate as described in Chapter 2.

Analysis of solids

The TS, VS and ash content of the sludge bed were analysed according to the *Dutch Standard Normalized Methods (1969).*

Sludge stability assay.

Anaerobic stability tests were conducted to determine putrefaction potential of the volatile (organic matter) content of the sludge i.e. of the volatile suspended solids (VSS). The experimental set up used is described in Chapter 3.

Sludge depth measurement

The sludge depth was measured using wet toweling method (Polprasert, *et al*., 1982). A long stick wrapped with rough white toweling and lowered through inspection openings (Fig. 4.1) to the bottom of the respective compartments showed the depths of the sludge. After a five-minute stay inside the sludge the stick was slowly removed from the reactor. Subsequently the depth of the sludge bed was determined by measuring the part of the toweled stick that was covered by sludge particles.

Start up of the plant reactor.
The septic tank was started without seeding in 1982 and rarely desludged.

4.2 Results and discussion

4.2.1 COD removal efficiency.

The average values for the COD removal efficiencies over 400 days of observations are shown in Table 4.1. The results in Table 4.1 clearly show that there is poor removal for all the COD during the observation period and in fact there is deterioration in treatment efficiency by the septic tank for COD_{tot} and CODfractions with the passage of time. The average CODfraction values in Table 4.1 show low removal for all CODfractions, viz. COD_{ss}, COD_{col} and COD_{sol}. Table 4.1 also shows percentage removal efficiency for CODtot and other parameters as well as the concentration of COD_{tot} effluent as reported by other researches in the identified countries.

Table 4.1:COD influent and effluent concentrations and removal efficiencies for the full-scale three-compartment septic tank at ambient tropical temperature 25-34 °C (Dar es salaam, Tanzania). Standard deviations are presented in brackets. Calculations are based on 91 samples.

	COD_{tot}	COD_{ss}	COD_{col}	COD_{sol}	HRT (days)
Influent conc. (mg/L)	470.8 (311.1)	176.5 (289.1)	101.4 (84.6)	204.5 (131.3)	
Effluent conc. (mg/L)	334.1 (205.3)	126.0 (154.0)	85.4 (87.7)	172.1 (202.4)	5.5 (8.2)
Removal efficiency (%)	29 (26)	29 (54)	16 (21)	16 (21)	

The average COD_{tot} removal efficiency found in this research is 29 percent (Table 4.1), is lower then reported values by other researchers which is in the range 40-47 percent (Table 4.1). The lower COD removal efficiency found in this research can be attributed to the high sludge level in the first compartment that has been subjected to scouring and washing out of settled solids by the horizontal liquid flow motion along with the septic tank effluent. This research it was found that there is almost complete washout of COD_{sol} from the septic tank during the research period. The predominant reason for this washout of the COD soluble fraction from the septic tank is due to little if any contact between the active anaerobic micro-organisms which are mostly available in the settled sludge and the non-settleable part of the organic matter in the influent. The main part of the dissolved or hydrolysed organic matter that could not be metabolised, leaves the septic tank with the effluent (Haandel & Lettinga, 1994). This is part of the inherent fundamental design prescription of the septic tank that has adverse effect on the removal efficiency of the non-settleable organic fraction of the influent sewage. As the sludge depth increases in septic tanks, the effective liquid volume and retention time decreases. From the literature the removal efficiency for COD by the septic tanks is reported to deteriorate when the sludge in the compartment(s) reach a certain level. At this level, sludge scouring increases, treatment efficiency falls off, and eventually more solids escape through the outlet (Polprasert, *et al*., 1982). The only way to prevent this hazard in septic tanks is by periodically pumping out of sludge from the tanks. The existing Canadian and United States of America (USA) septic tank regulations specify a minimum permissible distance between the surface of the sludge and the outlet fitting of the tank. In Canada, according to Ontario Septic Tank (OST) recommendations, the permissible highest level of sludge in 'normal' (i.e., septic tank for household wastewater treatment of a detention time between 2 and 3 days) septic tanks, used for domestic wastewater treatment, is 0.46 m below the bottom of the outlet fitting (Brandes, November 1978). In the USA the highest permissible sludge level is within 20-32 cm of the bottom of the outlet device (Polprasert, *et al*., 1982). Sometimes specifications that require cleaning the septic tank is given in terms of the thickness of scum layer. Polprasert, *et al*., 1982 reported that the scum and sludge accumulations in a septic tank should be inspected once or twice a year. When a tank is inspected, the depth of sludge and scum should be measured in the vicinity of the outlet baffle. The tank should be cleaned whenever the bottom of the outlet scum layer is within 7.6 cm of the bottom of the outlet device. Kalbermatten, *et al*., (1980) recommends that septic tanks must be desludged at regular intervals, usually once every 1 to 5 years and/or when the tank is one-third full of sludge. According to Kalbermatten, *et al*., (1980) two-compartment septic tank is generally preferred to one with only a single compartment because the suspended solids concentration in its effluent is considerably lower (Kalbermatten, *et al*., 1980, Metcalf & Eddy, 1991). Two-thirds of the tank volume is normally reserved for the storage of accumulated sludge and scum, so that the size of the septic tank should be based on 3 day's retention at start-up; this ensures that there is at least one-day retention just prior to each desludging operation (Kalbermatten, *et al*., 1980). Above-mentioned recommendations for septic tanks are arbitrary and not based on any scientific insights. Moreover, the recommendations are just based on the height of the sludge bed in the compartments and not on the stability of the sludge, which is important when it is disposed off. Besides that as more stabilised sludge is found at the bottom of the sludge bed, sludge should be discharged from there instead of the top of the sludge bed.

On day 387 of the research period the sludge depths of the first and third compartments were measured using wet towelling method (Polprasert, *et al*., 1982). The sludge depth in the first compartment measured 145 cm and in the third compartment it only was 18 cm. The distance between the level of the sludge and the liquid level, in the first compartment was 0.48 m. The Canadian septic tank regulations permit a minimum distance of 0.46 m. Meaning that according to Canadian regulation the septic tank was almost full. However the septic tank was not emptied during the observation period. This implies that there was a possibility that during the next 113 days of monitoring the sludge accumulated even more in the first compartment and reduced the distance between sludge and liquid level to shorter than the recommended depth of 0.46 m. However, there was no noticeable impact on the effluent quality.

The scum layer in the first compartment was 14.5 cm thick and in the third compartment there was negligible scum layer thickness (about a mm or two).

Table 4.2: Reported septic tank removal efficiencies by other researchers for identified parameters and effluent COD concentration

Source (: researcher, country)	Removal efficiency (%)			Effluent quality	HRT*
	COD	BOD_5	SS	(mgCOD/L)	(days)
Polprasert, *et al.*, (average values from compilation of seven research papers)	47	27	70	323	-
Philippi, *et al.*, (66-craftsmen of a Training Centre, 2-compartment septic tank treating combined domestic sewage with effluents from agro-foods processing discharges at the centre)	33	32		695±196	-
Brandes, (3-residents, 1-compartment septic tank treating grey wastewater, Canada)	40	37	31	366 (mean) 119-870 (range for 18 samples)	3
Viraraghavan, (12-residents, 2-compartment septic tank. Quebec, Canada)	45	46	18	550	4.6
Der Graaf, (some 3-compartment septic tanks treating wastewater, The Netherlands)	28-56	18-54	48-98	-	3-10
Cotton, *et al.*, (statistical review of literature survey on existing data on septic tanks-countries:USA, Canada, Sri Lanka, India, Brazil, Zambia and Nigeria. Loughborough University of Technology, UK.)	-	68	80	-	<2.8
This research	29	16**	29***	334	5.5

*Theoretical hydraulic retention time (HRT)= effective volume of the septic tank divided by the influent sewage flow rate
**Measured as removal of soluble COD
***Measured as removal of suspended solids COD

Sludge stability

The course of the methane production from the sludge samples taken on day 387 from the respective compartments of the full-scale septic tank is presented in Figure 4.2.

From this figure it can be observed that over a period of 500 days 0.35, 0.45 and 0.47 g COD/g COD sludge from compartment 1, 2 and 3 was converted to methane, respectively. The methane production from sludge of compartment 1 and one of the samples from compartment 2 seemed not to be inhibited. However, the methane production from the samples from the third compartment and the other sample from the second compartment did looks to be inhibited. A hypothesis for this could be that the sludge samples were incubated without any inoculum sludge. In case the sludge sample itself does not contain sufficient methanogenic activity accumulation of volatile fatty acids can be expected, which then subsequently inhibits the methane production. Apparently the sludge from the third compartment has a low methanogenic activity and contains a high fraction of biodegradable material (Haandel & Lettinga, 1994)) which accumulated in the reactor fairly recently, probably due to the fact that the other compartments were full with sludge. Unfortunately no VFA analysis during the stability tests was done to support this hypothesis.The production of COD_{sol} from the sludge in the third compartment and the lack of methanogenic activity in this compartment might have also contributed to the poor efficiency for removal of COD_{sol} as compared to other septic tanks (Table 4.2)

4.3 Final discussion

Septic tanks suffer from an inherent fundamental design failure that adversely affects the removal efficiency, especially for the non-settleable influent organic fractions that are present in the dissolved and hydrolysed forms. The horizontal flow mode of the sewage in septic tanks is the predominant design feature responsible for the insufficient contact between the influent and the active biomass available in the settled sludge. The most important mechanism for the removal of organic material in biological sewage treatment systems is by bacterial metabolism. This mechanism of bacterial metabolism can be optimised by improving the contact between the influent and the active micro-organisms. This contact is poor in the conventional septic tank design. Most of the substrate from the horizontal flow mode in septic tanks reaches the active biomass by trickling through the sludge downwards from top. This is a very inefficient mechanism of enhancing contact between substrate and active micro-organisms. Moreover, the horizontal flow mode of the influent sewage in septic tanks also has an adverse impact on the hydraulic retention time (HRT). The HRT theoretically corresponds to the ratio between the effective volume of the septic tank and the daily influent wastewater flow rate. It decreases with time of the septic tank operation because of accumulation of solids from the influent, which form the sludge, which more and more occupies volume of the tank (Brandes, May 1978). Consequently not only there is poor contact between the influent non-settleable organic fractions and the active biomass, but also the time for these non-settleable fractions to trickle in the sludge accumulating in the compartments as the sewage flows through the septic tank (Fig. 4.1) becomes shorter along the septic tank operation. However, septic tanks are designed for HRTs of not less than one day (Table 4.1 & 4.2). This long HRT requirement leads to demands of high space and relatively high cost for their installation (Kalbermatten, *et, al*., 1980), which limits the application of the septic tanks in congested urban and peri-urban areas that need low cost on-site sanitation systems.

The main functions for the septic tank are (1) removal of suspended solids, (2) removal of a part of the soluble biodegradable components, (3) stabilisation and storage of sludge. Frequent removal of the sludge from the septic tank is not only a costly activity but it also only serves the first function of the septic tank. To the other two functions it is merely a disadvantage, because it denies the sludge from further stabilisation, particularly the upper part of the sludge. Moreover the removal of sludge denies growth of active biomass for removal of soluble components.

In Tanzania the practice has been that owners of septic tanks whether at community or individual residential dwelling level are the ones who seek a paid service of emptying the septic tank when it is 'full'. Normally this is done when the users identify specific problems with operation of the septic tank, e.g. as identified when the septic tank is 'overflowing' which refers to sludge wash-out, and is identified as the septic tank being 'full' of sludge and needs to be emptied. However septic tanks in Tanzania are supposed to be inspected by 'Health Assistants', who are the staff of Health Departments of municipalities. These Health Departments have curative, preventive and public health tasks. The 'Health Assistants' have the responsibility of motivating owners to empty their septic tanks in time prior to overflowing. However it is questionable whether these 'assistants' really sufficiently understand the operation of septic tanks. On the whole, the current practice of managing septic tanks and other on-site wastewater treatment systems is not effective especially in meeting public health and environmental goals. Consequently the existing management approach has to be improved in the context of overall management of septic tanks. The septic tank regulations existing in different countries call for a further technical institutional arrangement that can more properly carry out the requirements for efficient operation of the septic tanks.

4.4 Conclusion

Examination of the performance of the full-scale three-compartment septic tank treating domestic wastewater reveals that the sludge level especially in the first compartment was relatively high during the research period, which impaired the effluent quality. However, the inherent design feature of septic tank, viz. the horizontal flow mode, is responsible for the poor removal of non-settleable organic fractions viz., dissolved and hydrolysed components. The observed poor performance of septic tanks treating domestic wastewater in this research and from the literature as well show that septic tanks operated in the present practical mode are not suitable as onsite treatment option for wastewater at community level. The most essential features that need to be incorporated in the common septic tank in order to improve this most likely will lead to application of the Upflow Anaerobic Sludge Bed (UASB) reactor (Bogte *et al.* 1993), which is the simplest available technology that can best be employed in the treatment of wastewater onsite both at individual residential dwelling and at community level (Chapters, 5 & 6).

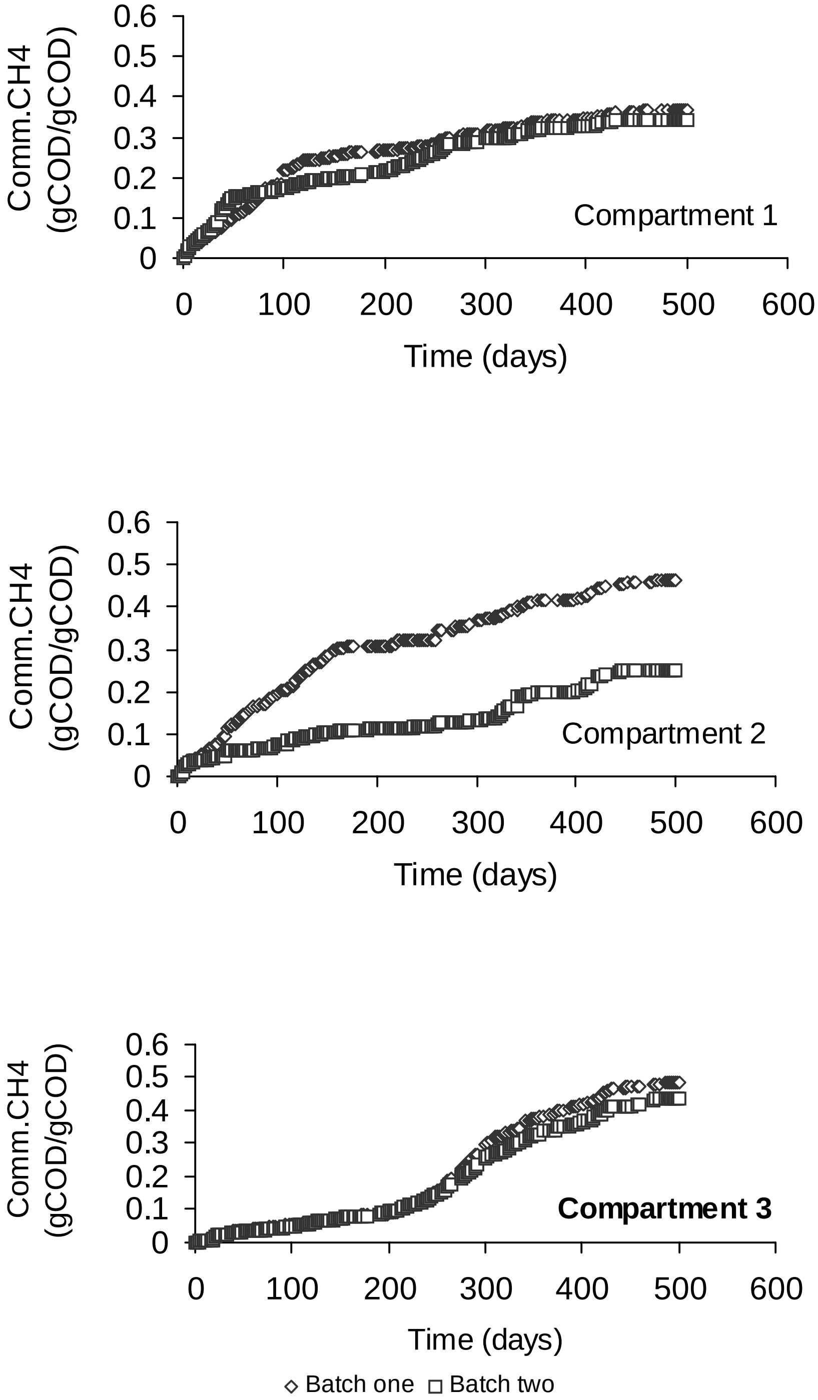


Figure 4.2: Course of the methane production from the sludge samples taken on day 387 from the respective compartments of the full-scale septic tank.

4.5 References

American Public Health Association (APHA); American water and works Association; Water Pollution and Control Federation, Standard methods for the Examination of Water and Wastewater (1992).

Brandes Marek (1978).Accumulation rate and characteristics of septic tank sludge and septage.*Journal of Water pollution control federation (WPCF).* May 1978 pp.936-943

Brandes Marek (1978).Accumulation rate and characteristics of septic tank sludge and septage.*Journal of Water pollution control federation (WPCF).* November 1978 pp.2547-2559

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

Cotton, A., Reed, R. and Okereke, C. (1996). Perception of septic tanks in the environment: An Overview. *Environment Protection Engineering*.Vol.22 No.3-4

van der Graaf, J.H.J.M., Fastenau, F. A. and van Bergen, A. H. M. (1989).Practical performance of various systems for small-scale waste water treatment during a two-year field test. *Wat. Sci. Tech*. **21.** pp**.** 1-12

Grin, P.C., Roersma, R. E. and Lettinga, G. (1983). Proceedings of the European Symposium Anaerobic Wastewater Treatment 23-25 November 1983, Noordwijkerhourt, The Netherlands.

Haandel A. C. van and Lettinga G. (1994). *Anaerobic sewage treatment-A practical guide for regions with a hot climate.* John Wiley & Sons Ltd. England.

Kabermatten, J.M., Julius D. S. and Gunnerson, C. G. (1980). Appropriate technology for water supply and sanitation. A sanitation field manual. *International Bank for Reconstruction and Development/World Bank/December 1980.*

Metcalf and Eddy (1991). *Wastewater Engineering:Treatment, Disposal, and Reuse.* 3rd edn. Revised by Tchobanoglous, G. and Burton, F.L., McGraw Hill, Inc.

Phillip, G. (1994). Recent developments in on-site domestic wastewater in Australia. *Proc. of the Seventh International Symposium on Individual and Small Community Sewage Systems, 11-13 December 1994*. ASE The Society for Engineering in Agricultural, Food, and Biological Systems

Philippi, L. S., da Costa, R.H.R. and Sezerino, P. H. (1999). Domestic effluent treatment through integrated system of septic tank and root zone. *Water Sci. Tech.* **40**(3), pp. 125-131

Polprasert C., Rajput, V.S., Donaldson D., Viraraghavan T. (1982). Septic tank and septic systems. *Environmental sanitation review*, No. 7&8, 1982.

Viraraghavan T. (1975). Septic tank efficiency. *Journal of Environmental Engineering Division, Proceedings of the American Society of Civil Engineers*, vol. 102, No. EE2.

Zeeman G.and Lettinga G. (2001). Objectives of DESAR-Decentralised Sanitation and Reuse.30. Abwassertechnisches Seminar & 13. DECHEMA-Fachgesprach Umweltschutz

5 PERFORMANCE OF A PILOT SINGLE-STEP COMMUNITY ON-SITE UASB REACTOR PRE-TREATING TROPICAL DOMESTIC WASTEWATER

Abstract – A pilot single-step community on-site upflow anaerobic sludge blanket (UASB) reactor (capacity: 1.5 m^3, height: 1.7 m) was operated over three and a half years treating part of the domestic wastewater from a University College of Lands and Architectural studies (UCLAS), Tanzania. The source of wastewater was from the cafeteria, toilets of some students' dormitories, and partly from university staff residential houses. The reactor was operated under gravity flow mode. The average influent organic pollution of wastewater amounted to 529 $mgCOD_{tot}$ /L with extreme values for the minimum and maximum, i.e. 46.4 and 4109 $mgCOD_{tot}$/L respectively. The temperature range of the wastewater was typical for tropical regions, viz. 25–34 ^oC. The applied hydraulic retention time (HRT) of the reactor varied i.e. the observed hydraulic retention times ranged from 1.7 to 40 hours, with an average value of 6 hours. Performance data obtained via regular monitoring of the treatment unit showed a declining removal efficiency over time with respect to COD_{tot}, which likely can be attributed to the increasing rate at which biogas was produced along with the growth of sludge bed and the presence of floating sludge. As a result the removal of dispersed sludge particles becomes poorer, which likely is reinforced by the 'less' optimal dimensions and design of the Gas-Solids-Separator (GSS) device. The average removal efficiency on COD_{tot} basis was 64 percent.

Key words – domestic wastewater, anaerobic treatment, sludge floatation, biogas generation, sludge development, sanitation.

Nomenclature-		
	UASB	upflow anaerobic sludge blanket
	COD_{tot}	chemical oxygen demand-total (mg/L)
	CODss	chemical oxygen demand-total suspended solids (mg/L)
	COD_{col}	chemical oxygen demand-colloidal (mg/L)
	COD_{sol}	chemical oxygen demand-soluble (mg/L)
	HRT	hydraulic retention time (h)
	SS	suspended solids (g/L)
	GSS	gas solids separator (three phase separator)

5.0 Introduction

Sanitation through employment of pit latrines and septic tanks in a community makes the presence of these onsite sanitation facilities too numerous to manage, apart from the question whether there are available good guidelines for operation and maintenance of these systems and/or these guidelines are followed. Consequently they pose increasing environmental pollution problems. The Upflow Anaerobic Sludge Blanket (UASB) reactor is a relatively simple wastewater treatment system, in which no moving parts are present (Lettinga *et al.,* 1980). Soon after its development, it was tested for treatment of domestic wastewater in tropical climates, Chapter 1. Although substantial experience on the design and operation of UASB plants for the treatment of domestic wastewater has been acquired during the last years (van Haandel and Lettinga 1994, Monroy, *et al*., 2000, Wiegant, 2001), much performance data have yet to develop for small-scale systems, such as community on-site domestic wastewater pre-treatment using UASB reactors.

In this research an investigation was carried out on the performance of a pilot-scale community-on-site single-step upflow anaerobic sludge blanket (UASB) that was employed to treat domestic wastewater in a typical Tanzanian environment. The characteristics of the environment were simulated to actual field conditions such that the flow of the wastewater to the plant was variable and in gravity flow mode at ambient temperature 25-34 ºC. There were no pumps involved in the operation of the plant. Performance of the reactor was studied in relation to sludge bed development, sludge floatation and gas production that took place during the research period. Understanding this relationship will in future improve the performance of the single-step UASB reactors that treat domestic wastewater in tropical climate environments in developing countries. They will serve a better replacement option for the existing low rate anaerobic sanitation facilities such as pit latrines and septic tanks.

5.1 Material and methods

5.1.1 Pilot single-step UASB reactor set-up

Shape and size

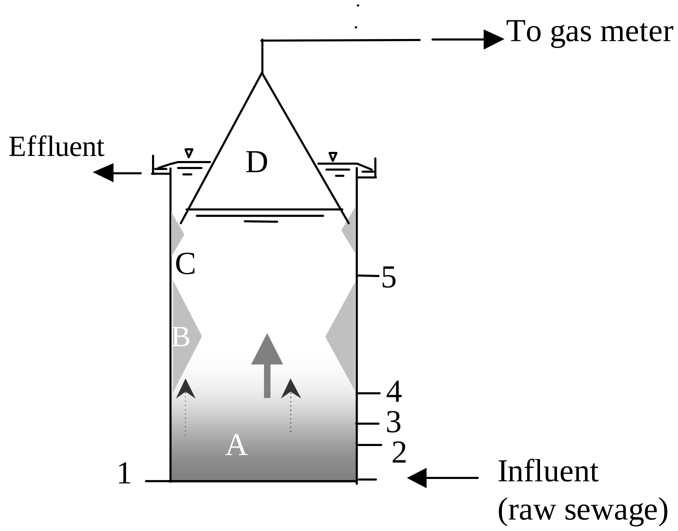


Figure 5.1: Schematic diagram of a pilot single-step UASB (capacity: 1.5m^3; height: 1.7m) A- Sludge bed; B-Lower baffle; C-Upper baffle; D- Gas collector; Liquid flow; Biogas production; 1- sludge sampling port at the base (0 cm); 2 - sampling port (20 cm from the base); 3 - sampling port (30 cm from base); 4 - sampling port (50cm from the base); 5 - sampling port (110 cm from the base).

Figure 5.1 shows the diagram of the single-step pilot plant UASB reactor that was constructed to assess the applicability of this technology for treating tropical domestic wastewater at community level. There were not any pumps needed in supplying influent to the UASB reactor, because the total treatment system was located below the ground level in order to facilitate natural gravity flow mode for the wastewater. As a consequence the reactor obviously had to handle quite extreme flow rate fluctuations, the more so, because in the rainy season also a considerable fraction of the rainwater arrived in the sewer, which frequently lead to excessive dilution of the wastewater.

The UASB reactor was constructed with locally available materials, i.e., cement-sand blocks of 13 cm thick, which commonly are available, at the local market. The baffles beneath the gas collector (together with the in-built settler constituting the Gas Liquid Solids (GLS) device (the three-phase separators)) were all constructed using a concrete mixture of cement, sand and aggregates. Water proofing cement was used for the inside and outside walls of the reactor. As shown in Figure 5.1, two levels baffles have been installed; the lower is located at 83 cm from the bottom (invert level) of reactor and the upper at a height of 138 cm from the invert level of the reactor. The lower baffle is slightly bigger than the upper one with an apex protruding 11.5 cm from the wall surface into the reactor, thus reducing the hydraulic flow through cross-section area of the reactor to 0.5 m^2 (physical measurements as constructed), from 1m^2. The apex of the upper baffle protrudes into the liquid phase of the reactor by 7 cm from the wall surface and this minimises the cross sectional area to about 0.74 m^2.

The net volume of the reactor is 1.5 m^3 and its height is 1.7 m. There is only one inlet for the influent, which was considered sufficient since the reactor cross-section area was only 1 m^2. However inside the reactor the influent pipe is split into two discharge points. A screen chamber preceded the pilot single-step UASB reactor.

Design criteria

The design of the pilot single-step UASB reactor has been based on hydraulic loading rather than organic loading. This is a common design principle in the case of a relatively low strength wastewater as found at the project area (Haandel and Lettinga, 1994). We have chosen for an average - six hours hydraulic retention time (HRT) based on operation at the daily average hydraulic loading flow rate.

As mentioned already above the actual HRT fluctuates significantly over time sometimes leading to extreme fluctuations in influent hydraulic load (Fig.5.2). However the HRT was regulated such that an average (standard deviation in brackets) of 6.16 (4.92) hours was achieved. Excess wastewater was diverted to other treatment units. No pumps were used in supplying the influent at constant rate. From literature it is known that an average retention time of six hours is sufficient in tropical and subtropical regions (T>18 °C) to achieve satisfactory treatment efficiency in one-compartment UASB reactors (Haandel and Lettinga, 1994).

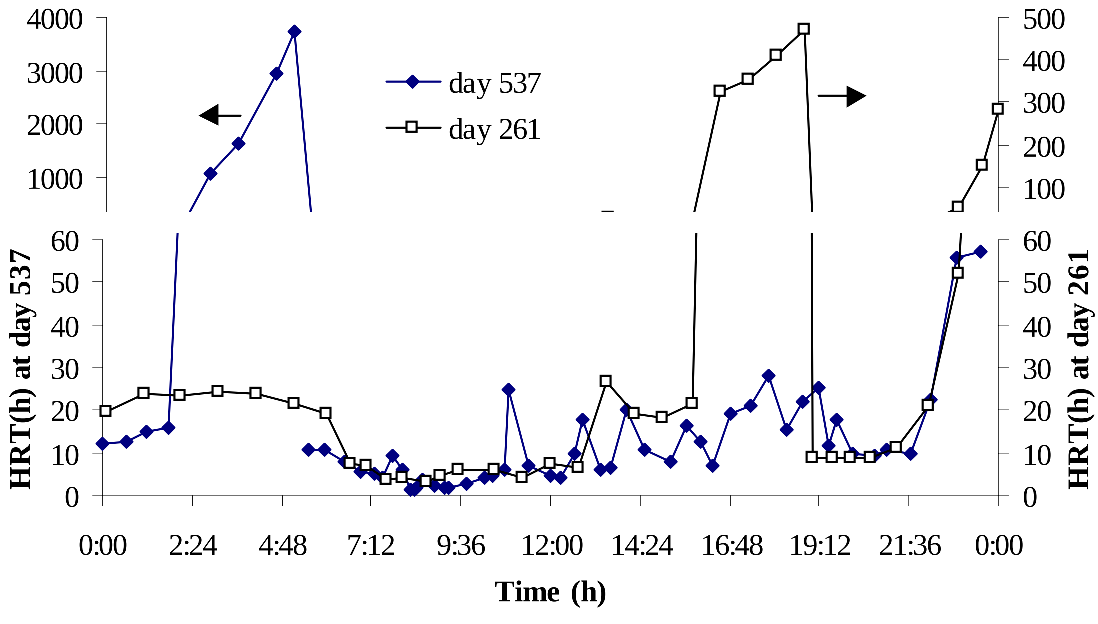


Figure 5.2: HRT variation over 24h on days 261 and 537 of operation of a single-step UASB reactor (capacity 1.5m3, height: 1.7m)

Wastewater characteristics

The composition and flow characteristics of the wastewater fed to the system were described in detail in Chapter 2 of this thesis. The average influent organic load of wastewater fed to the pilot single-step UASB reactor were (with standard deviations in brackets) in terms of COD_{tot}, COD_{SS}, COD_{col} and COD_{sol} as 529.4 (545.6), 264.4

(448.4), 130.5 (282.8) and 154.7 (112.5) mg/L respectively. The temperature of the wastewater with values ranging from 25–34 ºC was typical for tropical regions.

Sampling and analyses
Sampling of influent and effluent was made (almost) regularly twice to thrice in a week for a period of about three and a half years. One grab sample was taken usually at about 9 a.m. on every sampling day and analysed in duplicate as described in Chapter 2.

Analysis of solids
The TS, VS and ash content of the sludge bed were analysed by regular sampling of the sludge at different heights. Samples were analysed according to the *Dutch Standard Normalized Methods (1969).*

Specific methanogenic activity assay.
Glass bottles of 300ml capacity were used to accommodate 250ml of batch medium (Kato *et al*., 1993) with nutrients and trace elements. Besides that also 1000 mg COD per litre was added as sodium acetate and 2 gVSS/L of sludge. The bottles (in fact batch reactors) were incubated at ambient temperature of 26 to 30 ^{o}C for 24 h and shaken gently. After that the bottles were removed from the shaker and incubated at 25 ^{o}C. Here they were shaken manually once a day. The gas production was followed at incubation temperature by liquid displacement method (with 15% NaOH solution as displaced liquid).

Sludge stability assay.
Anaerobic stability tests were conducted to determine putrefaction potential of the volatile (organic matter) content of the sludge i.e. of the volatile suspended solids (VSS). The experimental set up used is described in Chapter 3.

Start up of the UASB-pilot plant reactor.
The reactor was started with 120 litres of digested sludge from the third compartment of the 56.3 m^3 (effective volume) septic tank described in Chapter 4. A detailed characterisation of the seed sludge was not made.

5.2 Results and discussion

5.2.1 COD removal efficiency.

The first 200 days of operation is regarded as start-up period as many technical problems had to be overcome. Therefore the performance of the system was evaluated starting from day 200. The calculated average removal efficiencies over a period of three and a half years (with standard deviation in brackets) for COD_{tot}, COD_{SS}, COD_{col} and COD_{sol} were respectively 64.2 (19.2), 56.5 (37.3), 42.3 (55.8) and 63.8 (34.9) %. These results are very similar to those reported by other researchers for well functioning UASB reactors in actual tropical field conditions at ambient temperature ≥ 20 ^{o}C. Recently Halalsheh (2002) reported results obtained in a 96 m^3 conventional one step UASB pilot plant treating a relatively high strength sewage (COD: 1531 mg/L) under subtropical conditions. The system provided an average COD_{tot} removal efficiency of 62% during summer and 51% during winter, when operated at 24 hours HRT at ambient temperature of 18-25 ^oC. As mentioned before (Monry *et al.*, 2000) the eight UASB reactors put in operation in Mexico at an average HRT of 6 hours and at an ambient temperature of 20 ^oC provided COD_{tot} removal efficiency in the range of 50- 95 %. The results in Fig. 5.3 show the assessed removal efficiencies for COD_{tot} and the separate distinguished COD fractions of our pilot plant. The extremely wide range of removal efficiencies can be attributed to both the highly variable pattern of hydraulic retention time and extreme variations in the influent organic load, as reflected by a very wide range of standard deviation values associated to respective influent COD fractions. This is a typical characteristic of hydraulic and COD generation pattern expected from a small community – where storm water is not or not sufficiently separated from the wastewater. A 24-hour monitoring of the influent hydraulic flow rate was conducted on day 261 and 537. The results, depicted in Figure 5.2, indicate that time instant of sampling, normally done at around 9 a.m. usually are preceded by a very high HRT i.e. exceeding 60 hours. Such high values of HRT could allow a very high degradation of the soluble COD-fraction in the reactor, but still not sufficiently long for a complete biodegradation of the already accumulated COD_{SS} and COD_{col} in the reactor. The results in Fig. 5.3 also show that COD_{tot} removal efficiency of the system declines over the experimental period. This decline can be attributed to wash-outs of COD fractions, i.e. especially the COD_{col} and COD_{ss} , that became more frequent with the passage of time especially day 500 of the reactor operation (Table 5.1). Many researchers have reported about the problem of wash-outs of COD fractions from UASB reactors treating a complex wastewater like domestic sewage. So for instance lately Halalsheh (2002) reported significant washouts in the big UASB reactor treating domestic wastewater of the city of Amman in Jordan. These wash-outs, although they mainly consist of poorly biodegradable (well stabilised matter), are responsible for the deterioration of the effluent quality.

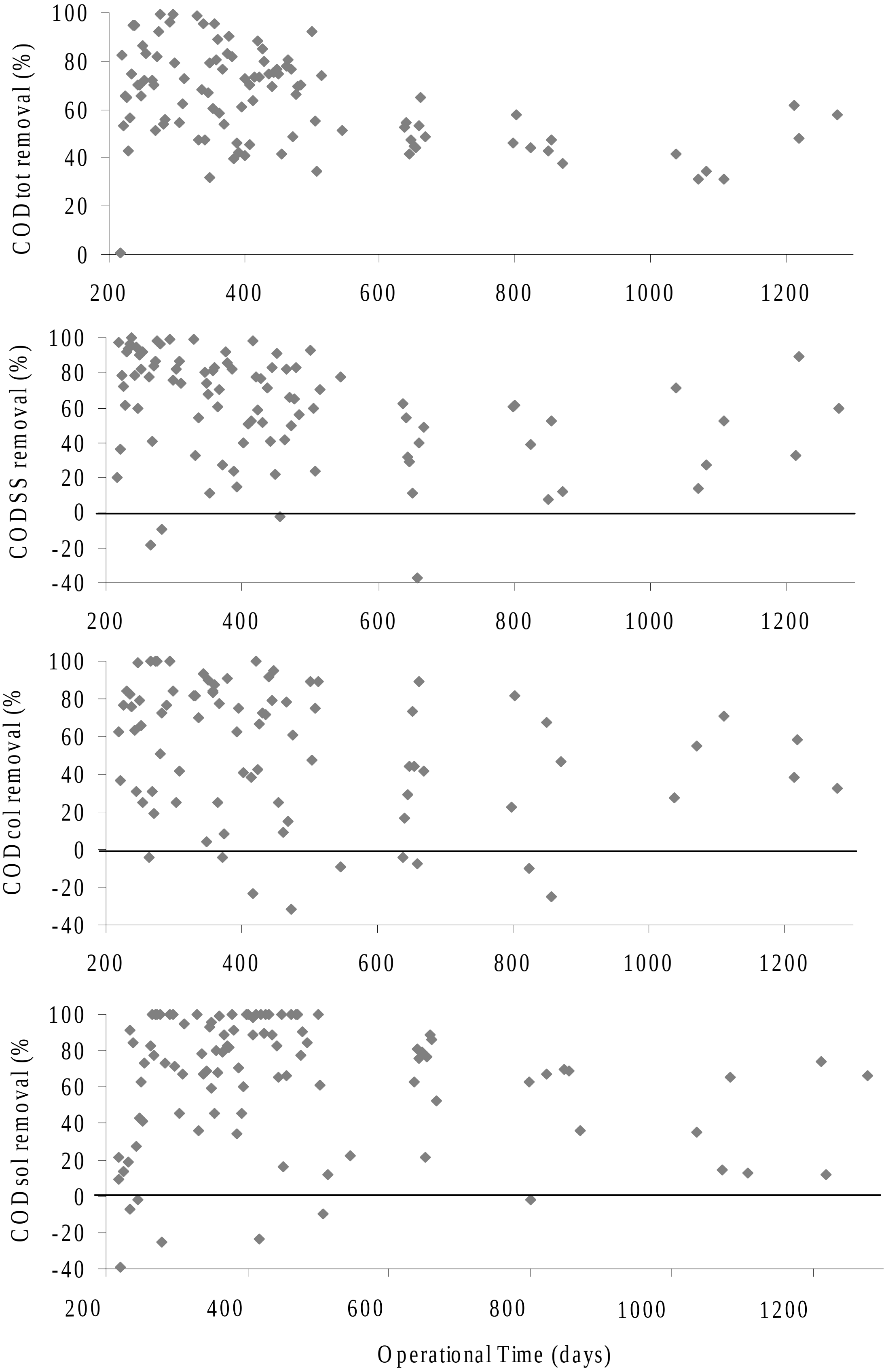


Figure 5.3: Percentage removal efficiencies for CODtotal and COD fractions overtime by a single-step UASB reactor (capacity 1.5m3, height: 1.7m)

Table 5.1: Average COD values for the influent and effluent of the UASB reactor grouped into two periods. Values between brackets are standard deviations.

	CODtot (mg/l)		CODss(mg/l)		CODcol (mg/l)		CODsol (mg/l)	
	Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
day 217-490 (80 samples)	545 (702)	130 (96)	308 (584)	47 (45)	128 (368)	50 (79)	137 (118)	50 (56)
Day 500-1276 (61 samples)	522 (202)	229 (123)	222 (120)	102 (69)	147 (91)	66 (57)	166 (96)	64 (50)

5.2.2 Sludge floatation phenomenon and its impact on sludge bed development

The results in Figure 5.5 show the sludge bed development in our single-step UASB reactor. Although the influent COD consisted of about 50% of suspended solids not only flocculant sludge developed in the reactor, but also granular sludge appeared (Figure 5.4).

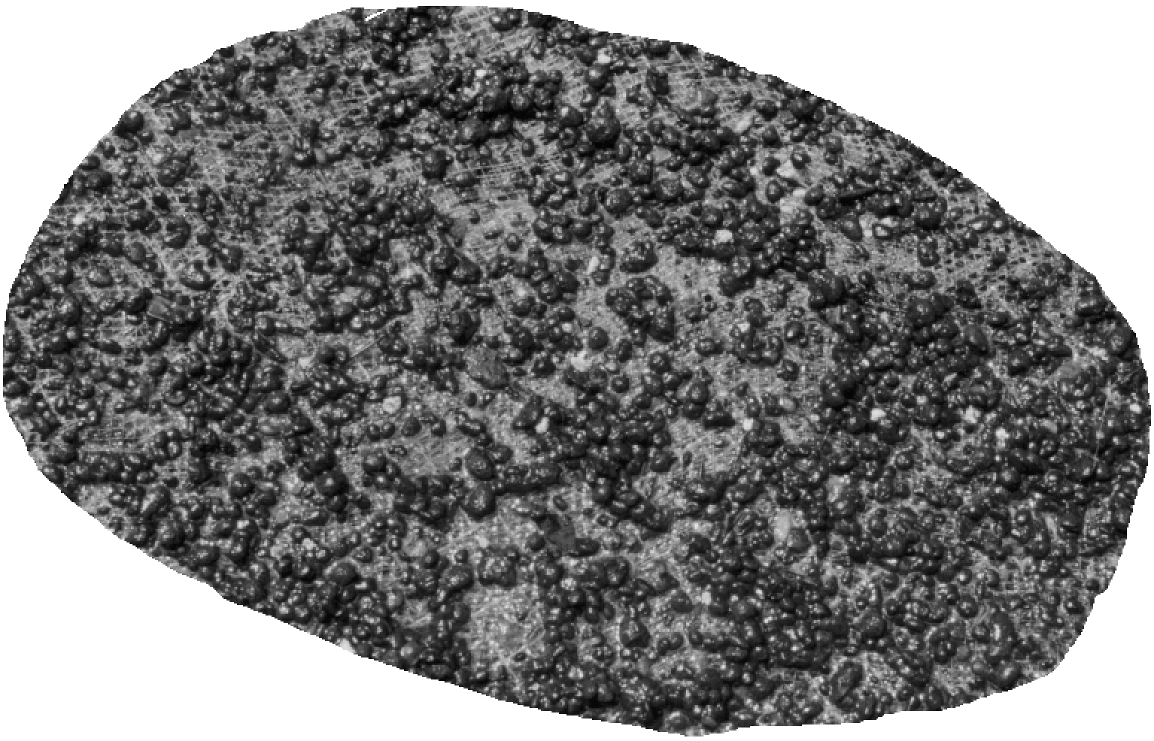


Figure 5.4: Sample of granular sludge developed in a single-step UASB reactor

Apparently the sludge bed growth did not proceed gradually but rather abruptly. When sludge bed growth happens the rate of filling up in a given depth section of the reactor height becomes rapid. The phenomenon of sludge floatation has been identified in our investigations through following up the TS concentration along the reactor height over the whole three and a half years experimental period. According to our observations the sludge bed growth particularly results from the settling of the occasionally collapsing of the sludge-floating (scum) layer.

Figure 5.7 presents the growth of the sludge bed in a more simplified manner. In this graph the days on which the sludge bed at the respective height along the reactor reaches a concentration of more than 30 g TS/l are depicted. It is assumed that once the sludge bed attains a steady concentration of 30 g TS/l or above the sludge bed is firmly established at that height. From the results in Fig. 5.7 it can be seen that from day 218 to day 311 no significant growth of sludge bed was taking place. During this period of little if any sludge bed growth in the reactor, in fact a floating sludge layer was developing. Most of the TSS removed by the system tends to buoy and as a result forms a sludge-floating layer inside and outside of the gas collector. During these periods of floating sludge the floating layer inside the gas collector was thick enough to completely block the GLS, which resulted in the fact that no gas could be collected. While inspecting the top water level of the reactor on day 304, the floating sludge layer was found to be 8 cm thick outside the gas collector. Upon inspecting the inside of the gas collector, there was a floating sludge layer measuring 18 cm thick. From day 311 onwards the sludge bed started to establish at a height of 20 cm (Figure 5.7). Moreover, from day 311 towards day 342 some gas was collected indicating that the scum layer in the GLS was less thick. It is therefore very likely that the sudden growth of the sludge bed towards 20 cm is the result from sludge settling from the floating layer into the sludge bed. However the sludge bed remained unstable during the next period from day 343 to day 455 and due to floating of the sludge bed the GLS was blocked again resulting in no collection of biogas. From day 462 the sludge bed showed a rapid growth to a height of 50 cm. After the sludge bed established at 50 cm also the gas collection started to resume (Figure 5.7). This again is an indication that sludge bed growth in the single stage UASB reactor was mainly due to settling of the scum layer.

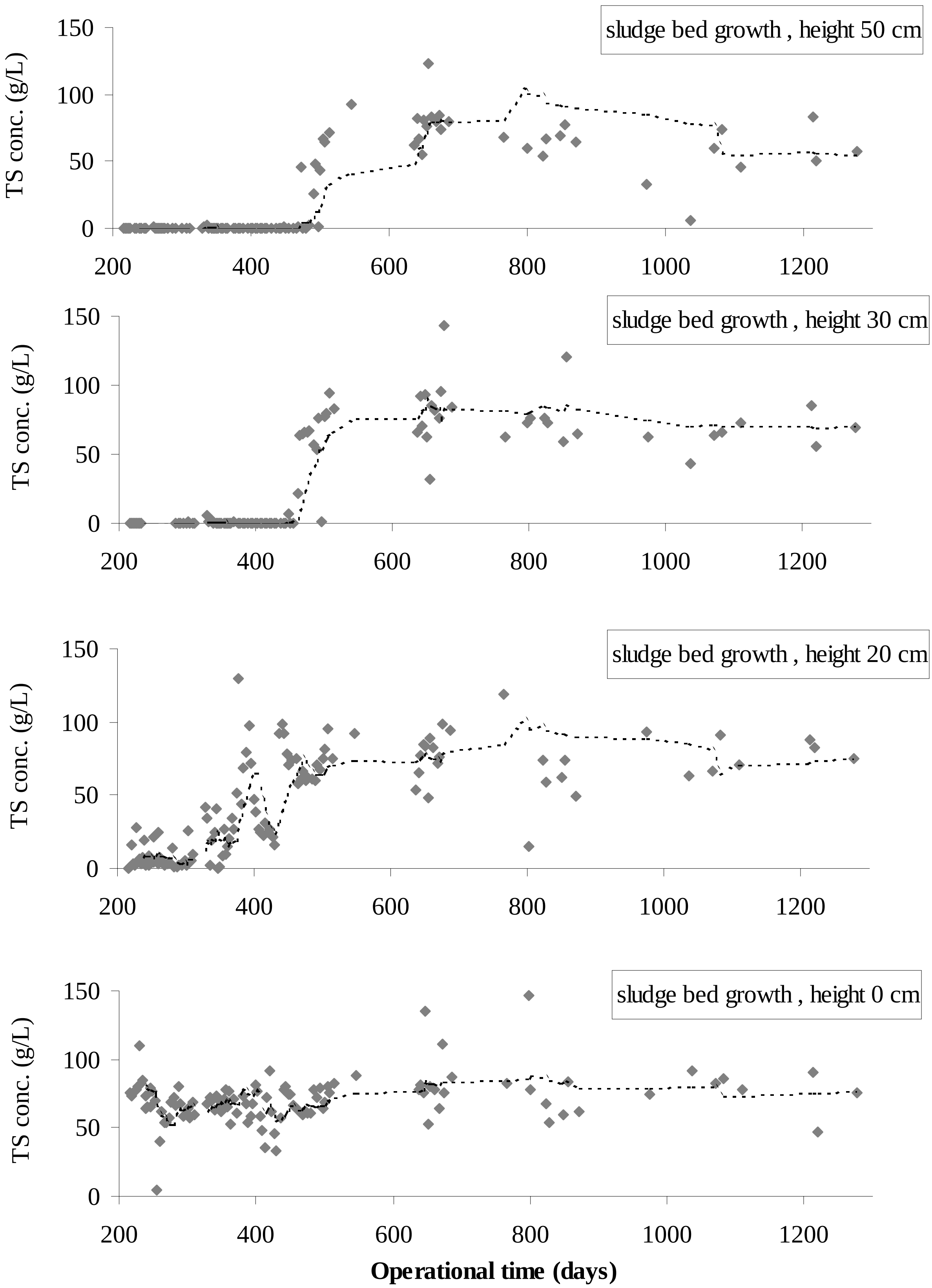


Figure 5.5: Sludge bed development-TS concentration along the height of the single-step UASB reactor (capacity 1.5m3, height: 1.7m) during the research period at height 0, 20, 30 and 50 cm from the invert level of the reactor. The dotted line represents the calculated moving average over a period of 10 samples.

Fig. 5.3 shows that days of sludge wash-outs clearly happened but they were not that alarming to have an impact on sludge bed development. This means that in the overall assessment the lack of the expected gradual increase of sludge bed predominantly can be attributed to the manifesting sludge floatation phenomenon rather than to any other reason that could be put forward. The sudden increase and decrease of the TSS concentration in the sludge bed reflects the sludge floatation phenomenon and it apparently has an impact on the biogas collection. Observations of the occurrence of floating sludge and associated problems under more or less similar field conditions like the wash-outs of COD fractions were also observed by Halalsheh (2002) while operating the 60 m^3 UASB pilot plant in Jordan and in the research (Haskoning, 1989) of the 64 m^3 UASB reactor in Cali Columbia.

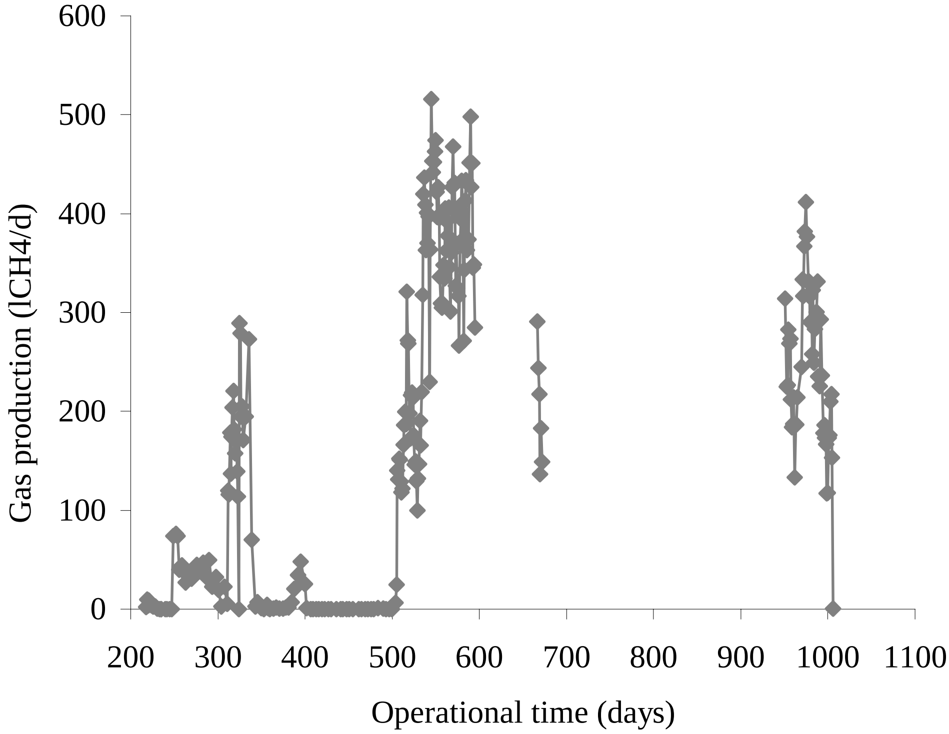


Figure 5.6: Amount of daily collected methane

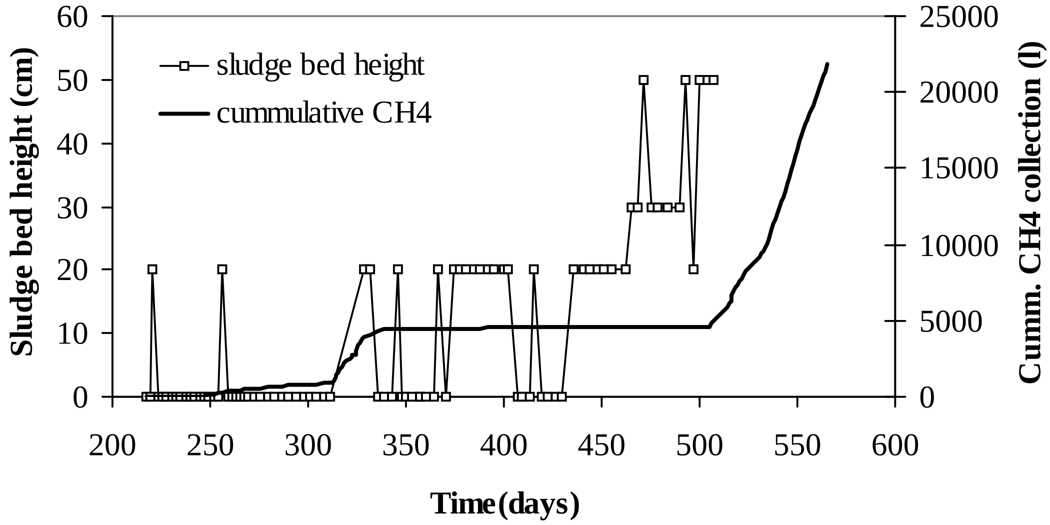


Figure 5.7: Correlation between the sludge bed growth and collected methane in the single step UASB system.

5.2.3 Sludge characteristics

Sludge methanogenic activity

On day 1215 of the operation of the reactor, the sludge specific methanogenic activity (SMA) was assessed along the reactor height. The results showed that the assessed specific methanogenic activity of sludge samples withdrawn from the bottom of the reactor and at the heights: 20, 30 and 50 cm from the invert level of the reactor was equal at 0.17±0.02 $gCOD(gVSS)^{-1}d^{-1}$. The SMA-assay was carried out at an average ambient temperature of 25 ºC. This results compare rather well to the findings of experiments with sludge samples withdrawn from the UASB pilot plant treating raw sewage at 30 ºC at the Wageningen Agricultural University, The Netherlands (Grin, et al, 1983), and the sludge developed in the UASB reactor in Cali Columbia (measured on a Volatile Fatty Acid substrate in a batch experiment) (Lettinga, *et al.,* 1987). The maximum specific methanogenic activity of sludge was found in the range 0.17-0.25 and 0.18-0.25 $gCOD(gVSS)^{-1}d^{-1}$ at Wageningen University and Cali Columbia respectively. These results imply that the methanogenic sludge along the reactor is of good quality and also that the floating sludge that has filled the reactor did not have negative impact on quality of sludge in the bed of the reactor.

Floating sludge physical characteristics

The results in Table 5.4 show the solids concentration of the floating sludge inside and outside the gas collector in samples taken on day 245. The TS and VS concentrations of the sample of floating sludge from inside the gas collector with values of 77.53 and 46.33 g/L respectively was higher than found in the sample drawn from outside the gas collector, viz. with values of 65.42 and 21.88 g/L respectively. The results show that VS-fraction mounted to about 60% of the TS for the floating sludge inside the gas collector whereas it was only 33% of TS for the floating sludge outside the gas collector. A possible reason for lower solids concentration on the outside of the gas collector likely might be that these solids are more susceptible to washouts from the reactor (with the effluent), which leads to a reduced TS concentration. The higher ash content found for the scum layer sludge outside the gas collector possibly can be explained by observation made by other researchers (Bolle, *et al*, 1986, Alphenaar 1994; den Elzen, *et al.*, 2000, Pietsch, *et al.,* 2002). Through macro-and microscopic in-situ observation of gas bubbles and sludge particles in a biogas tower reactor, Pietsch, *et al.,* 2002, observed that the suspended anaerobic sludge revealed an extensive fibrous structure (‘fur’) on the surface. The fibers had lengths of up to 0.5 mm. The observed micro fibers have a profound influence on the settling/floatation behaviour of the particles because they increase the effective particle volume, they may trap gas bubbles and they favour agglomeration (Pietsch, *et al.,* 2002). A pellet with an even surface (i.e. most likely non biogas producing biomass) moves totally different than a pellet with the mentioned fibers. From the literature methanogenic sludge is reported to have fibrous material (Pietsch, *et al.*, 2002). From optical observation high portions of sludge flocs filled with biogas bubbles were identified. These bubbles were trapped in the flocs compared to ‘free’ bubbles (Pietsch, *et al.,* 2002). Unless entrapped within sludge agglomeration, floating matter consisting of non-biogas producing biomass attaches poorer to biogas bubbles and hence likely less of such matter will be found in a floating sludge layer that is inside the gas collector. More such floating material will be swayed by the effluent and get trapped by the floating sludge layer outside the gas collector. The higher VS-fraction in the TSS found for the scum layer sludge inside the gas collector emphasises the significance of several observations we made that ‘well’ biogas producing biomass attaches to biogas bubbles (or better entraps these bubbles within the aggregates Pietsch, *et al.,* 2002). This observation underscores the fact that biogas producing sludge is faced with the continuous production of gas bubbles and consequently with the ‘problem’ to rid of them sufficiently fast. As a consequence in this situation it easily will float to scum layer present inside of the gas collector. Sludge floatation may seem to be more of a result of physicochemical effects (Pietsch, *et al.,* 2002).

Table 5.4: Observed solids concentration of floating sludge layer in a single-step UASB reactor (capacity: 1.5m^3, height: 1.7m) on Day 245 of reactor operation

Floating sludge from inside gas collector			Floating sludge from outside the gas collector		
TS (g/L)	VS (g/L)	Ash (g/L)	TS (g/L)	VS (g/L)	Ash (g/L)
77.53	46.33	31.2	65.42	21.88	43.54

Sludge stability

Table 5.5 shows results of the stability, test conducted on sludge samples withdrawn over the height of the reactor on day 1190. One objective of the test was to determine the influence of the occurring settling of scum layer sludge in the sludge bed on the stability of the sludge at that location. The test was conducted at average ambient temperature of 25 ^oC. The results of the test, shown in the profiles Figure 5.8, present the cumulative production measured on the 89th day of the test run. Although the results of the test conducted with the sludge from the top of the sludge bed, viz. located 110 cm from the invert level of the reactor (the profile not shown in Fig.5.8), were not reliable, we obtained indications that it was by far less stabilised compared to the stability of the sludge samples taken from lower parts of the sludge bed. The most stable sludge is located at the bottom of the reactor with a stability of 46 mlCH4/gVSS. Apart from the likely rather unstable sludge found at a height of 110 cm, the rest of the sludge bed is very well stabilised. The likely poorer stabilised sludge at a height of 110 cm was found to settle poorly, in fact it was present in rather finely dispersed state and consequently is highly susceptible to washouts. Probably major amount of sludge stabilisation (biodegradation) of this sludge proceeds once it is present in the scum layer. Fig. 5.8 shows the course of the test for stability.

Table 5.5: Stability test of sludge along a height of the reactor on day 1190 of the reactor operation

Sludge characteristic	Position of sludge sample from bottom of the reactor			
	Bottom sludge	Sludge at 20cm	Sludge at 30cm	Sludge at 50cm
Stability, mlCH$_4$/gSV, Day 89 of the experiment run	46.0	59.4	53.0	74.5

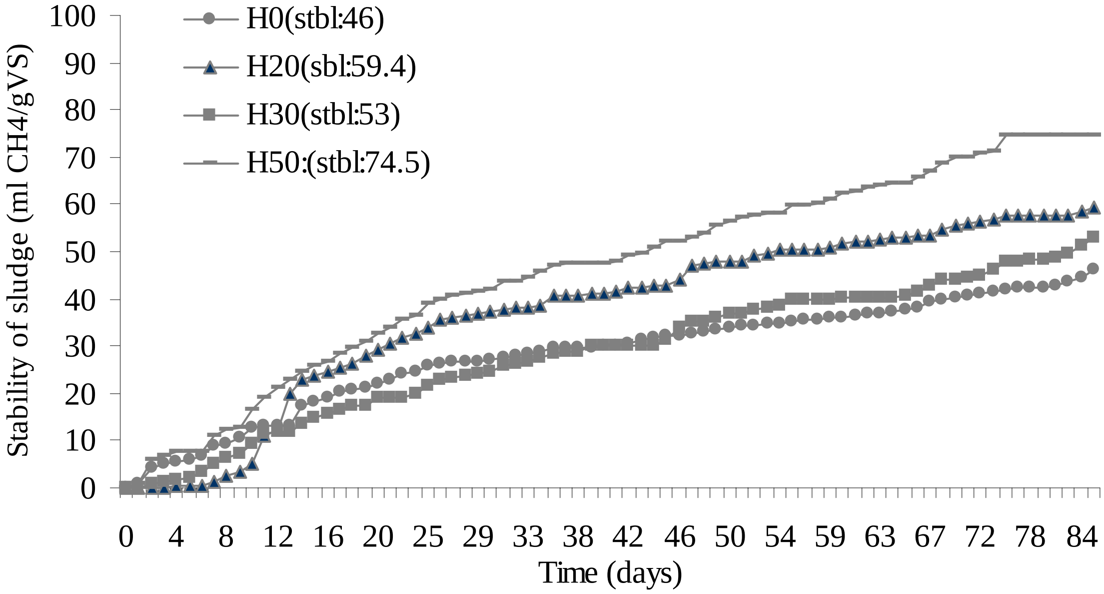


Figure 5.8: Sludge stability along the height of the single step UASB reactor on day 1190 of the reactor operation.

5.2.4 Observed relationship of gas production, sludge floatation, sludge washout and effluent quality

From the results presented it can be seen that from day 500 onwards the effluent quality declines. This was mainly due to a lower suspended solids removal (Table 5.1). The most likely reason for this is the continuous decrease of the distance between the top level of the growing sludge bed and the bottom of the scum layer, particularly after it starts crumbling at day 500. It can be expected that the deterioration of the effluent quality will become more serious when this distance becomes shorter, because, then the time available for the scum layer sludge particles to flocculate becomes shorter *(Bolle, et al., 1986).* And as a consequence the chance of wash-out of these settling particles increases. The time available to flocculate to sufficiently heavy aggregates simply becomes to short, and with that for a rapid sedimentation, consequently for accumulation on (entrapment in) the sludge bed. And even in case they would manage to settle, due to their relatively low density and likely high fragility they could still become easily redispersed as fine particles by the action of upward drift of the hydraulic load and the evolving biogas *(Pietsch, et al., 2002).* Both the upward drift of the hydraulic load and the evolving biogas with relatively high floatation force (especially when still present in extremely small gas bubbles) are the two forces counteracting the downward motion of the fallout floating sludge. The balance of all these – partially opposing - forces, together with factors like the size, shape and density of the primary particles, the strength, size and shape of flocculated aggregates, greatly determine the wash-out or the retention of dispersed sludge present above the sludge bed, i.e. present in the “sludge blanket”.
An aspect of crucial importance in the whole process certainly also comprises the rather extreme fluctuations in organic and hydraulic loading to which the system is exposed during the day, and also season. In this respect special attention needs to be afforded to the complexity of the scum layer phenomenon itself, i.e. the reasons of its formation, its dynamic behaviour and its disappearance. It should be emphasized here

that for a well functioning UASB-reactor there exist significant differences in the mixing-up intensity between the scum layer present in the gas collector and that present at the liquid-air interface of the settler. As a result of the gas production the scum layer in the gas collector is mixed quite well, while this is not the case for the scum layer in the settler, unless a significant amount of gas would not be entrapped in the gas collector. Due to the relatively good mixing conditions in the settler, the scum layer sludge a) remains completely wet and b) will become 'occasionally' released from entrapped, e.g. evolving, gas bubbles! The situation for the scum layer sludge in the settler is quite different; it is poorly mixed, while it is exposed to a drying out process because of exposure to sun and air. It will be evident that quite dramatic effects, like the falling down of the 'wetted' scum layer, crumbling of the sludge, can be expected from a sudden drastic change in the conditions, e.g. heavy rain or a sudden substantial escape of biogas via the aperture into the settler compartment. Regarding the high daily fluctuations in loads and the occurrence of heavy rains during the rainy season, such kinds of effects certainly would not be exceptional. In this connection it should be mentioned here that big amounts of biogas accumulated in the sludge bed during the daily period of low load, could be set free in a few seconds once the system suddenly is exposed to daily peak loads. It is quite doubtful if the release of relatively big quantities of gas can be completely collected in the gas collector and sufficiently fast released from the reactor via the gas collector. This highly depends on the design and construction of the system. Particularly the design, the shape, the construction and the dimensions of the GSS-device therefore constitute a factor of big importance with respect the performance of an UASB-reactor. Depending on the extent of scum layer formation, the degradation processes that can occur here, the amount of mixing and or drying out happening in the layer, the storage capacity within the gas collector, the system will suffer from a more or less serious wash-out of TSS. In case the storage capacity is small the residence time of the scum layer sludge in the collector will become short, and as a consequence, a significant part of the incoming buoying – still very poorly stabilised - matter reaching the bottom of the scum layer, likely rapidly will be forced into the settler compartment. And there it gives rise to formation of the external scum layer. Due to a too short residence time, the scum layer sludge within the gas collector also may not get sufficient time to stabilise. A too small storage capacity, relative to the extent of the formation rate and/or the slowness of the prevailing stabilisation rate within the scum layer, therefore also may lead to gradual worsening of the sludge retention of the system and consequently to a deterioration of the overall treatment efficiency. Regarding the size and the construction of the GSS-device, i.e. particularly the gas collector, the pilot plant used in our investigations presumably suffered from a small scum layer storage capacity, while also the required efficient collection and easy release of biogas may have been rather poor! The results in Figures 5.5 and 5.6 indeed suggest this. Apart from the results in Figure 5.6 we indeed observed that occasionally gas escaped via the settler.

5.3 Final discussion

Feasibility and potentials of a single-step UASB pre-treatment unit based on a pilot single-step UASB reactor (capacity: 1.5 m^3; height: 1.7 m) in treating tropical domestic wastewater

The performance of a pilot single-step UASB reactor in treating tropical domestic wastewater shows that it is feasible to attain even higher removal efficiencies of influent organic loads provided the problem associated with the settling of floating sludge associated washouts of COD fractions is addressed. The problem identified is the decline of the overall treatment efficiency of the treatment unit over time. The decline of treatment efficiency starts when the sludge-floating layer begins to collapse and occasional falling continues thereafter. The sludge bed initially, in this particular case about the first 200 days of operation, does not grow and the TS concentration is steadily maintained in the range of 75-80 gTS/L. The sludge bed concentration at the bottom of the reactor of 75-80 gTS/L appears to be maintained throughout the research period and it appears that TS concentration in excess of this range is not stable and therefore this range is maintained by letting some sludge to go afloat. Prior to the collapse of the floating sludge layer the removal efficiencies of COD fractions were high.

The collapse of the floating sludge layer signals the beginning of the new phase of the treatment performance. The sludge bed grows rapidly as the floating sludge layer settles on it. The effluent quality of the pilot plant deteriorates with the passage of time. Apparently the deterioration in effluent quality is brought by the washouts of COD fractions from the reactor that is accelerated by the settling of the floating sludge layer. The washouts of COD fractions are predominantly caused by the exertion of the upward movements of the hydraulic load and evolving biogas on the settling floating sludge fallouts. To some degree the shortening of the depth between the ever-growing sludge layer and the bottom of the collapsing floating sludge layer also accelerates the washouts. This depth is essential for fallouts flocculation as they settle down to the sludge bed. The magnitude and frequency of forming and deforming the floating sludge layer becomes higher with the passage of time and so is the worsening of the effluent quality appears to take place particularly with regard to COD washouts in form of SS, and colloids. However the quality of the settled floating sludge layer is good. It has a comparable high specific methanogenic activity; though less stable and can effectively degrade the COD of the influent wastewater to about 64% (Chapter 2).

5.4 Conclusions

The satisfactory performance of a single-step UASB treatment unit on long term basis depends on the improvement in its design that will take care of the effects of the settling floating sludge layer which apparently comes into effect a little later after the commencement of treatment operation. Performance data obtained via regular monitoring of the treatment unit showed a declining removal efficiency over time with respect to COD_{tot}, which likely can be attributed to the increasing rate at which biogas was produced along with the growth of sludge bed and the presence of floating sludge. As a result the removal of dispersed sludge particles becomes poorer, which likely is reinforced by the ‘less’ optimal dimensions and design of the Gas-Solids-

Separator (GSS) device (Section 5.2.4). The average removal efficiency on COD_{tot} basis was 64 percent.

Meanwhile a two-step UASB pre-treatment unit might be a solution since it will delay the effects of the settling floating sludge layer. It implies that the second-step will accommodate the washouts from the first-step as an impact of the settling of the floating sludge layer-taking place in that first-step. However finally both steps will experience the effects of settling floating sludge layers that need to be addressed.

5.5 References

Alphenaar P. Arne (1994). Anaerobic granular sludge:characterization, and factors affecting its functioning. *PhD. Thesis*. *Wageningen Agricultural U*niversity

Bolle, W.L., van Breugel, J., van Eybergen, G. C., Kossen N. W. F. and van Gils, W. (1986). An Integral Dynamic Model for the UASB Reactor. *Biotechnnol. Bioeng*. vol.**28**. pp. 1621-1636

den Elzen J.M. Jeffrey and Jeroen A. Koppes (2000). Controlling scum formation during anaerobic sewage treatment under Middle East conditions. *MSc Thesis, Number:00-21, Department of Environmental Technology, Wageningen Agricultural U*niversity

Grin, P.C., Roersma, R. E. and Lettinga, G. (1983). Proceedings of the European Symposium Anaerobic Wastewater Treatment 23-25 November 1983, Noordwijkerhourt, The Netherlands.

Haandel A. C. van and Lettinga G. (1994). Anaerobic sewage treatment-A practical guide for regions with a hot climate. John Wiley & Sons Ltd. England.

Halalsheh, Maha, M. (2002).Anaerobic Pre-treatment of strong sewage. A proper solution for Jordan. *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen.

Haskoning, (1989). Royal Dutch Consulting Engineers and Architects. Anaerobic treatment of domestic wastewater under tropical conditions-final report.

Kato, M.T., Field, J. A., Lettinga, G. (1993). High tolerance of methanogens in granular sludge to oxygen. *Biotech. and Bioeng*. vol. **42**. pp.1360-1366

Lettinga G., de Man Ad, Grin, P., and Hulshoff, L. P. (1987). Anaerobic wastewater treatment as an appropriate technology for developing countries. *Trib. Cebedeau*, N$^{\underline{o}}$ 519, 40, pp. 21-32. *Cebedoc Editeur 1987, Liège, Belgique*

Lettinga G., van Velsen, A.F.M., Hobman, S.W., de Zeeuw, W.J. and Klapwijk, A. (1980). Use of the upflow sludge blanket (UASB) concept for biological wastewater treatment, especially anaerobic treatment. *Biotech. and Bioeng*. vol. **22**. pp.699-734

Monroy O., Fatma G., Meraz M., Montoya L., and Macarie H. (2000). Anaerobic digestion for wastewater treatment in Mexico: State of the technology. *Water Res*. **34**(6), 743-750.

Pietsch, T., Mehrwald, R., Grajetzki, R., Sens J., Pakendorf, T., Ulrich, R., Kumpart, J., Matz, G. and Märkl, H. (2002).Macro-and microscopic in-situ observation of gas bubbles and sludge particles in a biogas tower reactor. *Water research*, vol.**36**, Issue 11, June 2002, pp. 2836-2842

Wiegant, W.M. (2001).Experiencies and potential of anaerobic wastewater treatment in tropical regions. *Water Sci. Technol*. **44**(8), 107-113

6 PERFORMANCE OF AN EFFECTIVE COMPACT COMMUNITY ONSITE TWO-STEP UASB PRE-TREATMENT UNIT FOR TROPICAL WASTEWATER

Abstract -This chapter deals with a research on the two-step UASB reactor configuration as follow up of the findings made in the previous research on the performance of a pilot single-step UASB reactor. The pilot two-step UASB reactor configuration was operated in parallel from day 665 of the research period to the then on going single-step UASB-reactor. The investigations were conducted under the same environmental conditions as the research on the single-step UASB reactor covered in Chapter 5. As demonstrated in Chapter 5 the wash-out COD fractions from the single-step UASB reactor was an important outcome of the dynamics of the scum layer, the fluctuating biogas evolution with high peak values as well as the design and construction of the pilot plant. So with the addition of a second-step UASB reactor in the present research we intended to contain the washouts and polish the effluent of the first step. The two-step UASB pre-treatment unit in this research therefore refers to two UASB reactors connected in series, viz. a first 2m high 1.8m^3 UASB reactor put in front of a second 1m high 0.852 m^3 UASB reactor. The second-step UASB reactor is a recipient of effluent including washouts from the first-step UASB reactor. Like in the investigations with the single step UASB-reactor the two step reactor configuration was operated under gravity flow mode. The treatment process was started without any seeding and was operated at ambient tropical temperature of 25 - 34 ^{o}C. The average organic loads of the wastewater imposed to the system with respect to COD fractions CODtot, CODss, CODcol and CODdis were 537.2 (165.3), 189.9 (109.5), 127.4 (75) and 223.4 (108.8) respectively. The investigations were conducted over a period of 630 days. The overall removal efficiency obtained on the basis of the distinguished COD fractions was far better than for the individual reactors, i.e efficiencies (with standard deviation in brackets) for CODtot, CODss, CODcol and CODdis, were respectively 68.7 (16.7), 51.2 (41.8), 62.1(38.2), and 71.8 (30.5) %. The imposed overall HRT was 7.4 (1.6) (i.e. 5 + 2.4) hours. The maximum specific methanogenic activity of sludge developing in the 1st-step reactor ranged from 0.061 – 0.135 gCH4-CODper gVSS per day and for the 2nd-step reactor it was in the range 0.104 - 0.272 gCH4-CODper gVSS per day. The advantages of the two-step UASB reactor configuration include 1) the distinct higher overall removal efficiency of the anaerobic pre-treatment system 2) the higher sludge age 3) the higher reliability of the anaerobic pre-treatment process 4) the two reactors can separately be operated in case of technical problems. The reactor system can be designed with a by-pass option such that in moments of emergency each separate reactor can take extra load of the other without adverse effects and without shutting down the plant. This would make the Two-Step UASB reactor pre-treatment configuration more robust and stable, consequently more reliable for community on site wastewater pre treatment needs.

Key words- Anaerobic; domestic sewage; UASB; Wastewater treatment; anaerobic phase digestion, Two-step

Nomenclature –	UASB	upflow anaerobic sludge blanket
	COD_{tot}	Chemical oxygen demand-total (mg/L)
	CODts	Chemical oxygen demand-total solids (mg/L)
	COD_{col}	Chemical oxygen demand-colloidal (mg/L)
	COD_{sol}	Chemical oxygen demand-soluble (mg/L)
	HRT	Hydraulic retention time (h)
	SS	Suspended solids (g/L)
	GLS	gas liquid separator (three phase separator)
	DESAR	Decentralised sanitation and reuse

6.0 Introduction

Several researchers have been working on two-step wastewater treatment systems with different objectives, viz. based on the specific difficulties experienced with the single step treatment process. Unlike this research, whereby the objective of involving a second-step UASB reactor is essentially to polish further the effluent from the first-step UASB reactor, research undertakings of the two-step wastewater treatment system in the past focused on achieving a certain extent of phase separation, viz. the first step focusing on hydrolysis-acidogenesis (but also allowing methanogenesis) followed by a mainly acetotrophic-methanogenic second treatment step (Zeeman, *et al*., 1999; Breure, 1993; Lettinga, *et al*., 1993). Usually the hydrolysis step is the overall reaction rate-limiting step in the case of domestic sewage treatment, especially this becomes notable in low temperature climates (5 - 20 ºC) (Lettinga, *et al*., 1993). Under such circumstances the overall treatment process of the system might deteriorate, due to a rapid accumulation of non-stabilised SS in the reactor. This would signify demands for reactors with higher volume capacity to cater for the excessive SS accumulation problem. As a response to this undesirable phenomenon several researches have been put in place that focus on finding proper solution for the treatment of domestic sewage under anaerobic stress conditions in the midst of the identified drawbacks. An example of a research on two-step wastewater treatment system is a system configuration that involves an 'Hydrolysis Upflow Sludge Blanket' (HUSB), as a hydrolysis reactor (as first step) and 'Expanded Granular Sludge Bed' (EGSB) reactor as second methanogenic step: (Wang, 1994). Another example of the two-phase process is a configuration involving two identical flocculent sludge UASB reactors operated intermittently for the removal and digestion of SS as a first-phase. The second-phase takes place in another granulars sludge bed UASB reactor (Sayed, *et al*., 1995). All these systems have been investigated under low temperature climates (5 - 20 ºC). Under warm climates (> 20 ºC) quite satisfactory experimental results have been obtained using a simple one-step UASB reactor system, viz. in terms of treatment efficiencies and the extent of sludge stabilisation as well. The other type of the two-step proposition comprises a UASB-Septic tank system, including also low temperature conditions (Lettinga, *et al*., 1993). The UASB-Septic tank system treatment process performs differently depending on whether it is summer or winter. The first reactor of a two step system during winter mainly serves for retaining solids, because then just a limited amount of hydrolysis, acidification and methanogenesis will occur. In the second reactor mainly methanogenesis will occur, even at low temperature conditions. In summer time hydrolysis and acidification will proceed, viz.

of both fresh and accumulated solids as well in the first reactor together with methanogenesis, while the second reactor acts as a polishing step for removing and converting the remaining volatile fatty acids (VFA) and suspended COD washed from the first reactor as a result of overloading (at temporay) (Zeeman, *et al*., 1999, Lettinga, *et al*., 1993). Table 6.1 shows a summary of application of reported UASB treatment systems so far investigated at laboratory scale or in full-scale operation (Elmitwalli, 2000). Mahmoud (2002) investigated the feasibility of anaerobic sewage treatment using a one-stage UASB at 15 ºC in combination with a digester system operated at 35 ºC. This combined UASB-digester gave substantially better removal efficiencies and conversion than the conventional one stage UASB reactor. The achieved removal efficiencies for COD_{tot}, COD_{ss}, COD_{col} and COD_{sol} in the UASB-Digester are “66, 87, 44 and 30” and for the conventional one stage UASB “44, 73, 3 and 5” % respectively. These drawbacks associated with low temperatures were not encountered in our research in the operation of the pilot single-step UASB reactor as already explained in Chapter 5. Instead the main problem we encountered in that research was the gradually increasing wash-outs of COD fractions, which lead to a deterioration of the effluent quality over time.

The research described in this chapter is a follow up of the findings made on our previous research dealing with the performance of a pilot single-step UASB reactor. The wash–out of COD fractions from the single-step UASB reactor was an outcome of the dynamics of floating sludge (scum layer), the highly fluctuating daily loading conditions and the size and construction of the reactor, particularly of the GSS-device. The research on single-step UASB reactor has demonstrated that the SS washouts emanating from the settling of floating sludge were settleable and likely also well biodegradable. So the objective of this research is to find out if with the addition of the second-step UASB reactor fed by the effluent from the first-step UASB reactor would be capable to contain the wash-outs and polish them further. This would lead to a distinct raise of the overall level of treatment efficiency compared to the single step system. Thus the pilot two-step UASB reactor configuration was operated in parallel from day 665 of the research period to the then on going operation of the single-step UASB-reactor. The two researches were conducted under the same environmental conditions. The two-step UASB pre-treatment unit in this research therefore refers to two UASB reactors connected in series. The first-step UASB reactor has a volume of 1.80 m^3, and its height is 2 m. The volume of the second UASB reactor is 0.852 m^3 and its height is 1m. The hydraulic load of the second reactor therefore is substantially higher than the first reactor, which obviously is a far from optimal situation for a satisfactory removal of SS washed out from the first reactor. This should be taken in mind. The overall aim of the investigations in the pilot scale two-step UASB reactor system is to assess its potential in pre-treating domestic wastewater on site at community level in a typical Tanzanian environment.

6.1 Materials and methods

Experimental set-up

Figure 6.1a shows the schematic diagram whereas Figure 6.1b an actual picture of the pilot two-step two-stage one-phase treatment process system configuration investigated. A 2 m high UASB reactor is connected to a 1m high UASB reactor. The volumes of the 2m and 1m high reactors are about 1.8 m^3 and 852 litres respectively. Structurally the UASB reactor was constructed with locally available materials. These are cement-sand blocks of 13 cm thick commonly found in the local market. The three-phase separators were constructed using concrete: a mixture of cement-sand-aggregates and 6 mm reinforcing bars. Water proofing cement was finally applied in the inside and outside of the reactor.

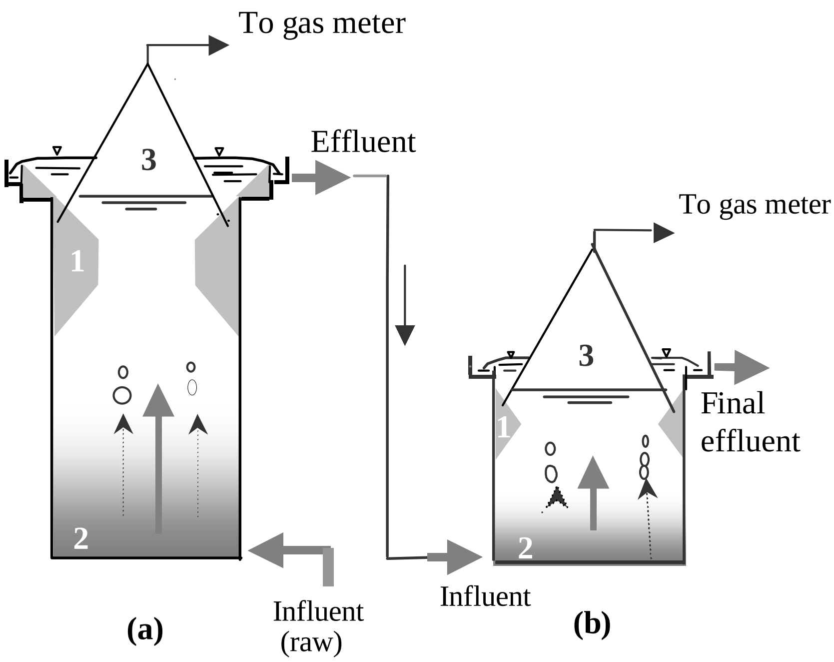
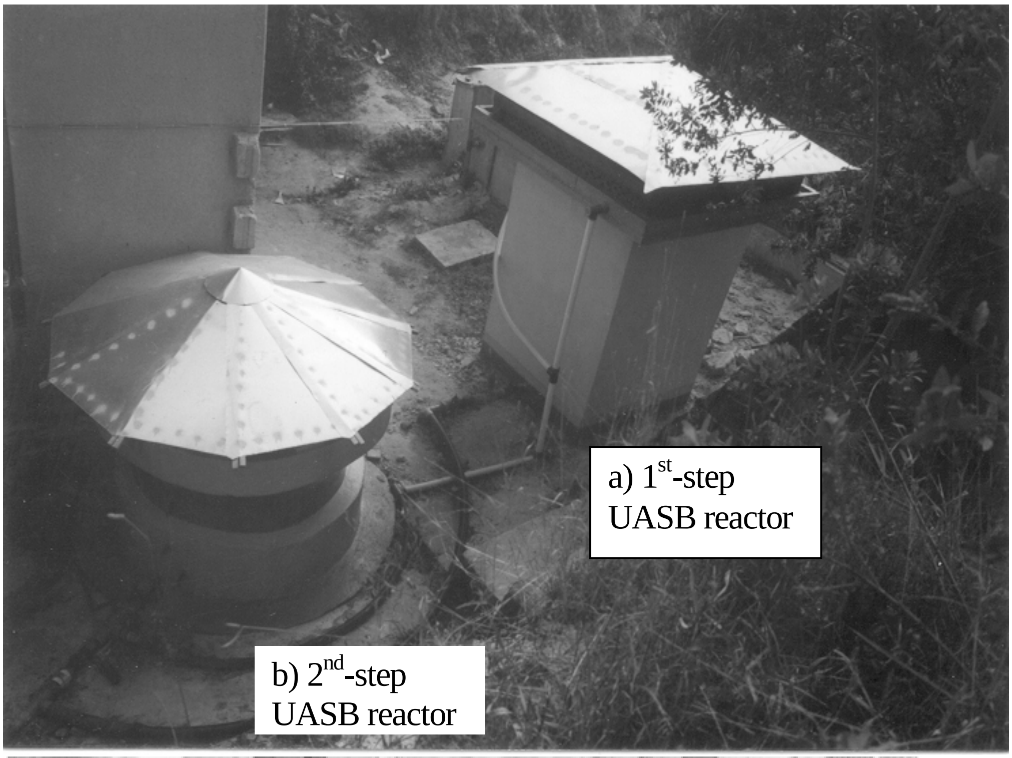


Figure 6.1a: Schematic diagram of a pilot UASBserial two-stage one-phase treatment process system configuration. (a), 2m tall 1.8m^3 UASB reactor; (b), 1m tall 0.85. m^3 UASB reactor, connected in series to (a); 1, baffle; 2, Sludge bed; 3, Gas collector.



c) 2nd-step UASB reactor effluent discharge channels

Figure 6.1b: Pilot two-step UASB reactor configuration.
(a), 2m tall 1.8m^3 UASB reactor; (b), 1m tall 0.85.
3

The pilot plant was fed with wastewater originating from students' dormitories and a cafeteria under continuous gravity flow. The flow characteristic in terms of volumetric rate and concentrations of organic load fluctuated all the time similar to what was fed to single-step UASB reactor in Chapter 5. The values for COD_{tot}, COD_{ss}, COD_{col}, COD_{sol} of the wastewater can be found in Figure 2.2 starting from day 665. The average values for influent wastewater to the Two-Step pilot plant for COD_{tot}, COD_{ss}, COD_{col}, COD_{sol} were 537.2 (165.3)[1], 189.9 (109.5), 127.4 (75.0), and 223.4(108.8) mg/L respectively. The hydraulic retention times (HRT) of the 2m and 1m high reactors are about 5 hours and 2.4 hours respectively and overall HRT for the pilot Two-Step UASB reactor was 7.4 hours, i.e. 5 + 2.4 hours. The pilot Two-Step UASB reactor treatment system started operating without seeding. The pilot plant was operated at ambient temperatures 25 - 34 ºC.

Sampling of influent, effluent and analyses procedures are similar to those applied for the pilot single-step UASB reactor discussed in Chapter 5.

6.2 Results and discussion

6.2.1 Performance of Two-Step UASB reactor configuration

Table 6.1 shows the quantitative pollution strengths of wastewater fed to the Two-step UASB reactor configuration and the quality of effluent discharged from the system during the research period. Figure 6.2 shows the daily performance of the anaerobic pre-treatment by Two-step UASB reactor configuration in terms of removal efficiency of the different fractions observed during the conduct of routine sampling over a period of 630 days. The average performances of the system is depicted in Table 6.2

Table 6.1: influent and effluent values for the 1st step and overall system

	influent	1st-step Effluent	overall system effluent
CODtot	537 (165)	308 (109)	167 (102)
CODss	190 (109)	114 (63)	72 (55)
CODcol	127 (75)	83 (54)	39 (41)
CODsol	223 (109)	113 (75)	59 (58)

The overall performance of the Two-step UASB reactor configuration shown in Table. 6.2 is far better than of the individual reactors separately. The results in Tables 6.1 and 6.2 show that the washed out COD fractions, particularly solids (SS) and colloids, from the first-step UASB reactor are further removed by the second-step UASB reactor.

The average percentage removals in Table 6.2 show that the second-step UASB reactor achieved fairly well the research objectives of removing the wash-outs, particularly SS and colloids and polish further the effluent from the first-step UASB reactor. The performance of the second-step UASB reactor even is better in removing SS and colloids than the first-step UASB reactor despite the short HRT. Considering

[1] Standard deviation value

also the poorly settleable character of SS-fractions present in the effluent of the first step, the efficiency of the – in fact overloaded – second reactor nevertheless is surprisingly high, the more so because the amount of sludge present in the system was quite low during major part of the experimental period.

If the effluent values of the first step as they are presented in Table 6.1 are compared to the values of the single-step reactor of Chapter 5 (Table 5.1) the results appear to show that the first-step of the two-step system performance worse than the single-step, especially with the removal of soluble COD. However Table 6.3 shows the results of the calculated hourly removal rates for CODsol per litre of influent pollution load based on respective HRTs for the first 500 days, (i.e. when the sludge beds of the respective reactors just reaches a height of about 50 cm) from the start of operation of the respective reactors. The results show that the removal rates for CODsol are 15, 21 and 23 mg/L.h respectively for the single-step and the first- & second-step of the two-step UASB reactors configuration. The second-step UASB reactor of the two-step system has the highest removal rate for CODsol. The first-step UASB reactor has a better removal rate for CODsol compared to single-step UASB reactor. Moreover, the first-step reactor was started without any seeding and with the passing of time and increase of the sludge bed the removal of CODsol appears to increase (Fig 6.2). The relatively low removal efficiency for CODtotal, CODss and CODcol achieved by the first-step UASB reactor as compared to single-step UASB reactor (Chapter 5) during the first 500 days of operation as observed in Table 6.3 can partly be attributed to the fact that the two-step system was started without seed sludge and that the first 200 days of operation of the single-stage system were not taken into account.

Table 6.2: Performance of 1st- and 2nd –step UASB reactors connected in series. Standard deviations are presented in brackets (Temperature 25-34 °C).

Step of UASB reactor	Vol. (L)	HRT (h)	Influent COD_{tot} (mg/L)	Average removal efficiency (%)			
				COD_{tot}	COD_{ss}	COD_{coll}	COD_{sol}
1st-step (*630 days*)	1800	5 (1.1)	537 (165)	42.5 (16.7)	23.7 (49.9)	24.6 (58.9)	46.5 (29.7)
2nd-step (*630 days*)	852	2.4 (0.7)	308 (109) (i.e. effluent from 1st-step)	47.8 (19.7)	26.0 (62.3)	41.8 (54.4)	44.9 (54.2)
Overall system (*630 days*)	1800 + 852	7.4 (1.6)	537 (165)	68.7 (16.7)	51.2 (41.8)	62.1 (38.2)	71.8 (30.5)

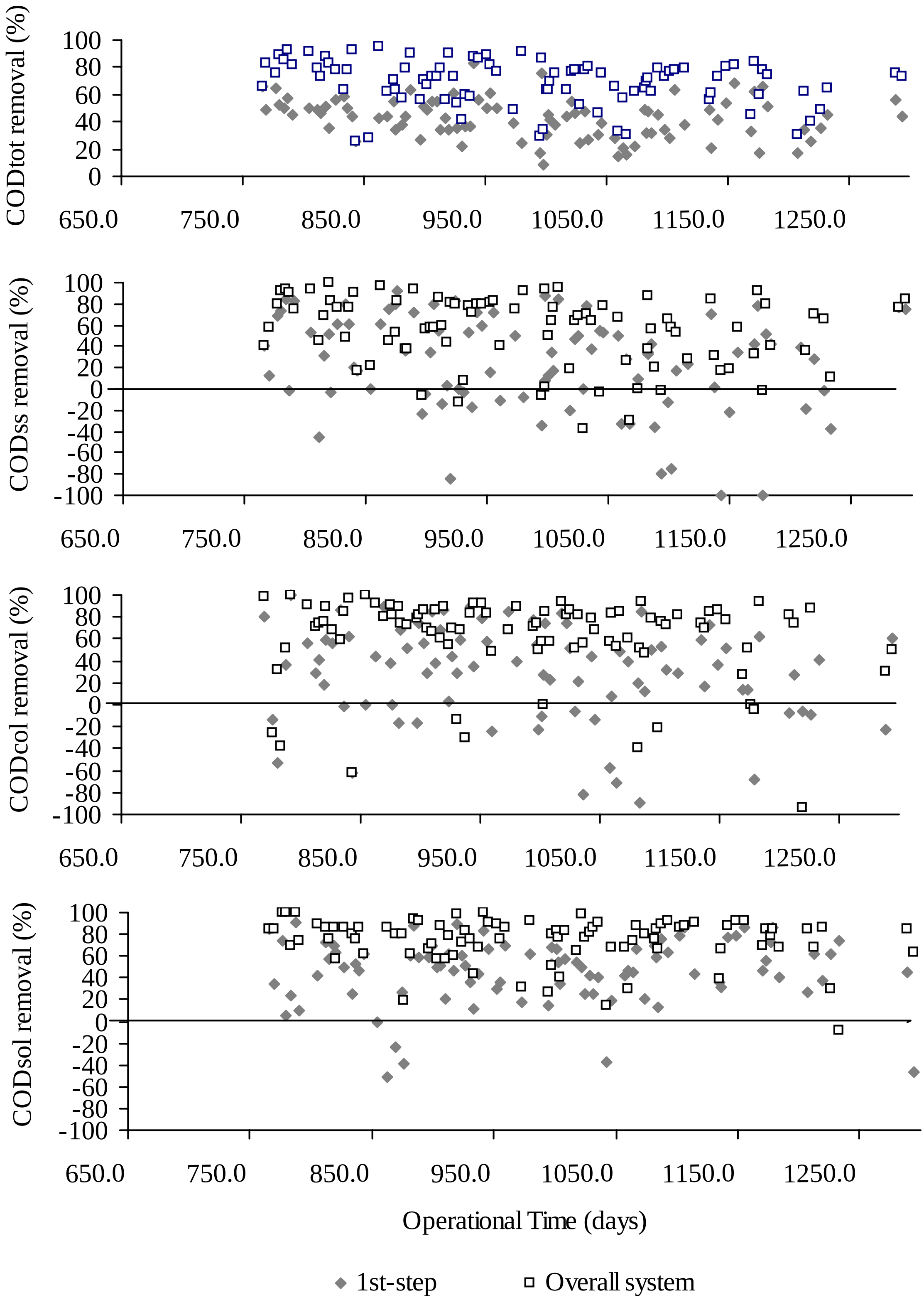


Figure 6.2:Comparison of percentage removal efficiencies for CODtotal and COD-fractions overtime by 1st-step UASB-reactor and Overall system during research period

Table 6.3: Comparison of results of single-step and the Two-step UASB reactors (first- and second-step) over the first 500 days from start of the operation of the respective reactors. Values between brackets are standard deviations.

	Average influent and effluent values (mg/L)								Rate of CODsol removal (mg/L /hour)
	CODtot		CODss		CODcol		CODsol		
	Infl-uent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent	
Single-step (seeded) (*Chapter 5*)	541 (698)	131 (96)	308 (588)	48 (45)	130 (367)	50 (79)	139 (117)	48 (56)	15
First-step (unseeded) (of Two-step system)	537 (165)	307 (109)	192 (104)	119 (68)	130 (77)	81 (55)	217 (113)	110 (76)	21
Second-step (unseeded) (of Two-step system) Effluent		163 (102)		72 (55)		37 (35)		54 (52)	23

Table 6.4 summarises removal efficiencies achieved in UASB reactors operating in other environments but with comparable tropical temperature conditions, viz. temperatures exceeding 20 ^oC. It should be taken in mind that the results in Table 6.4 concern centralised sewage treatment systems whereby during longer periods of time constant flow rates were maintained, unlike this research conducted in our pilot plant operated at community level at highly variable daily hydraulic flow loads. Results of the research of a single-step UASB reactor are incorporated in Table 6.4 as additional information on pilot plants operated in tropical regions. A major problem we experienced in the single stage UASB-reactor comprised TSS-wash-out, which was related to the occurrence of scum layer formation. The problem of washouts of TSS and colloids and the occurrence of scum layer (floating sludge) was also experienced by Halalsheh (2002) while operating a big Two-stage pilot plant, consisting of a first reactor of 60 m^3 and a second reactor of 36 m^3. Reactor in Jordan. The removal efficiencies observed there for COD_{tot} for first stage and second stage were 51% and 4-5% respectively. However the reactors were treating high strength sewage with COD_{tot} in the range of 1500-2000 mg/L.

Table 6.4: Summary of recent results for treatment of sewage under tropical conditions (>20 ^{o}C) in pilot and full-scale systems (Elmitwalli, 2000). Standard deviations in brackets.

Reactor	Volume	Temperature	HRT	Influent COD_{tot}	% Removal		Reference:
	(M^3)	(^{o}C)	(h)	(mg/l)	COD_{tot}	SS	
UASB	64	24-26	4-6	267	65	70-85	Shellinkout, *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	Vieira. 1988
UASB	1200	20-30	6	563	74	75	Draaijer, *et al.*, 1992
UASB	6600	25	5.2	380	60-80	-	Shellinkout, *et al.*,1992
UASB	1.5	25-34	6.2 (4.92)	529 (545.6)	64.2 (19.2)	56.5 (37.3)	*this research (Chapter 5)*

6.2.2 Sludge bed development

The results in Figures 6.4 & 6.5 show the sludge bed development for the 1st- and 2nd-step UASB reactors during the research period. As can be observed in Fig. 6.4 & 6.5 that both UASB reactors show a very similar pattern of sludge bed growth along the height of the reactor. The picture is very similar to what was witnessed already in Chapter 5 for the single-step 1.5 m^3 and 1.7m tall operated UASB at an average HRT of 6.1 hrs. Granular sludge was also developed in both 1st- and 2nd-step UASB reactors similar to what was observed in Chapter 5 (Fig. 5.4). However the granular sludge developed in the 1st-step UASB reactor was bigger in size than those developed in 2nd-step UASB reactor. The granular sludge developed in 1st-step UASB reactor were similar in size to those observed in the single-step UASB reactor, Chapter 5, Fig. 5.4.

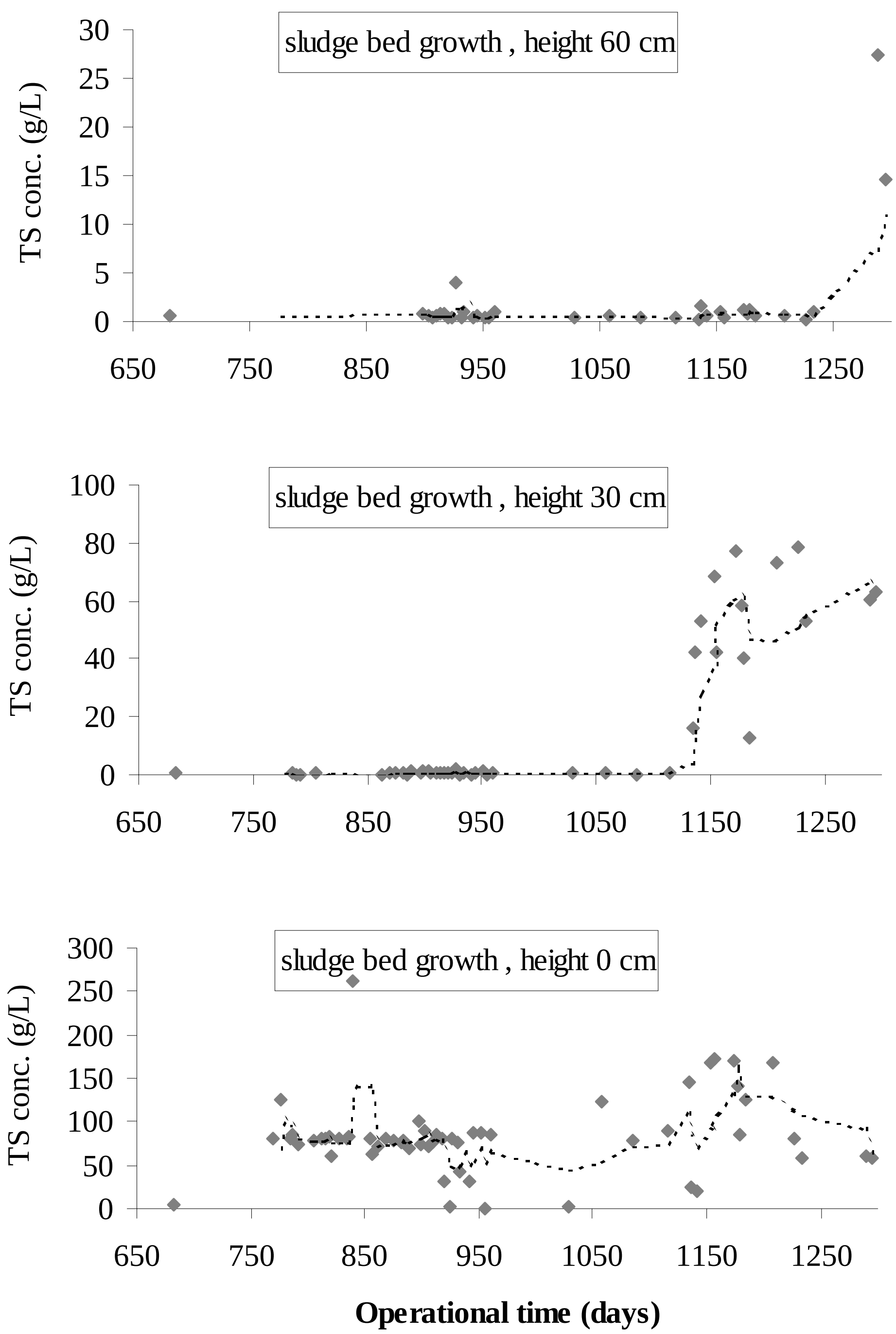


Figure 6.3 development of the sludge bed in the first step reactor

Figure 6.3 shows that sludge bed growth in the first-step UASB reactor at a height of 30 cm as monitored at day 453. During this period the TSS concentration at the bottom of the reactor remained more or less at an average value of 80 gTS/L. This implies that most of the TSS removed by the reactor was held in the floating sludge layer. The results in Figure 6.4 demonstrate, in fact similar as we already found Chapter 5 that a TSS-concentration of about 80gTS/L can be maintained for longer periods of time, but suddenly can drop. The sudden increase in TSS concentration that manifests reflects that settling sludge originating from the floating sludge layer on the sludge bed is taking place.

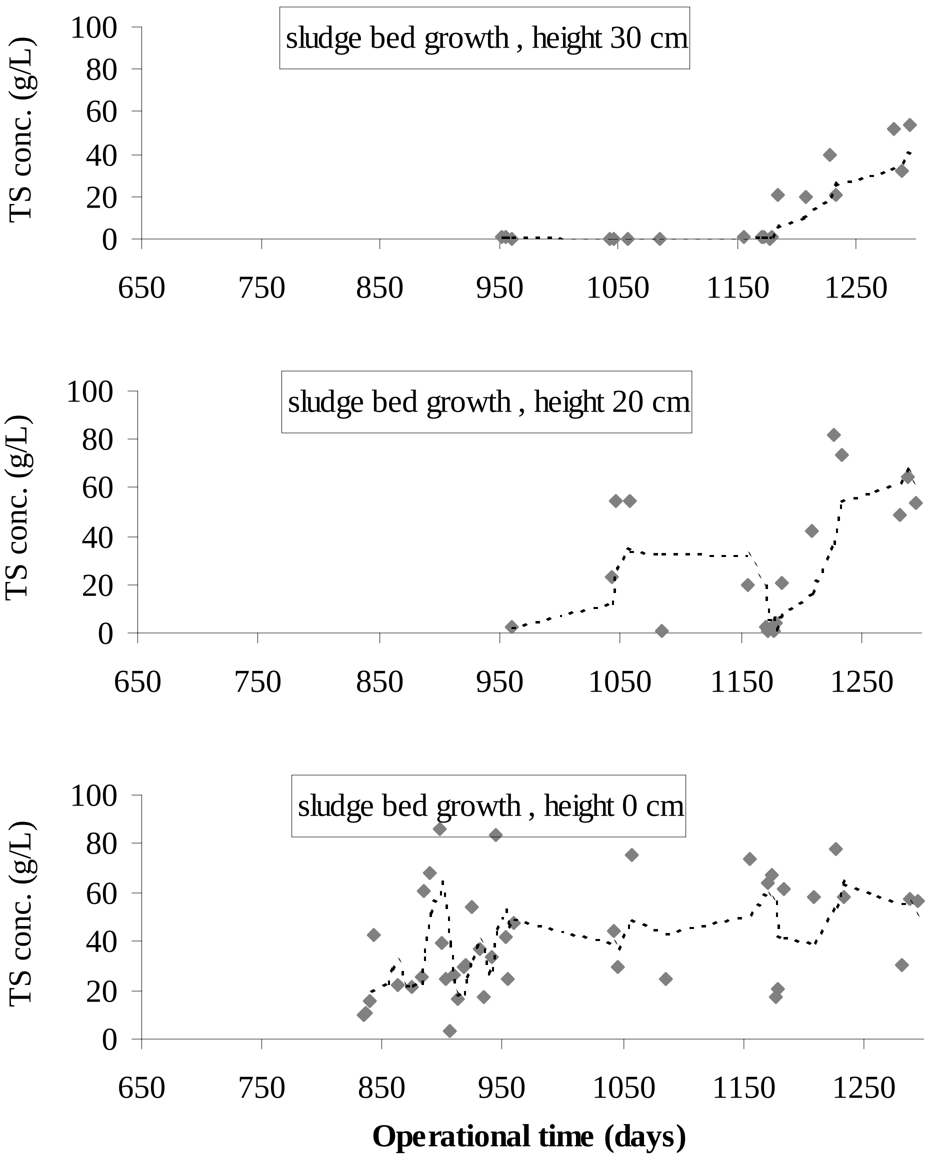


Figure 6.4 Development of the sludge bed in the second step reactor

As explained already in Chapter 5 the consequence of this situation is an increased wash-out of TSS and colloids from the reactor occur. However in the case of a two-step reactor configuration these washouts lead much less to a deterioration of the effluent quality, as is the case for a single-step UASB reactor. Figure 6.4 shows a similar sludge bed development for the second-step UASB reactor. Figure 6.5 presents the growth of the sludge bed in a simplified manner. In this graph the presence of the sludge at the respective sampling port heights (0, 30 and 60 cm) is indicated for TS concentrations above 25 g/l (similar to Figure 5.6). It also presents the cumulative collected methane over the researching period. It is can be seen from Figure 6.5 that from day 950 to day 1150 the gas collection was about 137 l/day and from day 1300 to day 1313 it was 245 l/day. The difference can be attributed to some blockage of the GLS by the scum layer. Unfortunately the relation between the gas collection and settling of the scum layer as observed in Figure 5.7 can not be seen in Figure 6.5 due to the fact that the gas meter was out of order from day 1125 to day 1300.

Sludge activity

The results of specific methanogenic activity (SMA) test conducted on sludge samples withdrawn from bottom and at a height of 30 cm of the 1st-step UASB reactor show a rather variable picture. The maximum SMA for sludge at the bottom of the reactor was relatively low with a value of 0.061 gCH4-CODper gVSS per day, while that at a height of 30 cm with a value of 0.135 gCH4-COD per gVSS per day was significantly higher, i.e. in the range found for the sludge in the single step UASB-reactor (Chapter 5). The SMA of the for sludge samples withdrawn from the bottom of and at heights 20 and 30cm the 2nd-step UASB reactor amounted to 0.272, 0.260 and 0.104 gCH4-CODper gVSS per day respectively. The values comparable well to those assessed for the sludge present in modified UASB-septic reactor treating black water at the Biofarma site, Bandung, Indonesia (IHE & WAU, 1991). The range of maximum SMA-values observed in this research reactor were in the range 0.017-0.228 gCH4-CODper gVSS per day. Observed results of activity tests reported a maximum SMA-value of 0.10 and 0.15 gCH4-CODper gVSS per day (at 33 ºC) for the 60 m^3 experimental UASB-reactor operated in Jordan (Halalsheh (2002)), for winter and summer respectively. The quality of sludge present in our reactors was within the range of observed values elsewhere in similar installations (Lettinga, *et al.*, 1987, Grin, *et al.*, 1983).

Contrary to the findings of Halalsheh (2002) the SMA of the sludge present in the 2nd-step UASB reactor has distinctly higher than for the sludge in the 1st-step UASB reactor. These differences are probably due to the fact that the COD_{sol}/COD_{ss} ratio in the influent of the second step reactor operated by Halalsheh (2002) were lower than the COD_{sol}/COD_{ss} ratio in the influent of the second step UASB operated in the research presented in this chapter.

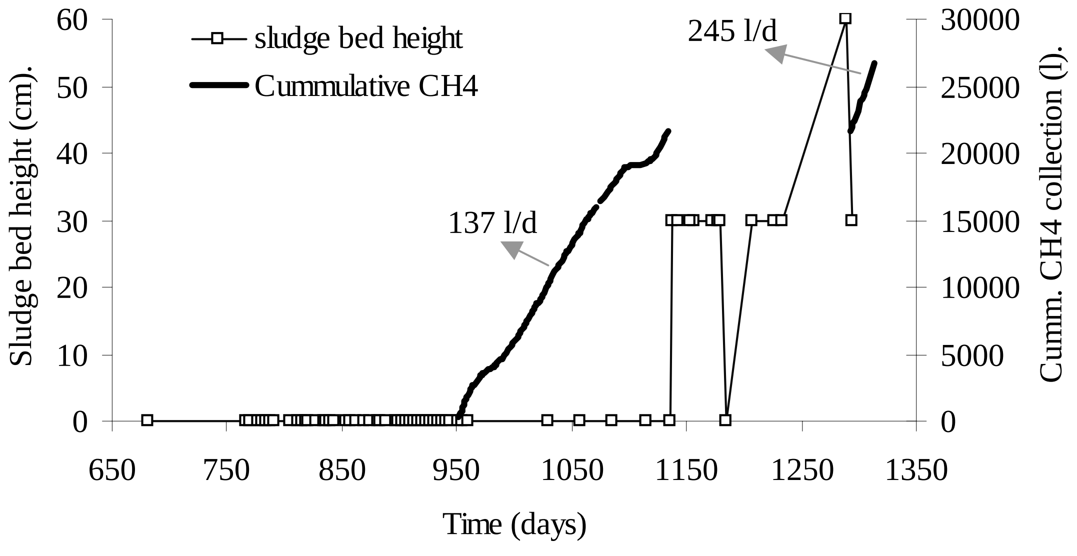


Figure 6.5: Correlation between the sludge bed growth rate and collected methane in the first step of the two-step UASB system.

6.3 Final discussion

A research on the two-step UASB reactor configuration is a follow up of the findings made on a previous research on the performance of a pilot single-step UASB reactor. The pilot two-step UASB reactor configuration was operated in parallel from day 665 of the research period to the then on going single-step UASB-reactor. The two researches were conducted under the same environmental conditions. As has already been demonstrated in Chapter 5 that ejection of washed out COD fractions from the single-step UASB reactor was an outcome of the dynamics of floating sludge and high biogas evolution that took place in that first (/single)-step reactor. So the addition of the second-step UASB reactor in the parallel pilot two-step UASB reactor configuration was intended to contain the washouts and polish them further. The research objective has been met in this aspect. The second-step UASB reactor was able to perform as expected, that is to contain the washouts and further polish the effluent from the first-step UASB reactor. The second-step UASB reactor of the two-step system has the highest removal rate for COD_{sol} in terms of pollution load that is removed in a litre of hydraulic load treated per hour then the rest of the steps of UASB reactors configuration. The overall removal efficiency for COD fractions for the two-step UASB reactor configuration was far better than individual reactors. The average overall removal efficiency with standard deviation in brackets for COD_{tot}, COD_{ss}, COD_{col} and COD_{sol}, were respectively 68.7 (16.7), 51.2 (41.8), 62.1(38.2), and 71.8 (30.5) %. The overall HRT was 7.4 (1.6) (i.e. 5 + 2.4) hours. The maximum specific methanogenic activity of sludge developed in the 1st-step reactor ranged from 0.061 – 0.135 gCH4-CODper gVSS per day and for the 2nd-step reactor the range was from 0.104 - 0.272 gCH4-CODper gVSS per day.

The advantages of the two-step UASB reactor configuration include 1) the distinct higher overall removal efficiency of the anaerobic pre-treatment system 2) the higher sludge age 3) the higher reliability of the anaerobic pre-treatment process 4) the two reactors can separately be operated in case of technical problems. The reactor system can be designed with a by-pass option such that in moments of emergency each separate reactor can take extra load of the other without adverse effects and without shutting down the plant. This would make the Two-Step UASB reactor pre-treatment configuration more robust and stable, consequently more reliable for community on site wastewater pre treatment needs.

In this research the two UASB reactors employed were physically separated spatially. However in actual field conditions these two reactors need not be spatially separated physically, but instead the Two-Step UASB reactor configuration can be constructed as a one compact unit comprising two reactors separated by a common wall in between them. *Conversely*, the two modules of this integrated two-step reactor each are operated at their own loading conditions, i.e. the effluent from the first module comprises the feed for the second module. From the outside the reactor assembly may look physically like a septic tank equipped with two gas collectors on top of them and the adequate feed distribution system and baffles. In terms of construction it remains extraordinary simple and low cost, and extremely robust, particularly when gravity flow can be applied. It then does not require any external supply of energy. As a matter of fact it comprises a sustainable solution for sanitation in those situations were for some reason it has been decided to (mis)use clean water for conveying human residues -and wastes to some on site treatment system. The anaerobically pre-treated sewage in principle is very suited for reuse as irrigation and fertilisation, particularly in tropical countries were crops could be grown all the year.

Finally it can be concluded that as has already been demonstrated in this research the Two-step UASB reactors configuration is a potential compact and effective community onsite pre-treatment unit for domestic wastewater. The system is more economical and affordable for local relatively poor communities since it can operate successfully without seeding and does not require any external supply of energy particularly when gravity flow mode can be achieved.

6.4 Conclusion

The results of the performance of the Two-Step UASB reactor configuration, which certainly was far from an optimal design - are encouraging. The results of this research clearly demonstrate that it is well possible to contain washouts in a second-step UASB reactor placed in series to the first-step UASB reactor. It will lead to distinct improvement of the effluent quality; consequently better overall performance of the anaerobic pre-treatment system. Despite the short HRT, 2.4h, for the second-step UASB reactor, (only) 1 m high, and its performance was rather satisfactory. However more study is needed for the second step to obtain steady state so that the design of the reactor system can be properly evaluated. For now it is uncertain if the relative short height of the reactor will promote wash out of suspended solids. The hydraulic conditions for sludge settling in a short reactor are less favourable, for flocculation and (primary and secondary) sludge particle entrapment in the sludge bed present in the system. Such dimensions also to some extent will decrease the possible settling of floating (scum layer) sludge by damping turbulence (Chapter 5) that might arise within the reactor in the event of high biogas evolution as was observed in the performance of the single-step UASB reactor in Chapter 5.

6.5 References

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

Breure, A.M.(1986) Hydrolysis and acidogenic fermentation of protein and carbohydrates in anaerobic wastewater treatment, *PhD thesis,* Amsterdam University, The Netherlands

Breure, A.M.(1993) Phase separation in anaerobic digestion, In:Anaerobic Reactor Technology, International Course on Anaerobic Wastewater Treatment, IHE Delft and Wageningen Agricultural University

Clesceri, Lenore S., Greenberg, Arnold E., Trussell, Rhodes R. (1989) Standard methods for the examination of water and wastewater. American Public Health Office/American Water Works Association/ Water Pollution Control Federation. Washington USA, 17th edition, 1989.

Elmitwalli, A.T., (2000) Anaerobic treatment of domestic sewage at low temperature. *PhD thesis*, Department of Environmental Technology, Wageningen University, The Netherlands

Halalsheh, Maha, M. (2002).Anaerobic Pre-treatment of strong sewage. A proper solution for Jordan. *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen.

IHE, Wageningen Agricultural University (1991). Upflow Anaerobic Sludge Blanket (UASB): Low Cost Sanitation Research Project in Bandung/Indonesia. *Final Report*

Last, A.van der, Man, A. de, Lettinga G. (1993) Anaerobic treatment of domestic sewage under Dutch conditions. In: In:Anaerobic Reactor Technology, International Course on Anaerobic Wastewater Treatment, IHE Delft and Wageningen Agricultural University

Lettinga, G., Man, A.d., Last, A. R. M., v. d., Wiegant, W., Knippenberg, K., Frijns, J. and Buuren, J. C. L. v. (1993) Anaerobic treatment of domestic sewage and wastewater. *Wat. Sci. Tech*., **27**(9), 63-73.

Lettinga, G. (1993) Anaerobic Lab. Work (Manual). International Course on Anaerobic Wastewater Treatment (June 22- August 4, 1993). IHE, Delft & Wageningen Agricultural University, Wageningen, The Netherlands.

Mahmoud, Nidal, J. (2002) *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen

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

Wang, K. (1994) Integrated anaerobic and aerobic treatment of sewage. *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen.

Zeeman, G. and Lettinga, G. (1999) The role of anaerobic digestion of domestic sewage in closing the water and nutrient cycle at community level. *Water Sci. Tech.* **39**(5), 190-91

Zeeman, G. and Sanders, W. (2001) Potential of anaerobic digestion of complex waste(water) *Water Sci. Tech.* **40**(8), 115-122

7 SUMMARY, DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

1 General introduction

Onsite wastewater treatment at individual residential dwellings or at community level has often been over looked in anticipation that one day all domestic wastewaters will be drained off through centralised sewerage systems and treated off site in a centralised wastewater treatment plant. But the reality is, that the latter is not always the case particularly in the developing world. That day never comes that soon. Meanwhile ‘low rate’ anaerobic pre-treatment units have become wide spread normally with little attention paid in their performance after construction as a result public health and the environment have become victims of false hope. Citing the case of Tanzania, the wide spread use of pit latrines and septic tanks, particularly in the urban and peri-urban areas makes their presence too numerous to manage and thus they pose increasing environmental pollution problems. An effective community onsite anaerobic wastewater pre-treatment system will be an attractive option under such circumstances since the number of onsite systems will be reduced to a manageable figure, the public health and environment will be further protected.

The major reason for the non-implementation of the conventional centralised sewerage system that is normally accompanied by a centralised wastewater treatment plant is that the system is certainly far too expensive and complex for poor countries (Zeeman, *et al*., 2001). Even so the sanitary engineers have also been too busy with larger treatment works to give attention to the onsite wastewater treatment units at individual residential dwelling or at community level (Viraraghavan, 1976) under the false assumption that ultimately the onsite wastewater treatment systems will be connected to the central sewerage system (Otis, *et al.*, 1994). Moreover, the sanitary engineers and associated responsible professions have not fully adapted themselves to the actual changing field needs of onsite wastewater system design for increased performance, resource recovery and environmental protection. Consequently effective, compact and cost-effective onsite wastewater treatment facilities have neither been researched enough in terms of scaling down the proven effective wastewater treatment technologies to prescribe to hydraulic and organic loading characteristics as well as resource recovery for different application scales in actual field conditions nor the technology dissemination to practitioners has not been that effective. For example, the major barriers for the rapid implementation of high rate anaerobic wastewater treatment (AnWT) technology in practice include the lack of knowledge and experience with AnWT performance (Lettinga, 2001).

However, on the basis of already available technical information concerning the UASB performance, the treatment process of domestic wastewater using the UASB reactor can possibly be stabilised at individual residential dwelling or at community level to a simple operational form and thereby come up with a proper community onsite pre-treatment unit that is compact, effective and low cost for tropical wastewaters, citing the case of Tanzania. Therefore instead of indiscriminate full-scale field installations based on performance data derived elsewhere, a study of pilot scale UASB reactors constructed at community level onsite for the purpose of gathering performance data that will confirm the feasibility at local conditions prior to full-scale installations is of vital importance.

This thesis presents the results obtained during the monitoring operations of constructed pilot scale, community onsite UASB reactor configurations as well as the results of a monitoring operation of a full-scale onsite community septic tank. The results give expected performances should the tested pilot-scale reactors be opted for full-scale installations. All monitored reactors were operated in parallel at ambient temperature ranging 25-34 ºC, though the start up was at different times during the research period.

2 Wastewater characteristics and flows

Unlike the centralised wastewater treatment plant the major constituents of the wastewater at community level are derived from the domestic sources compared to the other categories such as industrial, commercial, storm water in case of combined sewers and groundwater infiltration. The quantity and strength of domestic wastewater depends on the size and the socio-economic behaviour of the population constituting the community. In this research pollution concentration distribution of the wastewater over a day at community level was found to be highly variable with an hourly average of 564 mg COD/L and standard deviation of 230 mg COD/L whereas less variations were found while conducting a similar monitoring exercise of wastewater quality over a day for an existing centralised municipal wastewater discharges. The centralised municipal wastewater discharges had an hourly average of 470 $mgCOD_{tot}/L$ with a standard deviation of 98. The average ratio COD: BOD_5 found for the wastewater at the research area was 1.52 with standard deviation of 0.13. These variations constitute the typical characteristics of wastewater at community level. Small system flow rates and wastewater characteristics differ significantly from those of large systems (Metcalf & Eddy, 1991). The wastewater treatment plant to be installed on site at community level ought to be able to withstand these great variations in constituent concentrations and hydraulic loads without adverse effects on the effluent quality. The suspended solids and organic (biodegradable) material being the main constituents of sewage at community level can be highly reduced in concentrations using modern high rate anaerobic pre-treatment units such as Upflow Anaerobic Sludge Bed (UASB) reactors. Compared to clarigesters, the UASB reactors perform better with regard to the removal of COD and BOD and just about equally with respect to TSS (Wiegant, 2001).

In view of the fact that UASB reactors are high rate anaerobic systems then even at peak times of substrate supply they should be able to perform satisfactorily. However at very high upflow velocities, over 1.0 to 1.5 m/h (i.e. mostly happening at peak times in case of gravity flow operation mode) inclusion of a polishing step after a UASB reactor may be justifiable (Wiegant, 2001).

3 Interpretation of anaerobic sludge stabilisation assays for comparative stability assessment

All wastewater treatment processes produce quantities of wastewater material in the form of dilute solids mixtures known as sludge. The composition, solids content and stability of the sludge is a function of the characteristics of the raw wastewater flow and the treatment process background that generated the sludge. The excess sludge production in the anaerobic pre-treatment, i.e. the sum of the sludge accumulated in the reactor and washed out from the reactor is in the range of 5.0-8.6 kg TS per population equivalent per year, which is only 25-40 percent of that in a conventional aerobic treatment system combined with sludge digestion (Lettinga, *et al.,* 1983). At some stage during the reactor operation the accumulated sludge inside the effective volume of the reactor needs to be disposed of. A stable sludge is the one normally recommended for disposition. Sludge is stable when it has a high activity with a low fraction of biodegradable material (Haandel & Lettinga, 1994). From the literature it is clear that different approaches have been used in determining the stability of anaerobic sludge, moreover that the results of these assays are not very well comparable. In Chapter 3 a method for interpretation of stability assays is proposed. The method makes use of a simple model for the biological processes during a stabilisation test and the possibility to use computer software for estimation of the parameters of this model. With the method the percentages of biomass, degradable components and inert component in a sludge sample can be estimated. The proposed method was illustrated with stability assays with sludge from a UASB, Septic tank and Pit Latrine. The results showed that the sludge from the UASB and septic tank contained 21 and 18% biomass, respectively. No degradable solids were detected in these sludges indicating that the sludge from the UASB and septic tank were fully stabilised. The sludge from the pit latrine contained an undetectable amount of biomass. The amount of degradable components constituted 17% of the sludge.

4 Performance of a three-compartment full-scale community septic tank (volume: 56.3 m^3; height: 1.93 m)

A septic tank is the most known and commonly applied method for onsite anaerobic pre-treatment of sewage (Zeeman, *et al.,* 2001). In Tanzania Septic tanks are the most widely used community onsite wastewater pre-treatment units, however for individual residential dwellings septic tanks are second to pit latrines as the most used anaerobic pre-treatment facilities for domestic wastewater.

A full-scale community onsite septic tank constituting three-compartments was monitored for its performance for duration of 400 days. The septic tank was constructed in 1982 and treats wastewater from one of the hostel flats accommodating students at a University College of Lands and Architectural studies (UCLAS), Tanzania. The wastewater originates from the Hostel toilets, washing stalls and some apartment kitchens. The effective volume of the septic tank is 56.3 m^3. The objective of this research was to examine the treatment efficiency and performance limitations of a community full-scale septic tank for the purpose of determining its suitability as a wastewater treatment option at community level.

The performance of the septic tank during the monitoring period and from the literature survey is found to be poor. In this research the removal efficiency for COD_{tot}, COD_{ss}, COD_{col} and COD_{sol} were respectively 29 (26), 29 (59), 16(21) and 16 (21) percent. Compared to other septic tanks reported in literature the performance was poor. Washouts of COD fractions were often observed particularly that of COD_{sol}. The average theoretical HRT observed during the periods of taking samples was 5.6 (8) days. The observed worse performance of the septic tank despite the long HRT is mainly attributed to the horizontal flow mode of the influent sewage in septic tanks and the poor management of this particular septic tank. The long HRT requirement for septic tanks also leads to demand of large space and relatively high cost of installation (Kalbermatten, *et, al*., 1980). These qualities limit the application of the septic tanks in congested urban and peri-urban areas that need low cost and effective on site sanitation systems.

A significant low-cost/ low-tech improvement of the septic tank could be achieved by applying modern reactor technology to the system, i.e. upward flow and gas/solids/liquid separation at the top (Zeeman, *et al.*, 2001). These most essential features that need to be incorporated in septic tank so as to improve the performance will lead to features of the Upflow Anaerobic Sludge Bed (UASB) reactor (Bogte *et al.* 1993). Hence UASB treatment unit is the simplest available technology that can best be employed in the treatment of wastewater onsite both at individual residential dwelling and at community level.

5 Performance of a pilot single-step community on-site uasb reactor pre-treating tropical domestic wastewater

A pilot single-step community on-site upflow anaerobic sludge blanket (UASB) reactor (capacity: 1.5 m^3, height: 1.7 m) was operated over three and a half years treating part of the domestic wastewater from a University College of Lands and Architectural studies (UCLAS), Tanzania. The source of wastewater was from the cafeteria, toilets of some students' dormitories, and partly from university staff residential houses. The reactor was operated under gravity flow mode. The research on pilot-scale community onsite single-step UASB reactor was meant to raise the actual field performance results that could be accounted for in Tanzanian and elsewhere with similar environmental conditions while designing and installing future comparable full-scale plants. The average influent organic pollution of wastewater amounted to 529 $mgCOD_{tot}$ /L with extreme values for the minimum and maximum, i.e. 46.4 and 4109 $mgCOD_{tot}$/L respectively. The temperature range of the wastewater was typical for tropical regions, viz. 25–34 ºC. The applied hydraulic retention time (HRT) of the reactor varied i.e. the observed hydraulic retention times ranged from 1.7 to 40 hours, with an average value of 6 hours. Performance data obtained via regular monitoring of the treatment unit showed a declining removal efficiency over time with respect to COD_{tot}, which likely can be attributed to the increasing rate at which biogas was produced along with the growth of sludge bed and the presence of floating sludge. As a result the removal of dispersed sludge particles becomes poorer, which likely is reinforced by the 'less' optimal dimensions and design of the Gas-Solids-Separator (GSS) device. The average removal efficiency on COD_{tot} basis was 64 percent. Higher removal efficiencies of influent organic loads could be attained provided the problem associated with the settling of floating sludge-associated washouts of COD fractions is addressed. The problem identified is the decline of the overall treatment efficiency of the treatment unit over time. The decline of treatment efficiency starts when the sludge-floating layer begins to collapse and occasional falling continuing thereafter followed by high biogas evolvement. The sludge bed initially, in this particular case about the first 200 days of operation, does not grow and the TS concentration is steadily maintained in the range of 75-80 gTS/L. The sludge bed concentration at the bottom of the reactor of 75-80 gTS/L appears to be maintained throughout the research period and it appears that TS concentration in excess of this range is not stable and therefore this range is maintained by letting some sludge to buoy. Prior to the collapse of the floating sludge layer the removal efficiencies of COD fractions were high. Under such situations a two-step UASB pre-treatment unit constituting two UASB reactors connected in series might be a solution since the second-step reactor will be a recipient of the effluent from the first step reactor. Consequently the second-step UASB reactor will accommodate the washouts resulting from the settling of the floating sludge layer that is taking place in that first-step UASB reactor and thereby raise the overall treatment efficiency of the unit.

6 Performance of an effective compact community on-site pilot two-step UASB reactor pre-treatment unit for tropical domestic wastewater

The two-step UASB reactor configuration is a follow up of the findings made on a previous research, (Chapter 5) on the performance of a pilot single-step UASB reactor. The two-step UASB pre-treatment unit in this research refers to two UASB reactors connected in series, viz. a first 2 m high, 1.8 m^3 UASB reactor put in front of a second 1m high, 0.852 m^3 UASB reactor. This research was operated in parallel to the then on going single-step UASB-reactor from day 665 of the overall research period and the investigations were conducted over a period of 630 days. The two researches viz. the single-step and two-step UASB reactor configuration, were conducted under the same environmental conditions. The imposed overall HRT for the two-step UASB reactor configuration was 7.4 (1.6) (i.e. 5 + 2.4) hours. The treatment process was started without any seeding and was operated at ambient tropical temperature of 25 - 34 ºC. The maximum specific methanogenic activity of sludge developed in the 1^{st}-step reactor ranged from 0.061 – 0.135 gCH4-COD per gVSS per day and for the 2^{nd}-step reactor it was in the range 0.104 - 0.272 gCH4-CODper gVSS per day. The research findings in Chapter 5 have shown that washed out COD fractions from the single-step UASB reactor was an outcome of the dynamics of floating sludge and high biogas evolution that took place in that first (/single)-step reactor. So the addition of the second-step UASB reactor in the parallel pilot two-step UASB reactor configuration was intended to contain the washouts and polish them further for the purpose of achieving overall optimal reactor operation performance. The average organic loads of the wastewater imposed to the system with respect to COD fractions COD_{tot}, COD_{ss}, COD_{col} and COD_{sol} were 537.2 (165.3), 189.9 (109.5), 127.4 (75) and 223.4 (108.8) respectively. The overall removal efficiency obtained on the basis of the distinguished COD fractions was far better than for the individual reactors, i.e. efficiencies (with standard deviation in brackets) for COD_{tot}, COD_{ss}, COD_{col} and COD_{sol} were respectively 68.7 (16.7), 51.2 (41.8), 62.1(38.2), and 71.8 (30.5) %.

The results also showed that the second-step UASB reactor of the two-step UASB reactor configuration had the highest COD_{sol} removal rate per litre of influent pollution load than the rest of the reactors viz. single-step of Chapter 5 and the first-step UASB reactor of the two-step system. The removal rates for COD_{sol} are 15, 21 and 23 mg/L.h respectively for the single-step (Chapter 5) and the first- & second-step of the two-step UASB reactors configuration.

The advantages of employing the two-step UASB reactor configuration include 1) the distinct higher overall removal efficiency of the anaerobic pre-treatment system 2) the higher sludge age 3) the higher reliability of the anaerobic pre-treatment process 4) the two reactors can separately be operated in case of technical problems. The reactor system can be designed with a by-pass option such that in moments of emergency each separate reactor can take extra load of the other without adverse effects and without shutting down the plant. In terms of construction it remains extraordinary simple and low cost, and extremely robust, particularly when gravity flow can be applied. It then does not require any external supply of energy. As a matter of fact it comprises a sustainable solution for sanitation in those situations were for some reason it has been decided to (mis)use clean water for conveying human residues -and wastes to some on site treatment system. The anaerobically pre-treated sewage in principle is very suited for reuse as irrigation and fertilisation, particularly in tropical countries were crops could be grown all the year. The overall assessment of the treatment unit is that the Two-Step UASB reactor pre-treatment configuration is more robust and stable, consequently more reliable for community on site wastewater pre treatment needs.

7 Final discussion

This thesis is a presentation of research findings that were conducted on constructed pilot scale UASB reactors and on an existing conventional full-scale three-compartment septic tank. Based on the results of this research it has to be concluded that the UASB reactors were capable of dealing with the high changes in flow rate as well as COD load that come with community-on-site treatment facilities. The sizes of the reactors were of small capacities that were meant to represent comparable sizes of existing onsite community wastewater treatment units. When it is assumed that the average water consumption in Dar-es-salaam is 60 l/p/d it can be calculated that the 1.5 m^3 reactor can treat the wastewater of 100 people. The most important issue when dealing with the UASB reactor is the management of the sludge bed height and the scum layer, both are discussed below.

Increase of sludge bed height in septic tank and UASB reactors
Research findings have shown that increase of sludge bed heights have got impact on the effluent quality in both types of the reactors, the septic tank and the UASB reactor (Chapters 4, 5 and 6). This means that sludge depth/accumulations in both reactors need to be regulated by removing it from the reactors, to safe levels that will not impair the effluent quality. However the reasons for regulations are different, the causes for undesirable effects should the sludge be left to stay in the reactors to undesirable levels are also different and ultimately the seriousness of the impacts of deep sludge beds in both reactors are different too. In the UASB reactor deep sludge bed reduces the flocculation distance of the settling dispersed flocculants originating either from the settling floating scum layer or caused by high evolvement of biogas from the sludge bed. HRT in the UASB reactor is also relatively reduced with the increase of the sludge bed height since some reactor space is occupied by the sludge. However impact of this relative decrease in HRT in the overall performance of the UASB reactor is somehow compensated by the very presence of a lot of sludge implying more activity in the reactor. During this period of deep sludge bed in the UASB reactor relatively high COD_{sol} removal is still realised since there is still sufficient contact between the influent dissolved and hydrolysed substrate with more active biomass provided by the deep sludge bed as the wastewater flows upwards in

the reactor. The reverse is true in the case of the septic tank reactor. In the septic tanks the HRT gets shorter with time of operation as sludge accumulates in the tank, and at the same time more settled SS are scoured out the septic tank with the effluent. The washouts particularly of COD_{sol} become more apparent too. In the UASB reactor occasional washouts of COD fractions particularly COD_{col} and COD_{sol} will be experienced especially during the peak times of biogas generation normally accompanied by high influent hydraulic loads during peak hours in the case of gravity flow mode. This implies that for the septic tank to perform better improvements need to be made in its design. However the most essential features that need to be incorporated in the septic tank in order to improve its performance will ultimately lead to features of the UASB. A significant low-cost / low-tech improvement of the septic tank may be achieved by applying modern reactor technology to the system, i.e. upward flow and gas/solids/liquid separation at the top (Zeeman, 2001). These modification will lead to a so called UASB-septic tank system (Bogte,*et al*., 1993; Zeeman, 2001). The UASB-septic tank is designed with the same long HRT typical of conventional septic tank; say about 1-2 days and long sludge retention time, ideal for single residential dwelling.

Scum layer, washouts and effluent quality in the UASB reactors
This research has demonstrated an existence of a relationship that constitutes scum layer, washouts and effluent quality in the constructed pilot scale UASB reactors (Chapters 5 & 6). In this research and from literature survey there are reported cases of scum formation, sludge washout and effluent quality deterioration due to washouts in UASB reactors treating complex wastewaters (Lettinga *et al*., 1983; Lettinga, *et al*., 1987; Haskoning, 1989; Yoda, *et al*., 1997; Halalsheh, 2002). In the UASB reactor deep sludge bed reduces the flocculation distance of the settling dispersed flocculants originating either from the settling floating scum layer or caused by high evolvement of biogas from the sludge bed and this leads to washouts of COD fractions especially COD_{ss} and COD_{col}. This implies that in order to reduce amount and frequency of washouts then sludge bed depth must be regulated to a limited depth (Lettinga, *et al*., 1987). Limitation of sludge bed depth is achieved by discharging sufficient amount of sludge from the reactor in order to allow for a certain sludge bed expansion during periods of hydraulic peak loads (Lettinga, *et al*., 1987). However in the final analysis there are other various reasons for wash-outs too which culminate to lack of optimal design, construction and dimensions of the reactor, in particular the GSS-device.

Other researchers have also experienced the scum layer formation observed in this research. Again there are various reasons for scum layer formation, its magnitude, impact and handling. As seen in Chapter 5 the scum layer formation, its magnitude and extent of impact on the effluent quality depends on the design, shape, the construction and the dimensions of the GSS-device. However there are not many reported approaches in handling of scum layer once it occurs. Halalsheh, 2002, while operating a 60-m^3 reactor in Jordan, reported that scum layer was experienced throughout the research period, however sludge discharge to the extent of keeping a clear distance of 1.0 m underneath the GSS did reduce the production of scum. In Brazil, Kato, *et al*., 2001, reports that regular cleansing and removal of grit and scum layer in the reactor, as well as regular maintenance in the sewerage system are essential for obtaining a good performance of the plants. With such regular maintenance that included scum layer removal Kato, *et al*., 2001, reports a COD removal of up to 90 percent. However removing the scum layer prematurely i.e. before being well stabilised could complicate its disposition.

In this research the scum layer was not removed neither the sludge was discharged during the research period. The highest level the sludge bed reached during the research period was 50 cm from the invert level of the reactors for the single-step, (Chapter 5) and for the first-step of the two-step UASB reactor configuration (Chapter 6). The frequency and magnitude of wash outs in this research have been found to increase during the settling of the scum layer on to the sludge bed. The washouts mainly constitute the COD_{ss} and COD_{col} fractions. The two-step UASB reactor configuration is proposed in this research to contain the washouts and polish them further thereby increasing the overall organic load removal efficiency. The second-step UASB reactor is therefore introduced for this purpose.

8 Conclusions and recommendations

Applicability of the proposed two-step UASB reactor configuration-Tanzania
Most of Tanzanians live in decentralised format. Even in cosmopolitan city like Dar es salaam still people live in clusters of high population density with typical characteristics of – low water usage, poverty is relatively high, congested areas with less space for putting up infrastructure that will demand large areas for installation.

The two-step UASB reactor configuration is easily adaptive and very suitable for community infrastructure upgrading that often takes place in Tanzanian urban and peri-urban areas. With simple construction techniques the small-scale labour-intensive contractors who usually participate in upgrading community infrastructure in urban and peri-urban areas can easily handle the construction requirements for the two-step UASB pre-treatment units. Other areas were the two-step system could immediately be employed in pre-treating the wastewaters are in the institutions like boarding secondary schools colleges, universities, military barracks, residential flats and new residential areas.

However, unlike the current practice in Tanzania, for the provision of satisfactory, low-cost wastewater pre-treatment and post treatment onsite at community level an effective management programme has to be established. A satisfactory onsite wastewater treatment system is the one that ensures public health and environmental protection as well as recovery and reuse when need be. Effective management programs for onsite system shall include the following elements (Otis, et al., 1994):

- Clear and specific performance standards,
- Technical guidelines for site evaluation, design, construction and operation
- Regular compliance monitoring
- Licensing or certification of all service providers.
- Effective enforcement mechanisms

The proposed onsite system management is a departure from the current management practice that assumes – for individual housing dwelling siting, design and construction of a septic tank system say, can be performed adequately by untrained persons and that operation and maintenance of the system can be performed by an uniformed owner (Otis, et al., 1994)

9 References

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

Haandel A. C. van and Lettinga G. (1994). *Anaerobic sewage treatment-A practical guide for regions with a hot climate*. John Wiley & Sons Ltd. England.

Halalsheh, Maha, M. (2002).Anaerobic Pre-treatment of strong sewage. A proper solution for Jordan. *PhD thesis*, Department of Environmental Technology, Agricultural University, Wageningen.

Haskoning, (1989). Royal Dutch Consulting Engineers and Architects. Anaerobic treatment of domestic wastewater under tropical conditions-final report.

Kalbermatten, J.M., Julius D. S. and Gunnerson, C. G. (1980). Appropriate technology for water supply and sanitation. A sanitation field manual. *International Bank for Reconstruction and Development/World Bank/December 1980.*

Kato, M. T. and Florencio, L. (2001). *Poster*: The use of UASB reactors in small sewage treatment plants. *Anaerobic digestion for sustainable development: Papers of the farewell seminar of Prof. Dr. ir. Gatze Lettinga, March 29-30,2001, Wageningen, The Netherlands.*

Lettinga G., de Man,A., Grin, P. and Hulshoff Pol, L. (1987). Anaerobic wastewater treatment as appropriate technology for developing countries. *Trib. Cebedeau, Nº 519, 40, pp. 21-32 Cebedoc Editeur 1987, Liège, Belgique.*

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

Lettinga G., Roersma R., and Grin R. (1983). Anaerobic treatment of raw domestic sewage at ambient temperatures using a granular bed UASB reactor. *Biotechnology and Bioengineering*; vol **25**:1701-23

Metcalf and Eddy (1991). *Wastewater Engineering:Treatment, Disposal, and Reuse.* 3rd edn. Revised by Tchobanoglous, G. and Burton, F.L., McGraw Hill, Inc.

Otis, R.J. and Anderson, D.L. (1994). Meeting public health and environmental goals: performance standards for onsite wastewater treatment systems. *Proc. of the Seventh International Symposium on Individual and Small Community Sewage Systems, 11-13 December 1994.* ASE The Society for Engineering in Agricultural, Food, and Biological Systems

Viraraghavan T. (1975). Septic tank efficiency. *Journal of Environmental Engineering Division, Proceedings of the American Society of Civil Engineers,* vol. 102, No. EE2.

Wiegant, W.M. (2001). Experiences and potentials of anaerobic wastewater treatment in tropical regions. *Wat. Sci. Tech.*, **44**(8), 107-113

Yoda, M. and Nishimura, S. 1997). Controlling granular sludge floatation in UASB reactors. . *Water Sci. Tech.* **36**(6-7), 165-173

Zeeman, G. and Sanders, W. (2001) Potential of anaerobic digestion of complex waste (water) *Water Sci. Tech.* **40**(8), 115-122

Zeeman G.and Lettinga G. (2001). Objectives of DESAR-Decentralised Sanitation and Reuse.30. Abwassertechnisches Seminar & 13. DECHEMA-Fachgesprach Umweltschutz

Samenvatting, discussie, conclusies en aanbevelingen

1 Inleiding

Wanneer het gaat om behandeling van huishoudelijk afvalwater wordt over het algemeen nog steeds verondersteld dat in de toekomst al het afvalwater zal worden afgevoerd via een riolering naar grote centrale rioolwaterzuiveringen alwaar het behandeld zal worden. Echter, deze conventionele benadering van de zuivering van huishoudelijk afvalwater is zeker voor ontwikkelingslanden te duur en te complex en daarom geen duurzame oplossing (Zeeman et al. 2001). Terwijl de civiel ingenieurs ten onrechte nog steeds de conventionele centrale afvalwaterzuiveringconcepten voor ogen hebben (Otis et al. 1994) wordt in ontwikkelingslanden veelal sterk decentrale voorzuivering van huishoudelijk afvalwater toegepast. Hierbij gaat het voornamelijk om laag belaste anaërobe systemen bij individuele huizen, zoals pit latrines en septic tanks, die niet of nauwelijks worden onderhouden, met als gevolg dat zowel het milieu als de volksgezondheid schade oploopt. Een voorbeeld hiervan is Tanzania waar de pit latrines en septic tanks in de stad en buitenwijken door hun enorme aantal niet meer beheers- en controleerbaar zijn en een steeds grotere bedreiging voor het milieu vormen. Door deze laag belaste systemen bij individuele huizen te vervangen door effectievere anaërobe voorzuiveringssystemen per gemeenschap zou het aantal zuiveringssystemen terug worden gebracht naar een beheers- en controleerbaar niveau wat het milieu en volksgezondheid ten goede zou kunnen komen.
Echter tot op heden is er door civiel ingenieurs en aanverwante beroepsgroepen weinig aandacht besteedt aan zuiveringsystemen voor individuele huizen of gemeenschappen met als gevolg dat men zich nog niet heeft aangepast heeft aan de veranderde behoeften voor deze systemen naar een betere prestatie, terugwinning van hulpbronnen en bescherming van het milieu. Hierdoor is nog onvoldoende onderzoek gedaan naar doeltreffende, compacte en rendabele decentrale zuiveringsystemen. Met onderzoek op het gebied van schaalverkleining van zuiveringsystemen waarvan de doeltreffendheid al is bewezen zouden richtlijnen kunnen worden opgesteld voor de organische en hydraulische belasting van verschillende systemen voor de praktijk. Daarnaast is ook te weinig aandacht besteed aan de verspreiding van kennis en ontwikkelingen in de anaërobe zuiveringstechnologie onder mensen in de praktijk. Dit laatste is een van de grootste hindernissen voor de praktijktoepassing van compacte anaërobe zuiveringtechnologieën (Lettinga, 2001).

Op basis van de beschikbare kennis over de toepassing en doeltreffendheid van de Upflow Anaerobic Sludge Bed (UASB) reactor kan worden geconcludeerd dat het toepassen van de UASB reactor in een systeem voor de zuivering van huishoudelijk afvalwater op lokaalniveau zal leiden tot een goedkoop, simpel, compact en doeltreffend zuiveringsysteem voor tropische condities, zoals bijvoorbeeld in Tanzania. Echter, door het gebrek aan kennis op het gebied van het toepassen van dit soort systemen op lokale schaal is het niet mogelijk om de technologie direct in de praktijk toe te passen op basis van resultaten uit de literatuur. Daarom is het voor het ontwerp van een geoptimaliseerd en doeltreffend systeem nodig om extra gegevens te verzamelen in een proefreactor.

In dit proefschrift worden de gegevens gepresenteerd die werden verzameld tijdens onderzoek naar de toepassing van verschillende UASB reactorsystemen voor de behandeling van huishoudelijk afvalwater van de campus van het University College of Lands and Architectural Studies (UCLAS) in Dar-es-salaam, Tanzania. Alle reactoren werden parallel aan elkaar bedreven onder sterk wisselende hydraulische en organische belastingen bij een omgevingstemperatuur van 25 tot 34°C.

2. Afvalwaterdebiet en -samenstelling

De belangrijkste componenten in afvalwater dat op lokaal niveau wordt verzameld en behandeld zijn afkomstig van huishoudens. De hoeveelheid en de vervuilingsgraad van huishoudelijk afvalwater zijn geheel afhankelijk van de grote en het sociaal-economisch gedrag van de gemeenschap. In het geval het van huishoudelijk afvalwater woningen op de campus van de University College of Lands and Architectural Studies (UCLAS) in Dar-es-salaam, Tanzania bleek de vervuilingsgraad over de dag sterk te variëren. Gemiddeld was de vervuilingsgraad 564 mg CZV/l echter met een standaarddeviatie van 230 mg CZV/l. De CZV: BZV_5 verhouding van het afvalwater was 1.52±0.13. Ter vergelijking werd ook het afvalwater dat behandeld werd in de centrale rioolwaterzuivering van Dar-es-salaam gekarakteriseerd, hier was de vervuilingsgraad 470 mg CZV/l met een standaarddeviatie van slechts 98 mg CZV/l. De bovenstaande resultaten zijn consistent met wat er in de literatuur gepubliceerd is: afvalwater van kleine afvalwaterzuivering kent een veel grotere variatie in zowel debiet als vervuilingsgraad dan afvalwater voor een grote zuivering (Metcalf & Eddy, 1991). Om een goede zuiveringsgraad te garanderen in kleine zuiveringsinstallaties moet sterk rekening worden gehouden met deze grote variaties in debiet en vervuilingsgraad. Het afvalwater van UCLAS bevat een hoog percentage aan gesuspendeerd materiaal, circa 50% van het CZV.

In hoogbelaste anaërobe reactoren zoals de UASB reactor vind in vergelijking met bijvoorbeeld de clarigesters een betere verwijdering van BZV en CZV plaats, terwijl de verwijdering van gesuspendeerd materiaal vrijwel gelijk blijft (Wiegant, 2001). Hoewel voor UASB reactoren piekbelastingen geen probleem zijn, is het toepassen van een nabehandelingstap voor het effluent bij opstroomsnelheden boven de 1.0 tot 1.5 m/uur te rechtvaardigen (Wiegant 2001).

3. Interpretatie van anaërobe slibstabilisatieproeven

In alle afvalwaterzuiveringsprocessen wordt slib geproduceerd. De samenstelling, droge stofgehalte en stabiliteit van dit slib wordt bepaald door de samenstelling van het ruwe afvalwater en het zuiveringsproces. De slibproductie tijdens de anaërobe zuivering van rioolwater is slechts 25-40% van de hoeveelheid die wordt geproduceerd tijdens de aërobe zuivering van dat zelfde rioolwater (Lettinga, 1983). In elk zuiveringsproces moet op een gegeven moment slib gespuid worden. In de meeste gevallen wordt het aanbevolen om uitsluitend stabiel slib te spuien, dwz slib met een lage hoeveelheid biodegradeerbaar materiaal (Haandel & Lettinga, 1994). Uit de literatuur blijkt dat er verschillende methoden worden gebruikt om de stabiliteit van slib vast te stellen, waardoor de vergelijkbaarheid van de resultaten uit de verschillende tests te wensen over laat. In hoofdstuk 3 van dit proefschrift wordt daarom een methode voor de interpretatie van anaërobe slibstabilisatieproeven voorgedragen. In deze methode wordt gebruikgemaakt van een vereenvoudigd model van de biologische processen tijdens een slibstabilisatieproef en de mogelijkheid om de parameters van het model te schatten met behulp van een computersimulatiepakket. Met behulp van de voorgedragen methode kan het percentage biomassa, biodegradeerbare componenten en inert organisch materiaal van het slib worden geschat. De methode werd geïllustreerd met behulp van stabiliteitproeven van slib uit een septic tank, een UASB reactor en een pit latrine. De resultaten van de proeven lieten zien dat het CZV van het slib van de UASB en septic tank uit respectievelijk 21 en 18% biomassa bestond. In beide slibsoorten was de hoeveelheid biodegradeerbare componenten nihil, zodat geconcludeerd kon worden dat zowel het slib uit de UASB reactor als uit de septic tank volledig stabiel was. Het slib uit de pit latrine bevatte vrijwel geen biomassa. De hoeveelheid degradeerbare componenten in het slib bedroeg 17% van het CZV in het slib.

4. Prestatie van een septic tank met drie compartimenten op praktijkschaal voor de lokale behandeling van afvalwater

De septic tank is de meeste toegepaste decentrale zuiveringsmethode voor de anaerobe voorzuivering van afvalwater (Zeeman et al. 2001). In Tanzania worden septic tanks voornamelijk toepast voor voorzuivering van afvalwater op lokaalniveau. De behandeling van afvalwater voor individuele huizen vindt voornamelijk plaats in pit latrines.

In hoofdstuk 4 worden de resultaten weergegeven van een 400 dagen durend meetprogramma van een praktijk septic tank voor de locale behandeling van huishoudelijk afvalwater van het University College of Lands and Architectural Studies (UCLAS) in Dar-es-salaam, Tanzania. De septic tank is gebouwd in 1982 en heeft een effectief volume van 56.3 m^3 wat verdeeld is over 3 compartimenten. Al het afvalwater wordt in het eerste compartiment ingevoerd waarna het achtereenvolgens naar het tweede en derde compartiment stroomt. Uit de resultaten in hoofdstuk 4 blijkt de septic tank in vergelijking met resultaten uit de literatuur slecht te functioneren. De verwijderingspercentages voor total CZV, gesuspendeerd CZV, colloidaal CZV en opgelost CZV zijn respectievelijk 29 (±26), 29(±59), 16(±21) en 16(±21) %. The gemiddelde theoretische hydraulische verblijftijd in de septic tank gedurende de meetperiode was 5.6 (±8) dagen. De slechte prestaties van de septic tank werden toegeschreven aan slechte management van de betreffende septic tank en horizontale doorstroompatroon van septic tanks in het algemeen.

Daarnaast werd geconcludeerd dat de hoge hydraulische verblijftijd in septic tanks leidt tot relatief grote reactoren en daardoor ook hoge kosten voor de bouw van de tanks. Dit beperkt de toepassing van septic tanks in de arme en drukbevolkte stads- en buitenwijken van Dar-es-salaam. De prestaties van bestaande septic tanks zouden wel kunnen worden verbeterd door aanpassing te doen die de UASB reactor kenmerken met name de influentinvoer. Door het influent onderin in de reactor in te voeren ontstaat er een opstroom in de septic tank waardoor het effectieve volume van de reactor beter benut wordt en bovendien er een beter contact ontstaat tussen micro-organismen en afvalwater. Het tevens aanbrengen van een drie fasenscheider voor scheiding van gas, vaste stof en vloeistof in het effluent van de reactor zal leiden tot een betere verwijdering van gesuspendeerd materiaal (Bogte et al. 1993, Zeeman et al, 2001).

5 Prestaties van een één-traps UASB proefreactor voor de lokale behandeling van tropisch huishoudelijk afvalwater

Hoodfstuk 5 geeft een samenvattig weer van de prestaties van een één-traps UASB proefreactor voor de lokale behandeling van tropisch huishoudelijk afvalwater over een periode van 3½ jaar. Het afvalwater was afkomstig van het cafetaria, sommige toiletten in studentenflats, en enkele huizen op de campus van het University College of Lands and Architectural Studies (UCLAS) in Dar-es-salaam, Tanzania. De UASB reactor had een volume van 1,5 m^3 en een hoogte van 1.7 m, het afvalwater werd onder vrij verval in de reactor geleid hierdoor varieerde de hydraulische verblijftijd van de reactor tussen de 1,7 tot 40 uur met een gemiddelde van 6 uur. De gemiddelde vervuilingsgraad van het afvalwater was 529 mg CZV/l met minimum- en maximumwaarden van respectievelijk 46,4 en 4109 mg CZV/l. Uit het verloop van de resultaten van de reactor kon worden opgemaakt dat de verwijdering van totaal CZV in de loop van de onderzoeksperiode verslechterde. Dit werd toegeschreven aan toename van de gasproductiesnelheid, de groei van het slibbed en de dynamiek van de drijflaag. Hierdoor en door een niet optimaal ontwerp van de drie-fasen scheider konden gedispergeerde deeltjes uitspoelen. De gemiddelde verwijdering van totaal CZV over de gehele periode bedroeg 64%. Een betere verwijdering zou bereikt kunnen worden bij behandeling van het afvalwater in een twee-traps UASB systeem waarbij het effluent van de eerste trap in de tweede trap wordt nabehandeld. Een de resultaten van een dergelijk systeem worden behandeld in hoofdstuk 6.

6 Prestaties van een doeltreffende compacte twee-traps UASB proefreactor voor de lokale voorbehandeling van tropisch huishoudelijk afvalwater

In hoofdstuk 6 worden de resultaten beschreven van een twee-traps UASB proefreactor voor de lokale behandeling van afvalwater afkomstig van het cafetaria, sommige toiletten in studentenflats, en enkele huizen op de campus van het University College of Lands and Architectural Studies (UCLAS) in Dar-es-salaam, Tanzania. De proefreactor bestond uit twee UASB reactoren in serie. De eerste reactor had een volume van 1,8m^3 en een hoogte van 2m en de tweede reactor had een volume van 0,85m^3 en een hoogte van 1m. Al het effluent van de eerste reactor werd gebruikt als influent voor de tweede reactor. Het twee-traps reactor systeem werd parallel bedreven aan het één-traps systeem beschreven in hoofdstuk 5. Het twee-traps systeem werd op dag 665 van de onderzoeksperiode met het één-traps systeem opgestart en vervolgens voor een periode van 630 dagen gevolgd. De reactor werd zonder entslib opgestart en de toegepaste hydraulische verblijftijd in het reactorsysteem bedroeg 7,4 uur (5+2,4 uur). De maximale specifieke methanogene activiteit van het slib in de eerste trap lag tussen de 0,016 en 0,135 g CZV/g VSS.dag. De maximale specifieke methanogene activiteit van de tweede trap lag hoger, namelijk 0,104 tot 0,272 g CZV/g VSS.dag. Het onderzoek met de één-traps reactor (hoofdstuk 5) had aangetoond dat de lage verwijdering van gesuspendeerd CZV veroorzaakt werd door de hoge gasproductiesnelheid en de dynamiek van de drijflaag in de reactor. In een twee-traps systeem zou de verwijdering van gesuspendeerd CZV hoger moeten zijn doordat in de tweede trap het uitgespoelde slib uit de eerste trap word opgevangen en verder gestabiliseerd. De gemiddelde waarden voor de totaal CZV, gesuspendeerd CZV, colloïdaal CZV en opgelost CZV van het influent van het twee-traps systeem bedroegen respectievelijk 537.2 (±165.3), 189.9 (±109.5), 127.4 (±75), 223.4(±108.8). De gemiddelde verwijdering van de verschillende CZV fracties bedroegen 68.7(±16.7), 51.2(±41.8), 62.1(±38.2) en 71.8 (±30.5) procent voor respectievelijk het totaal CZV, gesuspendeerd CZV, colloïdaal CZV en opgelost CZV. De omzettingssnelheid van opgelost CZV in de reactoren was 15 mg CZV/l.uur voor de één-traps reactor en 21 en 23 mg CZV/l.uur voor respectievelijk de eerste en tweede trap van het twee-traps systeem.

De voordelen van toepassing van een twee-traps UASB systeem zijn 1) een verbeterde CZV verwijdering ten opzichte van septic tank systemen 2) Een hogere slibleeftijd 3) Een hogere betrouwbaarheid van het systeem 4) De twee reactoren kunnen in het geval van technische problemen afzonderlijk bedreven worden. Het reactorsysteem kan hiervoor uitgerust worden met een bypass leiding zodat in noodgevallen het influent direct naar de tweede trap kan worden geleid.
Met betrekking tot de bouw van een dergelijk systeem kan worden gezegd dat het een zeer eenvoudig, goedkoop en robuust systeem is. Dat geld zeker wanneer het een systeem betreft waar het afvalwater onder vrij verval in wordt gebracht, omdat er dan ook geen pompen nodig zijn en de energiebehoefte nul is. Goed bezien biedt het twee-traps UASB systeem een duurzame oplossing voor die situaties waarin, om welke redenen dan ook, gekozen is voor het misbruiken van schoon water voor het transporten van fecaliën naar een lokaal zuiveringsysteem. Anaëroob gezuiverd afvalwater is bijzonder geschikt voor hergebruik als irrigatiewater en bemestingsstof in de landbouw, zeker in tropische gebieden waar het groeiseizoen het gehele jaar voortduurt.

7. Algemene discussie

In dit proefschrift worden de resultaten gepresenteerd van onderzoek in proefreactoren naar de toepassing van UASB reactoren voor de lokale behandeling van tropisch huishoudelijk afvalwater. Daarnaast zijn de prestaties van een bestaande full-scale septic tank voor de lokale behandeling van dat zelfde afvalwater bekeken. Op basis van de resultaten kan worden geconcludeerd dat de UASB reactor systemen geschikt zijn om de grote variaties in afvalwater samenstelling en debiet zoals ze op lokaal niveau voorkomen op te vangen. Het volume van de gebruikte UASB proefreactoren (1.5m^3 en 1.8m^3) kwam overeen met de het volume dat in de praktijk toegepast zou kunnen worden. Wanneer wordt aangenomen dat het gemiddelde waterverbruik in Dar-es-Salaam op 60 l/p/dag ligt kan worden berekend dat de 1.5m^3 proefreactor geschikt was om het afvalwater van ongeveer 100 mensen te behandelen. De belangrijkste punten met betrekking tot het management van een dergelijke UASB reactor zijn de hoogte van het slibbed en de drijflaag. Beide punten zullen hieronder worden besproken.

Toename van de slibbedhoogte in septic tanks en UASB reactoren
De resultaten van het onderzoek laten zien dat zowel in septic tanks als in UASB reactoren de toename van de hoogte van het slibbed grote invloed heeft op de kwaliteit van het effluent (Hoofdstuk 4, 5, 6). Dit betekent dat de diepte van het slibbed en accumulatie van slib in beide type reactoren gereguleerd moeten worden door spuien van slib. Echter de redenen waarom de slibbed hoogte gereguleerd moet worden zijn verschillend. In de UASB reactor betekend een hoog slibbed dat de gedispergeerde stoffen die afkomstig zijn uit de drijflaag of door de gasproductie omhoog worden gestuwd te weinig ruimte hebben om uit te vlokken en te bezinken waardoor de uitspoeling van gesuspendeerd CZV toe zal nemen. Daarnaast heeft het

slib in de reactor ook volume waardoor de relatieve hydraulische verblijftijd daalt. Doordat de reactor vanonder in het slibbed wordt gevoed vindt er ook gedurende de periode dat het slibbed in de reactor heel hoog is een goede menging plaats tussen afvalwater en biomassa. Hierdoor wordt, voor wat het opgeloste CZV betreft, de lagere hydraulische verblijftijd in de reactor over het algemeen gecompenseerd doordat meer slib meestal ook meer biologische activiteit inhoudt. Voor septic tanks geld in dit laatste geval juist het omgekeerde omdat het influent van bovenaf in de reactor wordt ingevoerd en waardoor er weinig menging van biomassa en afvalwater plaatsvindt. Hierdoor wordt bij een hoog slibbed in de reactor de verwijdering van opgelost CZV lager. Daarnaast zal door afschuiving van het slibbed ook meer uitspoeling van gesuspendeerd CZV plaatsvinden. Het aanpassen van het invoer van het influent in de septic tank en toevoegen van een drie-fasenscheider zal de kwaliteit van het effluent sterk verbeteren en leiden tot een zogenoemde UASB-septic tank reactor (Bogte et al., 1993, Zeeman et al. 2001).

Drijflaag, slibuitspoeling en effluentkwaliteit van de UASB reactoren
In dit onderzoek is gebleken dat er een relatie bestaat tussen het bestaan van een drijflaag, slibuitspoeling en de effluentkwaliteit in de UASB proefreactoren (Hoofdstuk 5&6). Ook in ander onderzoek wordt dit fenomeen gerapporteerd (Lettinga et al. 1983; Lettinga et al, 1987; Haskoning 1989; Yoda et al. 1997; Halalsheh, 2002). Om een goede effluentkwaliteit te waarborgen moet een slibbed beneden een maximale hoogte worden gehouden door het regelmatig spuien van slib. Hierbij moet voldoende ruimte worden gecreëerd voor expansie van het slibbed tijdens piekbelasting (Lettinga et al., 1987). Tevens zou een optimalisatie van de dimensies en constructie van de reactor en de drie-fasenscheider bij de gebruikte proefreactoren de effluentkwaliteit kunnen verbeteren. Het ontstaan en de dikte ontstane drijflaag hangt samen met het ontwerp van de drie-fasenscheider. In de literatuur is hier nog weinig over bekend. Halalsheh (2002) rapporteerde vorming van een drijflaag in een $60m^3$ reactor. Echter, het spuien van slib tot een slibbedhoogte tot 1.0 meter onder de drie-fasenscheider verbeterde de effluentkwaliteit niet. Kato et al. (2001) rapporteerde dat het regelmatig verwijderen van de drijflaag en zand uit de reactor en een goed onderhoud van het riool essentieel is voor een bereiken van een goede effluentkwaliteit. Kato et al. (2001) rapporteerde een tot 90% CZV verwijdering bij regelmatig verwijderen van de drijflaag. Echter het slib van deze drijflaag is niet gestabiliseerd en zal dus nog nabehandeld moeten worden.
In het hier gepresenteerde onderzoek werd geen slib gespuid. In het één-traps systeem (Hoofdstuk 5) en de eerste trap van het twee-traps systeem werd een maximale slibbedhoogte van 50 cm bereikt. De meeste uitspoeling van gesuspendeerd en kolloidaal materiaal uit de reactor vond plaats in de periode waarin de drijflaag begon te bezinken. Het toepassen van een tweede UASB achter de UASB voor het opvangen en omzetten van het uitgespoelde materiaal wordt daarom aanbevolen.

8 Conclusies en aanbevelingen

Toepassing van de twee-traps UASB reactor configuratie in Tanzania
De meeste inwoners van Tanzania wonen in kleine gedecentraliseerde gemeenschappen, zelfs in grote steden zoals Dar-es-Salaam. Deze gemeenschappen kunnen worden gekenmerkt door een zeer lage levensstandaard een laag

waterverbruik en een hoge bevolkingsdichtheid, waardoor er weinig plaats is voor infrastructuur.

In de steden van Tanzania wordt regelmatig gewerkt aan verbetering van de infrastructuur in bestaande woongemeenschappen en het twee-traps UASB systeem is zeer geschikt voor toepassing in deze situaties. De bouwwerkzaamheden tijdens de verbeteringen aan de infrastructuur worden over het algemeen uitgevoerd door kleine lokale aannemers. Het twee-trapssysteem is eenvoudig en kan ook door deze lokale ondernemingen worden uitgevoerd. Ook in andere woon en werkomgevingen zoals kostscholen, middelbare scholen, universiteiten, kazernes, flatgebouwen en nieuwbouwwijken kan het twee-trapsysteem eenvoudig worden toegepast.

Echter, in tegenstelling tot de bestaande situatie in Tanzania, is het nodig om een doeltreffend controle- en beheersprogramma op te zetten zodat een goede voorzuivering op lokaal niveau te kunnen waarborgen. Een doeltreffend controle- en beheersprogramma voor lokale behandeling van afvalwater moet de volgende elementen bevatten (Otis et al. 1994):

- Duidelijke en specifieke effluentnormen
- Richtlijnen voor technische evaluatie, constructie en bedrijf van het systeem
- Regelmatige controle
- Verstrekking van vergunningen of certificatie van alle betrokken dienstverleners
- Doeltreffende mechanismen voor handhaving van de richtlijnen

Het voorgestelde controle- en beheersprogramma wijkt af van de huidige praktijk waarbij wordt aangenomen dat voor individuele huizen het ontwerp en constructie kan worden gedaan door ongeschoolde arbeidskrachten. Bovendien wordt in de huidige praktijk het bedrijf en onderhoud van het systeem overgelaten aan de huiseigenaren (Otis et al. 1994).

Curriculum vitae

The author of this dissertation Shaaban Mrisho Mgana was born in November 1953 in Morogoro, Tanzania. In 1979 he obtained his Bachelor of Engineering degree in Civil Engineering from the University of Roorkee, India. In 1983 he was conferred a degree of Master of Science in Civil Engineering from the Pittsburgh University, USA. He worked as an engineer in the Ministry of Water in Tanzania from 1979 to 1985 and thereafter he became part of the academic staff of the University College of Lands and Architectural Studies (UCLAS – formerly Ardhi Institute), Tanzania. Provisionally in 1993-94 he started a PhD "sandwich construction" programme by attending an international course on anaerobic wastewater treatment and thereafter followed by literature review on the research area and anaerobic laboratory experimental set-ups and analyses training at the Sub-Department of Environmental Technology of the Wageningen University (WU), in Wageningen, The Netherlands. The actual PhD research took off in early 1999 at his home institute, Ardhi Institute/UCLAS, Tanzania whereby the period from 1994 to 1999 were used for mobilisation of research funds, acquiring of limited research equipment and construction of pilot UASB reactors for research activities at his home institute.

His address in Tanzania:

Mgana S.
Department of Environmental Engineering,
UCLAS,
P.O.Box 35176
Dar es salaam,
Tanzania.

E-mail: mgana@uclas.ac.tz
Or smgana@hotmail.com

Propositions

1) Sludge floatation and the wash-out or retention of dispersed flocculent sludge present above the sludge bed (i.e. present in the “sludge blanket”) of the UASB reactor originates from both inadequate reactor design and construction, as well as from mechanical phenomena prevailing in the system.

-This thesis

2) Sudden increase and decrease of TS concentration in the sludge bed of a UASB wastewater treatment unit is a clear manifestation of sludge floatation phenomenon

-This dissertation

3) In developing countries access to basic sanitation by the majority of the population could best be achieved through decentralised/community sanitation approach with its much lower burden of debt, compared to centralised sanitation approaches with its deep burden of debt for the people, while generally merely the prosperous section of the population will be the beneficiaries.

-This thesis

4) Sustainable wastewater treatment is the most effective approach of reclaiming freshwater that is being increasingly depleted by the increasing generation of wastewater as a result of growing population and anthropogenic activities.

5) Science and technology should enhance the implementation of anaerobic and facultative decomposition processes in favour of living human civilization endeavour, since they are potent natural decaying processes that every organic matter finally succumbs to.

6) Future competitive wastewater treatment plants will be raw materials production plants for products such as organic sulphur, ammonia, nitrogen, biogas etc and the emphasis for standards will shift from quality of effluent and receiving waters to the standards of recovered raw materials and their reuse.

7) Excellent high rate anaerobic performance for wastewater treatment starts with the employment of UASB reactor(s) and step-wise ends up with EGSB reactor.

8) Since being rich is a human natural desire, elimination of poverty in the world is a ‘utopia paradigm’ with emphasis on wishful thinking.