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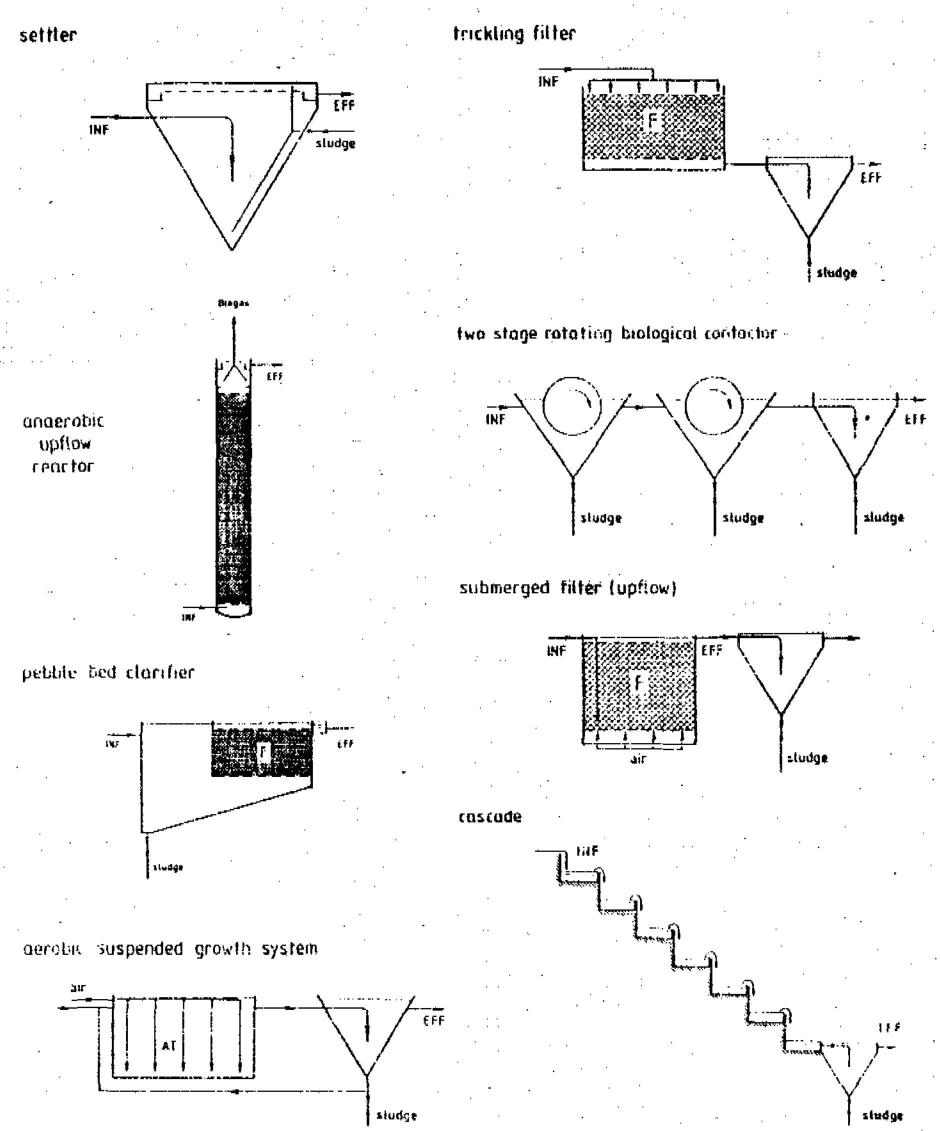

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Ir. J.C.L. van Buuren

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DEPARTMENT OF ENVIRONMENTAL TECHNOLOGY
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POSTTREATMENT METHODS FOR EFFLUENT OF UASB REACTORS TREATING DOMESTIC WASTEWATER

Joost C. L. van Buuren
Dept. of Environmental Technology
Agricultural University Wageningen
Postbox 8129
6700 EV Wageningen
The Netherlands

Summary

A survey of the research carried out at the Department of Environmental Technology of the Agricultural University Wageningen is reported. Emphasis is placed on the posttreatment of domestic wastewater pretreated in UASB-reactors. A synopsis of 22 quality parameters of UASB-effluent is presented.
Batch treatment of UASB-effluent under diverse conditions was studied. Under aerobic conditions it showed first order degradation rate constants for filtered COD and faecal coliforms of -0.43 and -1.30 day^{-1} respectively.
For continuous wastewater treatment the following posttreatment methods were studied: sedimentation, anaerobic upflow filtration, aerobic suspended growth, attached growth systems and the cascade. Among the attached growth systems attention is paid to the high and low-rate trickling filter, biorotor and submerged aerated filter. As for the wastewater constituents to be removed our research was focused on suspended solids, organic matter (COD), ammonia, phosphorus and fecal coliforms. Sedimentation and anaerobic filtration are successful in removing suspended solids, but less so with regard to other parameters. A UASB-reactor (HRT 6 h) followed by a suspended growth activated sludge system (HRT of 3 to 5 h) achieves 90% COD_{tot} removal and 50 - 80% nitrification. Nitrification efficiency is high up to sludge loading rates of approximately 0.3 kg COD/kg sludge.d.
A UASB reactor(HRT 6h) plus a two stage rotating biological contactor (HRT 3.5 h) provides 95% COD_{tot} removal and 70-90% nitrification. Faecal coliform removal in this system seems less than in suspended growth activated sludge installations. Submerged filters show promising faecal coliform removal efficiencies. Trickling filters and cascades show relatively poor performance with respect to all parameters.
With regard to sewage treatment in urban and mountainous areas in tropical developing countries the UASB pretreatment reactor followed by small lagoons or rotating biological contactors seem to be the most promising methods.

Contents

1. Introduction

In the 80s the anaerobic treatment of sewage and industrial wastewater became a grown-up technology. Especially in tropical countries anaerobic treatment of domestic wastewater appeared to be an appropriate alternative to conventional aerobic methods.
Since 1983 a $64m^3$ UASB pilot plant for sewage was operated in Cali Colombia (Louwe Kooymans and Van Velsen, 1986). Full-scale plants are functioning or under construction now in many countries, especially in urban surroundings where flat land is scarce. Efficient removal of organic matter, low excess sludge production, low energy input (for pumping needs only), and relatively low investment and maintenance costs are assets of UASB treatment stations. On the other hand the effluent not always complies with the local standards and requirements. Anaerobic treatment plants do not remove nitrogen and phosporous and pathogenic enteric organisms are eliminated up to only 80%.

The quality ranges of effluents from UASB-reactors treating domestic wastewater are given in table 1.

Parameter		Range	Typical
pH		6.5-7.6	7.4
EP	mV	-200- -350	-300
EC	mmho	0.4 - 2	1.0
DO	$mg.l^{-1}$	0 - 1.0	0.5
COD_{tot}	$mg.l^{-1}$	100 - 650	200
COD_{fil}	$mg.l^{-1}$	80 - 300	150
COD_{diss}	$mg.l^{-1}$	60 - 300	130
BOD_{tot}	$mg.l^{-1}$	50 - 220	80
BOD_{diss}	$mg.l^{-1}$	40 - 120	60
TOC_{fil}	$mg.l^{-1}$	35 - 100	50
TSS	$mg.l^{-1}$	20 - 350	60
VFA	$mg.l^{-1}$		5
N_{kj}	$mg.l^{-1}$	25 - 90	65
NH_4-N	$mg.l^{-1}$	25 - 90	60
NO_2-N	$mg.l^{-1}$	0	0
NO_3-N	$mg.l^{-1}$	0	0
PO_4-P	$mg.l^{-1}$	5 - 30	10
S_{tot}	$mg.l^{-1}$	5 - 25	12
Faecal E.Coli	n.100 ml^{-1}	$5*10^5$ - $2*10^7$	$5*10^6$
Faecal streptococci	n.100 ml^{-1}	$2*10^5$ - $4*10^6$	$6*10^5$
Clostridia-spores	n.100 ml^{-1}	$1*10^3$ - $9*10^4$	$4*10^4$
Total Plate Count	n.100 ml^{-1}	10^8 - 10^9	-

Table 1: Some quality parameters of UASB-effluent (this work, DeFraiture (1978), Van Vree (1987), Luyten(1985), Haas and de Gooyer (1984).
* EP= electrode potential measured with platinum indicator and calomel reference electrode. During incidental sludge-washout from the anaerobic reactor high concentrations of organic matter are found.

With regard to several destinations of the UASB-effluent some further improvement of the quality by posttreatment will be required. The main posttreatment processes are suspended solids removal, nitrification, sulphide oxydation and pathogen removal. As sewage effluents in developing countries are often reused in a planned or unplanned way in the household, agriculture and aquaculture the main emphasis should be laid on hygienic quality.
In principle secondary treatment of UASB-effluent can be effectuated by many known wastewater treatment methods also applied for settled sewage. We may roughly distinguish between capital- and maintenance-extensive methods such as stabilization ponds and intensive methods like aerated activated sludge installations, trickling filters etc.. If sufficient flat land is available stabilization ponds are hard to beat as an appropriate posttreatmemnt system in

tropical countries. Stabilization ponds are cheap, easy to maintain and offer excellent treatment efficiencies at hydraulic retention times of 10 - 25 days.
The application of intensive methods comes into consideration whenever flat land is scarce or the climate is less favourable to pond systems. This paper reviews the most well-known mechanical and biological methods.
Treatment efficiencies are correlated with hydraulic retention time and organic loading rate thus expressing a first approximation of the relationship between treatment efficiency and construction cost.

Methods

Most experiments were conducted in continuous flow pilot- or bench-scale installations fed with UASB-pretreated domestic wastewater. SS-removal is calculated on the basis of determination of TSS- or COD_{ss}-concentrations, COD_{tot}-removal on the basis of influent- and effluent COD_{unf},
COD_{fil}-removal on the basis of influent- and effluent COD_{fil}.
Generally, the analyses were performed on composite samples.

Analytical methods

TSS	Total suspended solids (Dutch Standards No. 6621)
BOD_{tot}	BOD determination of unfiltered sample according to Dutch Standard No.
BOD_{diss}	BOD determination after 0.45 μ filtration
COD_{unf}	COD determination on unfiltered sample with micro COD method
COD_{fil}	COD determination after filtration with 7 μ or 1.5 μ filter
COD_{diss}	COD determination after 0.45 μ filtration
TOC_{fil}	TOC determination after filtration with 7 μ filter according to NPR 6522
N_{kj}	Kjeldahl nitrogen (NEN 6641)
NH_4-N	Ammonia nitrogen (NEN 6472)
FC	Faecal coliform determination in unfiltered samples using (Dutch Standards No. 6261)

Table 2. List of analytical methods used in the Department of Environmental Technology.

2. Preliminary batch experiments on the treatability of UASB-effluent

2.1. Experiments

De Fraiture (1978) studied the character of the suspended solids in UASB-effluent. The effluent he used had passed a spongy material which was kept in the top of the reactor. The sponge filtration of fresh effluent decreased the COD-value from 358 $mg.l^{-1}$ to 323 $mg.l^{-1}$: a reduction of 10%. It turned out that 10 - 15% of the remaining COD_{tot} could be centrifuged off in 15 min. at 5,000 g. Increase of the g-value up to 28,000 g did not result in a higher COD-removal. 0.45 μ filtration after centrifugation made COD decrease from 286 to 268 mg/l: 6% removal. From these experiments it can be derived that 20 - 25% of the COD_{tot} in fresh effluent can be called 'suspended', about 5% is colloidal.

Centrifugation of the suspended solids was also performed on UASB-effluent kept for 14 days at 4° C under air. It turned out that at 3,000 g appr. 30% of the COD_{tot} and at 30,000 g 40% of the COD_{tot} could be centrifuged off in 15 min.
The preservation of the UASB-effluent samples made it easier to separate the COD_{ss} present. This COD may partly consist of newly formed bacterial sludge particles, though at 4° C no quick sludge growth is expected. Also a coarsening of fine suspended particles takes place.

Luyten (1985) determined the particle size distribution of UASB-effluent by means of microscopy.

The following results were found:

Particle Diameter (μ)	Fraction (%)
0-4	79
4-10	15
10-20	3.7
20-40	1.2
40-100	0.5

Table 3. particle size distribution in fresh UASB-effluent.

Further the particle size distribution as a function of time was assessed in samples which were kept at 20°, 4°, and at 20° C in the presence of 40 $mg.l^{-1}$ $HgCl_2$. $HgCl_2$ was added to suppress biological activity.
A flocculation of the fine particles was observed in each of the three cases. It was slower at 4° than at 20 ° C. Flocculation also took place in the presence of $HgCl_2$, which indicated that flocculation is independent of of biological activity. Although the flocculation is clearly visible the flocs represent only a minor part of the total TOC present.

De Fraiture (1978) and Lanting (1980) did batch experiments in which they investigated COD and TOC removal from UASB-effluent under various conditions.

De Fraiture (1978) applied the following procedure. 3 litres of freshly sampled and sponge filtered UASB-effluent were put in a slowly stirred vessel. The effect of evaporation during the experiment was eliminated by making up with demineralized water. The temperature was 21° C. The reaction time was approximately 200 hrs.

The results are summarized in table 4. and figure 1. and 2.

Exp. Nr.	Ambient atmosphere	Added substances	COD_{fil}-rem.	TOC_{fil}-rem.
1.	air	none	66	69
2.	nitrogen	none	10	?
3.	air contact, 4° C	none	42	43
4.	nitrogen	nitrate	36	37
5.	nitrogen	sulphate	25	26
6.	nitrogen	den. sl.	28	21
7.	nitrogen	nitr. + den. sl.	49	50
8.	nitrogen	sulph.+ sr.sl.	21	24
9.	nitrogen	$Fe(II)SO_4.7H_2O$	35	56
10.	air	compressed air	79	77

Table. 4. Experimental conditions and removal of COD_{fil} and TOC_{fil} after 200 hrs in batch treatment of UASB-effluent (De Fraiture, 1978)
den. sl. : addition of denitrifying sludge;
sr.sl. : addition of sulfate reducing sludge.

If the UASB-effluent is aerated with compressed air (10) the removal rate in the first day is increased (50% COD_{fil}-removal in 1 day) as compared to the vessels without forced aeration. This will partly be brought about by stripping of volatile substances like fatty acids, hydrogensulfide, etc.. After prolonged reaction times not much difference between COD removal percentages is observed. The final removal after 14 days is about 70% in both the stirred and the aerated vessels: final COD_{fil} = 75 mg/l.
During the exposure to air floc formation is clearly visible.

Fresh UASB-effluent sampled without air contact contains 6 - 10 mg/l DCOD (directly chemically oxydizable compounds) as determined by iodometric titration. The DCOD is oxydized within the first minutes the effluent is exposed to air. This DCOD consists of ferrosalts , sulfides, sulfites, and aldehydes.

If the vessel had not any air contact (2) the COD removal was very slow and did still contain at least 80-90% of its initial COD_{fil} after 2 weeks.

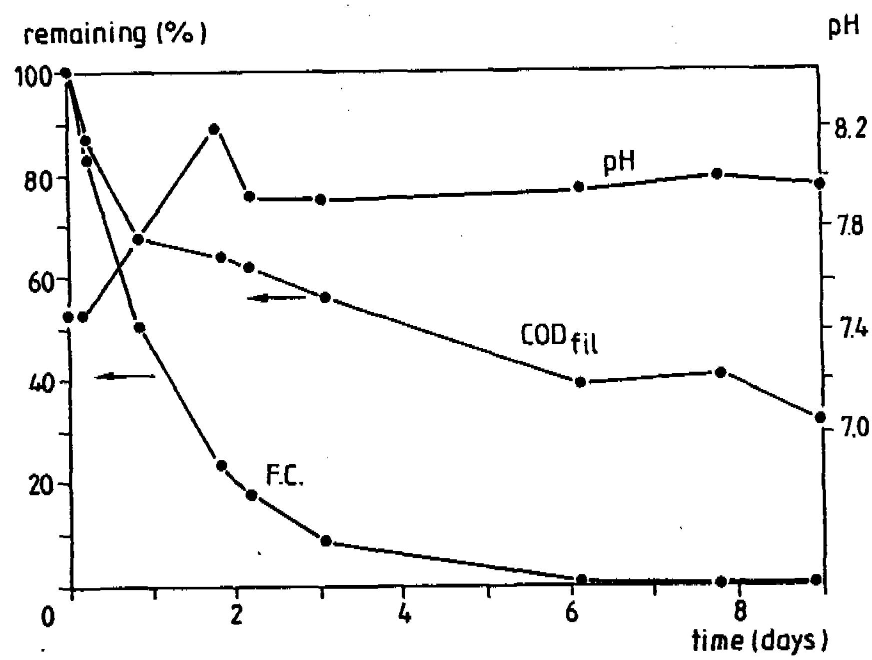


Figure 1. COD_{fil} , pH, and faecal coliform count with time in sponge filtered UASB-effluent under continuous slow stirring.

UASB-effluent, if exposed to air and slightly stirred, undergoes a quick removal of COD_{fil} to about 65% of its initial value of appr. 250 mg/l within 1 day. See figure 1.

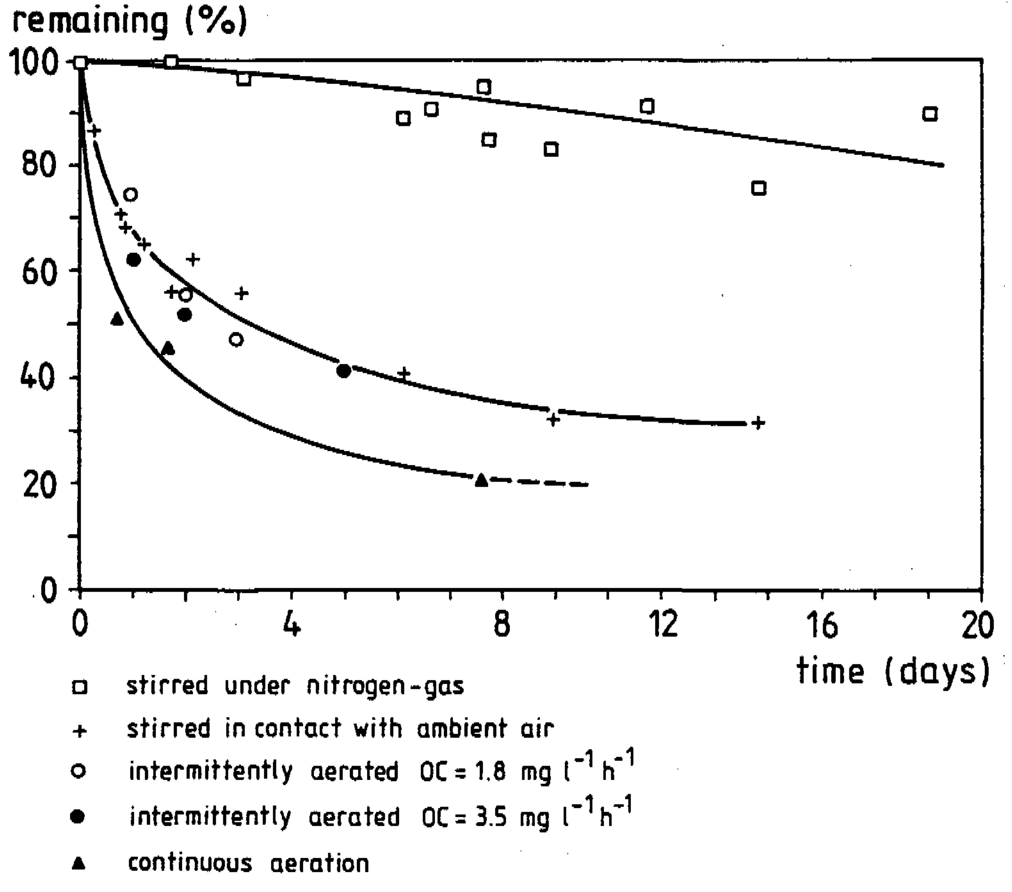


Figure 2. COD_{fil} removal as a function of time in UASB-effluent exposed to various intensities of aeration. T = 20 ° C.
The pH in slightly stirred experiments went up rapidly 0.5 - 1 pH-unit during the first 2 days, stayed constant for about 7 days at a pH in the range of 7.5 - 8.0 and then started to drop (nitrification).
In experiments with forced aeration the pH-effect was more pronounced. During the first 2 days the pH went up to about 8.4 (increase of 1 - 1,5 unit) and acidification started at day 6. Without air contact pH went up only slowly 0.25 units in 14 days.

When nitrate (4) or sulphate (7) were added as electron acceptors and air was excluded COD-conversion was considerably slower than with oxygen. Addition of denitrifying (5) and sulphate reducing sludge (8) did not improve the rate of COD or TOC removal. Addition of $FeSO_4$ but exclusion of air (9) caused flocculation but only a limited COD_{fil}-removal (35% after 215 hrs).

In separate experiments under the same conditions (T = 20° C) the faecal coliform decay was studied. During the first three days FC-decay followed a first order rate with an average decay constant K (natural logarithm basis) 1.30 day^{-1} (See Fig. 1). After this period the decay slowed down.

Batch experiments in a 800 litre aeration tank carried out by Lanting (1980) produced the results given in Table 5 and figure 2.
The aeration occurred intermittently by means of Brandolt tubes. The oxygenation capacity was changed by varying the proportion between aerated and non-aerated periods of time. To obtain a OC-value of 1.8 mg $O.l^{-1}.h^{-1}$ aeration was applied during 30 sec in a cycle of 10 minutes. At OC 3.5 aeration took place during 30 sec in a cycle of 5 min.

OC (mg $O.l^{-1}.h^{-1}$)	days	Reaction time 0	1	2	3	4	5	6	Overall COD-removal (%)
		COD($mg.l^{-1}$)							
1.8	COD_{unf}	183	140	-	79	-	-	-	57
	COD_{fil}	168	126	94	79	-	-	-	53
3.5	COD_{unf}	354	289	231	-	-	-	182	49
	COD_{fil}	200	125	104	-	-	84	83	58

Table 5. Batch treatment of UASB-effluent under microaerophilic conditions at two OC-values (Lanting,1980)

In these batch experiments without inoculation of sludge a two days aeration results in a COD_{fil}- removal of 44 and 48 %. The final COD_{fil} - value is about 80 $mg.l^{-1}$. Suspended solids removal expressed as COD_{ss} (2 days) were respectively 100% and 18%. The first value probably is mistaken due to an error in the COD analysis.
The data of Lanting are in accordance with those of De Fraiture (1978). The COD degradation is relatively fast during the first two days but decreases afterwards. The DO of the mixed liquor was far from constant. It increased from an initial zero to 2 - 5 $mg.l^{-1}$ after 36 h. This increase probably coincides with the completion of BOD degradation.
The oxygen input efficiency (= COD (g) removed/ O_2 (g) supplied) is high during the first day but strongly decreases afterwards. At both OC-values (1.8 and 3.5 $mg.l^{-1}.h^{-1}$.) an effective COD_{fil} removal was found. Oxygen input efficiency was highest at the lower OC-value. OC-values in continuously aerated suspended growth processes usually are about 20 mg O $l^{-1}.h^{-1}$.

2.2. Discussion

The fraction of COD_{tot} in UASB-effluent which can be removed by sedimentation was found to be 25%, but in practice varies considerably. It tends to increase on preservation of the UASB-effluent. On standing or aeration UASB-effluent undergoes a physico-chemical coagulation process. Colloidal particles become considerably bigger and settling is easier.

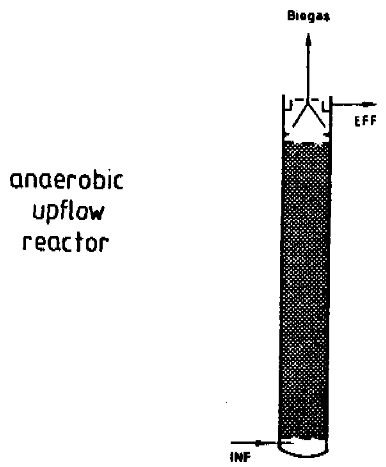
It can be concluded from batch experiments that exposure of UASB-effluent to air results in COD_{fil}-removal of about 35% in 24 hrs. The first order COD_{fil} conversion rate constant amounts to -0.43 d^{-1}. After 1 day the removal rate slows down and after 200 hrs about 70% of the COD_{fil} is removed. The contact with oxygen is more effective than with nitrate and sulfate in bringing about a removal of soluble COD and TOC. Forced aeration produces a COD_{fil} drop of 49% in 18 hrs. Keeping samples at 4° C in contact with ambient air brings about 32% COD_{fil} removal in 5 days.

3. Posttreatment of UASB effluent by means of Anaerobic Filtration

3.1.Introduction

Several investigators have used an anaerobic upflow filter (AF) as a second treatment step after a first anaerobic reactor. (Coulter et al. , 1957, Pretorius, 1971, Polprasert and Hoang, 1983)
The basic idea is an additional removal of SS, soluble COD and pathogens. No ammonium removal should be expected and neither much pathogen removal as long as the retention time is short.

Some data on posttreatment of UASB-effluent with an anaerobic upflow filter (figure 3) and a modification, the pebble bed clarifier (figure 4), are reported here.



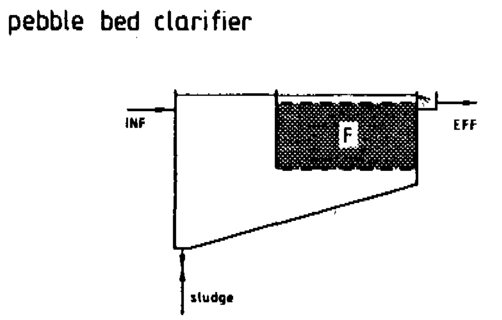


Figure 3. Anaerobic upflow filter   Figure 4. Pebble bed clarifier

No data are available about downflow filtration and horizontal anaerobic filtration. Downflow filtration in practice will be less useful as it has the disadvantages of bigger gas entrapment in the reactor as the direction of gas and liquid flow are opposite and too strong sludge wash out.

Horizontal flow has a movement of liquid and gas at a right angle which is not disadvantageous. Horizontal flow may have the disadvantage of the active anaerobic sludge settling on the bottom and staying out of reach of the fresh influent. So theoretically, downflow and horizontal flow are inferior methods.

3.2. Theoretical background of COD, SS and pathogen removal in anaerobic filters as a posttreatment system

The effect of merely sedimentation on UASB-effluent was tried by Haskoning (1985). These experiments were carried out in Colombia at an ambient temperature of about 25° C. The settler used had a volume of 780 litres . 51 % TSS removal, and 30% COD_{tot} removal were found at an HRT of 30 min. This treatment efficiency is very high.
With anaerobic filtration followed by sedimentation as a posttreatment method a 70% TSS removal and 49% COD_{tot}-removal was found at an HRT of 20 min.
At long retention times pathogen removal in upflow filters may be quite effective. The mechanism of pathogen removal is based on die-off in the liquid phase and possibly entrapment of microorganisms in the sludge, followed by die-off during the relatively long cell residence time. Polprasert and Hoang (1983) used an AF in the posttreatment of septic

tank effluent. They found a first order removal rate constant K_d (natural logarithm base) of -0.92 (d^{-1}) in the anaerobic filters corresponding to a 99% faecal coli elimination in 4 to 5 days liquid retention time at a temperature of approximately 28°C.

Haas and DeGooyer (1984) studied pathogen removal in an anaerobic filter fed with presettled domestic wastewater.
Average HRT was 10 hrs and water temperature 17 - 21 °C. Mean E. coli removal was 44% corresponding to a first order removal rate of -1.44 (d^{-1}) (assuming plug flow). Such faecal coliform removal rates do not indicate adsorption of the intestinal bacteria to the anaerobic sludge.

3.3. The Anaerobic Upflow Filter

3.3.1. Systems description

Lanting (1980) did pilot-plant experiments with a anaerobic filter of 4 m height and 0.30 m diameter. The filter was filled with PPR 2, grade A,700 Hydronyl as a medium for attached growth. In the top compartment a gas/solids/liquid-separator was mounted. This AF was fed with effluent from the $6m^3$ UASB-reactor at Bennekom. The UASB-reactor was fed with domestic wastewater. No inoculum of anaerobic sludge was applied.
Lanting varied the liquid retention time in the filter between 17.6 h and 6.7 h. Water temperatures were near 20° C.

3.3.2. Treatment efficiencies of the Anaerobic Filter as a posttreatment system

			Treatment efficiencies		
HRT (h)	OLR (kg $COD.m^{-3}.d^{-1}$)	HSLR ($m^3.m^{-2}.h^{-1}$)	TSS (%)	COD_{tot} (%)	COD_{fil} (%)
6.7	0.74	0.56	44	27	14
9.7	0.66	0.39	72	31	17
12.4	0.42	0.30	97	35	22
17.6	0.30	0.21	53	24	17

Table 6. Treatment efficiencies of an anaerobic upflow filter in the posttreatment of UASB-effluent at various hydraulic retention times (HRT). HSLR is the vertical flow velocity (empty column).
TSS was measured as COD_{ss}.

3.3.3. Conclusions

At the lower HRT values no decrease of COD removal efficiencies was observed as compared to upflow filtration at longer retention times, so it may be possible to obtain similar results at even higher loading rates. The removal of soluble COD is small, but SS-removal amounts to more than 40%. After approximately half a year an appreciable accumulation of sludge was found in the inferior one third part of the column. The TSS-removal in this work (44% at 6 h HRT, at TSS_{eff} = 32 $mg.l^{-1}$) was found to be significantly lower than in Haskoning (1985): 70% at 20 min, TSS_{eff} = 52 mg/l. But SS-removal was higher than the 25% that might be expected from the centrifugation and filtration experiments of DeFraiture. High removals may be possible at high average influent TSS-concentrations and a different character of the suspended solids in the Colombian experiments. COD_{fil} treatment efficiencies in our laboratory and the Colombian experiments did not differ much.
Pathogen removal in anaerobic filters seems limited by the relatively little entrapment in the sludge. More research on this last subject is necessary.

3.4. The pebble bed clarifier

3.4.1. Systems description

The pebble bed clarifier (PBC) as described by Mara (1977) (figure 4) consists of a sedimentation tank followed by an in-built pebble filter of small bed depth (0.15 - 0.5 m). The pebbles have a size of 1 - 2 cm.
The water passes the filter in an upflow direction and conditions are anaerobic so when used for wastewater it is one of the modifications of the anaerobic upflow filter
Because of its modest bed depth the HRT is small and filter cleaning is easy. The filter is drained once a week and the sludge is effectively removed during this procedure.

Experiments were carried out by Luyten (1985) and Postma (1987).
The pebble bed clarifier used had a net volume (after subtraction of the volume of the pebbles) of 20.1 litres. The filter section had a surface of 500 cm^2 and a depth of 15 cm. The filterbed volume was 7.5 litres of which approx 45 % was occupied by the pebbles.
At a loading rate is 8.4 $l.h^{-1}$ the HRT of the entire PBC is 2.4 h, while the HRT in the filter is about half an hour.

3.4.2. Treatment efficiency of the pebble bed clarifier as a posttreatment method

			Treatment efficiencies	
HRT (h)	Organic LR ($kg\ COD.m^{-3}.d^{-1}$)	HSLR ($m^3.m^{-2}.h$)	SS (%)	COD (%)
0.9	-	10.9	32	44
2.4	2.7	4.0	42	15
4.8	-	2.0	41	-

Table 7. The efficiencies of UASB-effluent treatment in a pebble bed clarifier. HSLR = hydraulic surface loading rate related to the empty filter bed.

3.4.3. Discussion

It is possible to remove 30 - 40% of the TSS by simply passing the UASB-effluent through a sedimentation tank and a pebble filter of 15 cm bed depth at a retention time of about 1 h. HRT values in the filter bed can be about 15 minutes. It is understandable that in such a short time no removal of soluble COD is to be expected.
In one of the experiments an average COD removal of 44% was found. This seems hardly possible as most of the COD found is in a soluble or very finely divided colloidal form.

Posttreatment methods based on particle separation such as settlers and anaerobic filters may remove 40 - 70 % of TSS (suspended and colloidal) at an HRT of 30 - 60 min. The finest colloidal fraction which constitutes the last 20 - 50 mg/l of TSS is not so easy to remove.
It is useful to further optimize the design of upflow filters as a posttreatment method.

4. Posttreatment of UASB-effluent by means of aerobic suspended growth methods

4.1. Introduction

In technologically developed countries the aerobic activated sludge system is already succesfully used in the posttreatment of UASB-effluent. This posttreatment system aims at additional organic matter removal and nitrification. Especially in tropical countries nitrification has to be combined with denitrification since unintentional denitrification (nitrogen gas formation) in the final settler impedes settling and a steady functioning of the activated sludge process (Van Haandel, 1990). A faecal coliform removal of 90 - 99% may be achieved (Feachem et al.,1983)

The conventional aerobic suspended growth process consists of an aeration tank (a completely stirred tank reactor) followed by a sedimentation tank in which activated sludge is settled and separated from the effluent (see figure 5). This sludge is recycled to the aeration tank in order to maintain a proper MLSS-concentration of 2 - 4 $g.l^{-1}$. To maintain a constant MLSS concentration excess sludge is wasted.
In correspondance with the required treatment efficiency the designed sludge loading rate (kg $BOD.kg(sludge)^{-1}.d^{-1}$) varies between 0.1 and 1. For nitrification low loading rates are necessary (0.1 - 0.2). Hydraulic retention times in the aeration tank vary between 0.1 and 1 day. The tank depth is 1 - 2 m. The DO level in the aeration tank is maintained at 2 - 5 $mg.l^{-1}$.

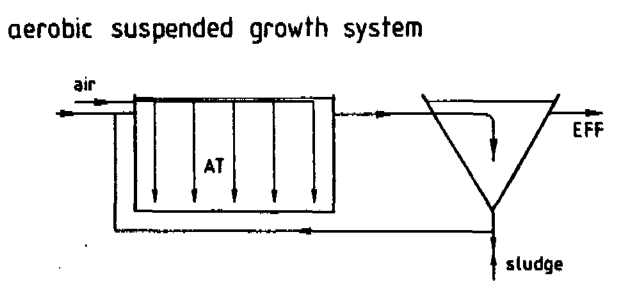


Figure 5. Aerobic suspended growth posttreatment system.

When the amount of oxygen in the aeration tank is limited DO levels stay low (< 1 $mg.l^{-1}$) and a microaerophilic flora may develop. Only little nitrification is expected. Such a treatment with restricted aeration is called microaerophilic treatment.
It may have the advantage over more forcefully aerated activated sludge systems of a lower energy input. Lanting (1980) did preliminary work on the posibilities of this process.

4.2. Theoretical background

When anaerobic effluent comes into contact with activated sludge a considerable adsorption of organic matter takes place. De Man (1983) measured this biosorption of COD_{fil} from UASB-effluent to aerobic activated sludge and found 51% removal in 30 min. This amounted to an adsorption of 16 mg COD_{fil}/ g sludge (d.m.). The further biosorption at contact times longer than 30 min. was not worth mentioning.
Van der Drift et al (1977) found that biosorption contributed 70% of the faecal coliform removal during aerobic activated sludge treatment of settled sewage.

The average BOD_{5diss}, COD_{diss} and COD_{tot} of UASB effluent amount to 60, 130 and 200 $mg.l^{-1}$ respectively (See table 1).

The sludge loading rate $B_x = S_0/X.t$,
in which

S_0 = influent BOD_{5diss} or COD_{fil} concentration	$(kg.m^{-3})$
X = sludge concentration (MLVSS) in the aeration tank	$(kg.m^{-3})$
t = theoretical hydraulic retention time of the influent in the aeration tank	(d)

The treatment efficiency in aerobic suspended growth systems as a function of SLR may be derived from empirical literature data.
If soluble influent BOD_5 (S_0) = 0.06, X is 2 and t is 0.21 (5 h) the BOD_{5diss} sludge loading rate amounts to 0.14 kg $BOD.kg^{-1}.d^{-1}$. At this low sludge loading rate a BOD treatment efficiency of over 90 % may be expected (Koot, 1972)

BOD_5 and COD treatment efficiencies may also be calculated on the basis of kinetic parameters and mass balances (Metcalf & Eddy, 1979).

$$r_{su} = \frac{Q.(S_0 - S)}{V} = \frac{\mu.X}{Y} = \frac{\mu_{max}.X.(S)}{Y.(K_s + S)} \quad (1)$$

Here, r_{su}	= substrate utilization rate	$(kg.m^{-3}.d^{-1})$
Q	= influent flow rate	$(m^3.d^{-1})$
S	= substrate concentration in mixed liquor (soluble BOD, COD)	$(kg.m^{-3})$
V	= volume of the aeration tank	(m^3)
μ	= specific growth rate	(d^{-1})
μ_{max}	= maximum specific growth rate	(d^{-1})
Y	= maximum yield coefficient	$(kg.kg^{-1})$
K_s	= half-velocity constant	$(kg.m^{-3})$

The value of this procedure is more theoretical than practical as it holds for soluble BOD only. Table 8 presents the treatment efficiencies of soluble BOD conversion in an aeration tank at assumed values of μ_{max}/Y using equation (1).

μ_{max}/Y (d^{-1})	S_0 $(kg.m^{-3})$	S $(kg.m^{-3})$	BOD treatment efficiency (%)
1	0.06	0.0126	79
3	0.06	0.0046	92
5	0.06	0.0028	95

Table 8. Soluble BOD treatment efficiencies as a function of assumed values of μ_{max}/Y. Further assumptions: t = V/Q = 0.21 d, K_s = 0.1 kg $BOD.m^{-3}$, X = 2 (kg $VSS.m^{-3}$).

The μ_{max} and Y values for UASB-effluent are as yet unknown. According to Metcalf & Eddy typically $\mu_{max}/Y = 5\ d^{-1}$. But this parameter is likely to be lower in UASB-effluent as the organic matter in UASB-effluent is more difficult to degrade.

As the BOD/N_{Kj} ratio in UASB-effluent from domestic origin is 3 or less, the nitrifying capacity of the sludge can usually develop well. This development depends on the growth conditions of the nitrifying organisms in the mixed liquor such as pH (opt. 7.2 - 9.0) , temperature (10 - 30 ° C) , DO-content (> 1.5 - 2 $mg.l^{-1}$).

Whether nitrifying organisms can sustain in the mixed liquor basically depends on the sludge age t_s.

t_s = sludge mass in aeration tank/sludge discharge ($kg.kg^{-1}.d^{-1}$) = sludge mass/ sludge growth ($kg.kg^{-1}.d^{-1}$) (steady-state conditions).

When the sludge discharged with the effluent can be neglected, the following equations hold:

$$r_g' = Y.r_{su} - k_d.X = \mu.X - k_d.X \qquad (2)$$

$$t_s = \frac{V.X}{V.(r_g')} = \frac{V.X}{V.(\mu.X - k_d.X)} = \frac{1}{\mu - k_d} \qquad (3)$$

In which:

r_g'	= net rate of sludge growth	($kg.m^{-3}.d^{-1}$)
k_d	= endogenous decay coefficient	(d^{-1})

A nitrifying flora can develop in the mixed liquor

$$\text{if } t_{sM} \geq \frac{1}{\mu_N - k_{dN}}$$

Usually $\mu_N >> k_d$ and the more simple approximation for the minimal cell residence time is used:

$$t_{sM} \geq \frac{1}{\mu_{max(N)}}$$

t_{sM}	= minimal cell residence time(d)
μN	= specific growth rate of nitrifying organisms(d^{-1})
$\mu_{max(N)}$	= maximum specific growth rate of the nitrifying organisms(d^{-1})
k_{dN}	= endogenous decay coefficient for nitrifyers(d^{-1})

A typical value for the rate-limiting conversion of ammonia to nitrite by Nitrosomonas : $\mu_{max(N)} = 0.3\ d^{-1}$ (at 20° C), neutral pH and a DO-value of 2.5 $mg.l^{-1}$. At lower DO-values a lower nitrification rate may be expected.
The theoretical minimal sludge residence time under these conditions $t_s \geq 3.3$ days. At low COD sludge loading rates this sludge residence time is easily achieved. According to Metcalf and Eddy (1979) the fraction of nitrifyers found at the prevailing BOD/N_{Kj} ratio of 3 is 0.083.

4.3. The activated sludge system

4.3.1. System description

Experiments on the posttreatment of UASB-effluent with a suspended growth system were carried out by De Man (1983).
The influent was UASB-effluent from a 6 m^3 UASB reactor fed with domestic wastewater. The UASB-effluent was first presettled and subsequently fed into the aeration tank (volume: 0.13 m^3). The secondary settler had a volume of 70 litres and the surface load was 0.39 $m^3.m^{-2}.h^{-1}$ at $B_{COD} = 0.3\ kg.kg^{-1}.d^{-1}$ and a 100 % sludge recycling (HRT = 0.78 h).

The temperature varied between 15 and 18° C.
In the experiments series 1 the activated sludge used for initial seeding was cultivated in domestic wastewater. In the first period (1A) the COD-loading rate was relatively high (0.6 $kg.kg^{-1}.d^{-1}$) and little nitrification was found. In order to stimulate nitrification the COD-loading was reduced to 0.21 $kg.kg^{-1}.d^{-1}$(1B) and later enhanced again to 0.30 kg $COD.kg^{-1}.d^{-1}$(1C). At this loading rate finally bulking sludge occurred and the experiment was finished. In series 2 a 6 litre contact vessel in order to prevent bulking sludge was applied and a nitrifying activated sludge was seeded (nitrifying capacity 0.03 kg NH_4-$N.kg^{-1}$ sludge $(d.m.).d^{-1}$). The COD-loading was kept at 0.12 $kg.kg^{-1}.d^{-1}$ (2A) and later enhanced in a step-wise way (2B).
The MLSS-concentration was 2 - 4 gr $(d.m.).l^{-1}$.
Air was diffused by means of Brandolt-tubes. DO was kept at values between 2 and 5 mg/l.
In the experiments with a relatively high degree of nitrification (1 B, 1C, 2A and 2B) the pH of the mixed liquor was kept constant at 7.0 by means of an automatic titrator.

Lanting(1980) did experiments in which the conditions were 'microaerophilic'.
The continuous experiments were carried out in a 900 litre aeration tank followed by a 70 litre sedimentation tank. Settled sludge was recirculated from the sedimentation tank into the aeration tank. Aeration was carried out intermittently by Brandolt tubes.
By intermittently supplying air low dissolved oxygen concentrations were obtained. The oxygenation capacity (OC) in these cases was low as well.
The ratio between the aeration time and the length of the total cycle of an aerated and non-aerated period is indicated with f_a.
At first the OC-value was 3.5 mg $O.l^{-1}.h^{-1}$ at f_a = 0.5 min aeration per 5 min cycle.
As treatment results were poor OC values were increased to 5.4 (f_a = 0.5 : 2.5) and later to 11.4 at f_a = 0.5 : 2.5. Here, the flow of air was intensified.

4.3.2. Treatment efficiencies of an activated sludge system as a posttreatment system for UASB-effluent

#	HRT (h)	B_{COD}	B_N	Treatment efficiencies SS (%)	COD(*) (%)	COD_{fil} (%)	BOD_5 (%)	NH_3-N (%)	TKN (%)
1A	2.5	0.6	0.18	-250	-4	42	81(*)	-	23
1C	3.6	0.3	0.095	55	59	60	-	45	55
1B	5.4	0.21	0.069	37	46	48	90	88	79
2A	10.5	0.12	0.035	96	73	64	95	73	-
2B	2.7	0.55	0.16	-	75	70-80	-	15	-
2B	var.	0.3-0.6	0.09-0.15	-	75	70-80	-	50-80	-

Table 9. Treatment efficiencies of an aerated suspended growth system in the post treatment of UASB-effluent (data from G. de Man, 1983)
*Efficiency based on unfiltered influent and filtered effluent from the settler.
Temperature varied between 15 and 18° C., Sludge loading rates (SLR) expressed in $kg.kg^{-1}.d^{-1}$. COD_{fil} based on samples filtered through filterpaper MN nr 615 ¼.

OC (mg O/l.h)	f_a (-)	MLSS ($g.l^{-1}$)	HRT (h)	Treatment efficiencies COD_{unf} (%)	COD_{fil} (%)
3.5	0.5 : 5	low	29	19	15
5.4	0.5 : 2.5	low	29	37	24
11.4	0.5 : 2.5	2.3	25.5	35-40	35-40

Table 10. Treatment of UASB effluent in a microaerophilic continuous process. Temperatures varied between 17 and 20° C (from Lanting (1980))

At the start of Lanting's experiments no activated sludge was added as inoculum. At the end of the experiments a MLSS-concentration of 2.3 $g.l^{-1}$ was reported. Then, the COD_{unf} treatment efficiency was about 35% at a HRT of 25.5 h.

4.3.3. Discussion

Removal efficiencies (COD and N) at various loading rates
In experiment 1A (Table 9) at $B_{COD} = 0.60$ $kg.kg^{-1}.d^{-1}$ the MLSS-concentration was 2.4 $kg.m^{-3}$ on the average and there was sludge wash-out from the final settler as a consequence of filamentous sludge. This resulted in a negative value of the SS-removal and no COD_{tot} removal. Nitrification is low at this relatively high loading rate. The N-loading rate is 0.18 g $N.g^{-1}$ (d.m.).d^{-1}, which is considerably higher than the nitrifying capacity of the sludge.

In the next experiments (1B, 1C, 2A) the COD- and N-loading rates were lower and HRT higher. SS and COD removal efficiencies were higher and nitrification proceeded satisfactorily. In 1B at a B_{COD} of 0.21 $kg.kg^{-1}.d^{-1}$ the nitrification amounts to 88% NH_4-N removal. In this experiment a sludge age of 48 days was reported. In 2A at B_{COD} 0.12 the sludge age was 90 days. These sludge ages are very high. Probably, sludge escaping with the effluent has not been taken into account.
In 2B the HRT was in step-wise way diminished from 5.4 h to 2.7 h and B_{COD} enhanced from 0.28 to 0.60 $kg.kg^{-1}.d^{-1}$. At a retention time of 2.7 h the sludge age was approximately 12 days, which appeared too short to maintain the nitrification. If we compare the experimental sludge age (> 12 days) with the calculated value (> 3.3 days) it may be concluded that the growth rate of nitrifyers was relatively low: $\mu_N - k_{dN} < 0.08$. If k_{dN} is assumed to be 0.02 d^{-1}, the value of μ_N would be 0.1 d^{-1}. This difference between theoretical and found sludge age may also be explained by the neglect of sludge in the effluent.

Figure 6 shows the NH_4-N removal rate as a function of the NH_4-N-loading rate as found in experiment 2B.

In this figure it can be seen that NH_4-N-removal from UASB-effluent is > 80% as long as long as B_{COD} is lower than 0.3 $kg.kg^{-1}.d^{-1}$ and NH_4-N loading rate (B_N) smaller than 0.1 $kg.kg^{-1}.d^{-1}$. At higher organic loading rates the sludge age is too short to maintain growth of Nitrosomonas.

Phosphate removal amounted to approx. 14%, which can be ascribed to incorporation of phosphorus in the bacterial mass.
The residual COD_{fil} amounts to approx 50 - 70 mg/l as compared to 30 - 40 mg/l in the effluent of an activated sludge installation without anaerobic pretreatment. The residual effluent BOD_{fil} was 7 mg/l.

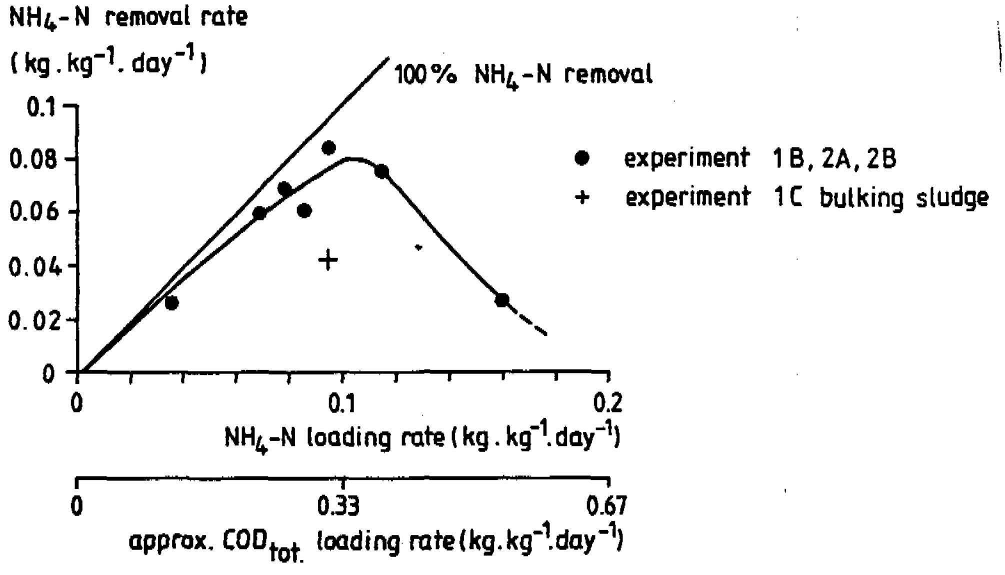


Figure 6. Ammonia removal rate ($kg.kg^{-1}.d^{-1}$) as a function of loading rate in an aerobic suspended growth system treating UASB-effluent. The COD_{tot} loading rate has been calculated from the NH_4-N- loading rate on the basis of an average proportion of COD_{tot}/NH_4N= 3.33 (See table 1).

The experiments by Lanting are not easy to interprete as probably no steady state was attained. In the beginning there is only very little activated sludge and the system acts as a system without sludge recycle.
In fact the experimental set-up corresponds with that of an aerated lagoon in which the sludge age equals the hydraulic retention time. Later the sludge concentration had increased and so had the treatment efficiency. From the batch experiments of COD_{fil} removal as a function of time (see figure 2) a (pseudo-)first order degradation rate constant could be derived. At both OC-values approximately the same K-value was found. Calculated over the first day K = -0.38 d^{-1}; over the first 3 days K = -0.21 d^{-1}. Applying these data on the continuous flow system and assuming K = -0.38 d^{-1}, HRT = 29 h. and completely mixed behaviour in the aeration tank, a COD_{fil}-removal of 31 % is expected.
The value found is 24%., which corresponds to a K-value of 0.26 d^{-1}.

It is not yet clear whether something like a microaerophilic process should be distinguished among the aerobic suspended growth processes. A criterion for such a process would be the growth of a special bacterial flora. It appears that on air contact some relatively quick COD removal takes place as a consequence of oxydation of readily degradable substances. It is an activated sludge process in which less energy is used for turbulence.
At any rate is seems useful to further study the possiblities of posttreatment at minimal energy expenditure.

Sludge characteristics
In 1A (De Man,1983) the sludge was flocculent. Gradually the number of filaments increased but later they disappeared. In 1B the SV was about 65 $ml.l^{-1}$. Later in 1C the SV increased again as a consequence of organic overloading.
Depending on the fate of the SS entrapped from the UASB-effluent in the activated sludge a net sludge yield factor of 0.03 - 0.25 kg(d.m.).kg^{-1} COD converted was found.

In 1A the nitrification capacity of 0.04 kg.NH_4-N $kg^{-1}.d^{-1}$ was found. Later in 2B the nitrification capacity of the sludge amounted to 0.08 kg NH_4-N $kg^{-1}.d^{-1}$.

As a summary it should be noted that COD removal from UASB-effluent in aerobic suspended growth systems is well possible at B_{COD} up to 0.6 kg COD.$kg^{-1}.d^{-1}$. Even at this loading rate COD_{fil} treatment efficiencies of 70 - 75% are found. Sufficient nitrification, however, needs a lower loading rate. A 50 - 80 % NH_4-N removal can be found at COD loading rates up to 0.35 kg.$kg^{-1}.d^{-1}$.
This type of posttreatment reactor could well be designed based on the nitrifying capacity of the sludge: e.g. 0.07 gr N/gr d.m. day. At organic loading rates of 0.6 kg.$kg^{-1}.d^{-1}$ and higher bulking sludge can be a problem. Growth of the filamentous sulphide oxydizing Thiothrix species could be the cause.

5. Posttreatment of UASB-effluent by means of Aerobic Attached Growth systems

5.1. Introduction

An attached growth reactor involves a solid medium on which an active bacterial mass, the biofilm, grows. The systems used for wastewater purification are trickling filters(TF), rotating biological contactors (RBC) and submerged filters (SF). Assets of attached growth systems as compared to the suspended growth systems are the absence of sludge recirculation and hence an independence of the settling characteristics of the sludge, good settling and dewatering characteristics of the excess sludge, less energy input for the oxygenation (especially for TF and RBC) and simple maintenance. Drawbacks may be lesser controlability, slightly higher capital costs (Wouda, 1982) and higher susceptibility to toxic compounds. Posttreatment of UASB-effluent can be conducted in single or two stage systems. It has been shown that especially two stage systems, making use of two types of sludge, can be highly efficient. In the first stage BOD removal takes place. Here, the biofilm consists of heterotrophic bacteria and has a high capacity to adsorb suspended particles. In the second stage nitrification takes place.

A single stage trickling filter followed by a settler was studied by Haskoning(1985) for effluent of a 64 m^3 UASB reactor in Cali, Colombia. The treatment results were poor as the filtermedium consisted of big stones (diam. 9 cm) with a very low specific surface for attached growth. At a surface loading rate of 20 $m^3.m^{-2}.d^{-1}$ the average COD_{tot} removal was 16%. Nitrification was 10%. FC removal was slight.

Trickling filters and biological contactors (biorotors) were studied extensively by Hack & Klapwijk (1984), Bovendeur & Klapwijk (1986) and Bovendeur (1989).

5.2. Theoretical considerations

An analytical theory of the conversion of COD and ammonia in aerated attached growth systems has been presented by Harremoës (1982), Harremoës (1984), Arvin & Harremoës (1990).

When wastewater comes into contact with the biofilm soluble organic matter (COD_{diss}), ammonia and oxygen enter the bacterial film by molecular diffusion. The COD-oxydation and nitrification rates in the bulk of the reactor depend on the thickness of the film, on the availability of any one of the reactants in the film, the composition of the bacterial flora in the biofilm and the transport of reaction products out of the film. In the treatment of domestic wastewater and UASB-effluent COD-oxydation and nitrification may take place simultaneously. The slower growing nitrifiers have to compete with the heterotrophic organisms. So, nitrification can only take place at reduced COD and BOD levels and sufficient DO. Harremoës (1982) concludes that a stable nitrification requires a BOD-concentration less than 20 $mg.l^{-1}$ at DO = 3 $mg.l^{-1}$. It can be concluded that an advanced nitrification of UASB-effluent (BOD = 80 $mg.l^{-1}$) requires a loading rate such that BOD in the reactor is < 20 $mg.l^{-1}$ or removal of BOD in a first-stage reactor prior to nitrification in a second stage.

The most usual circumstance in the treatment of UASB-effluent with attached growth systems is an oxygen limited simultaneous conversion of biodegradable COD and NH_4-N. Figure 8 presents three possible cases of concentration profiles of the reactants in the biofilm.

In the first case the organic matter concentration in the bulk liquid phase is relatively high (> 20 $mg.l^{-1}$) and nitrifiers cannot grow well. The nitrification rate is small. This could be the case in a single stage attached growth posttreatment system.

In the second case the organic matter concentration has been reduced (BOD < 20 $mg.l^{-1}$) and nitrifiers can compete well with heterotrophs. In this case the conversion reactions are 0-order as to the NH_4-N concentration and ½- order as to the oxygen concentration. As to organic substrate the film is partly penetrated and the BOD degradation reaction is ½-order in BOD-concentration.

When the ammonia concentration in the bulk is reduced the nitrification rate may become limited by ammonia instead of oxygen (case 3)
At a DO and BOD of 5 and 10 $mg.l^{-1}$ respectively the transition between case 2 and 3 takes place at an NH_4-N-concentration of approximately 1 $mg.l^{-1}$.

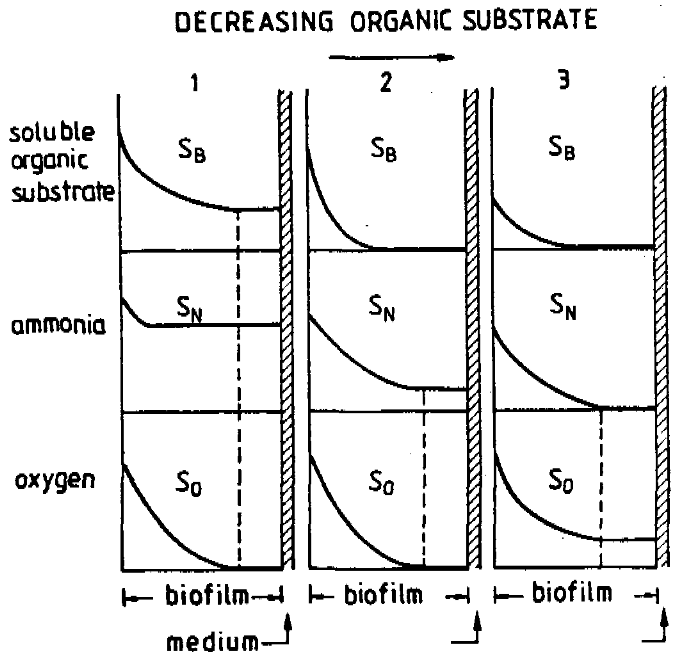


Figure 8. Three cases of penetration of reactants into a biofilm.

In the three cases the reaction rates can be expressed as follows:

Case	r_{aB}	r_{aN}
1	$\frac{1}{M_{BO}} . k_{½BO}.S_O^{½}$	$\frac{1}{M_{NO}} . k_{½NO}.S_O^{½}$
2	$k_{½B}.S_B^{½}$	$\frac{1}{M_{NO}} . k_{½NO}.S_O^{½}$
3.	$k_{½B}.S_B^{½}$	$k_{½N}.S_N^{½}$

Here,

r_{aB} = BOD conversion rate per unit of film surface (g $O.m^{-2}.d^{-1}$)
r_{aN} = nitrification rate per unit of biofilm surface (g $N.m^{-2}.d^{-1}$)
M_{BO} = stoichiometric constant of the BOD conversion
(= 0.5 g O_2/ g BOD)
M_{NO} = stoichiometric constant of the nitrification
(= 4.3 g O_2/ g NH_3-N)

$k_{½BO}$= ½ order rate constant of BOD conversion limited by oxygen diffusion (g $O)^{½}.m^{-½}.d^{-1}$)

$k_{½NO}$ = ½ order rate constant in a process limited by oxygen diffusion $(g\ O)^{½}.m^{-½}.d^{-1}$)

$k_{½B}$ = ½ order rate constant of BOD conversion limited by BOD $(g(BOD))^{½}.m^{-½}.d^{-1}$)

$k_{½N}$ = ½ order rate constant of nitrification limited by ammonia $(g\ (NH_4\text{-}N))^{½}.m^{-½}.d^{-1}$)

S_B = soluble BOD concentration ($g.m^{-3}$)
S_N = NH_4-N concentration ($g.m^{-3}$)
S_O = dissolved oxygen concentration ($g.m^{-3}$)

The values of M_{BO} (=0.5) and M_{NO} (=4.3) deviate from the theoretical values of 1.00 and 4.57 respectively because a part of the substrates is not oxidized but fixed into the biofilm without conversion.
The values of $k_{½BO}$ and $k_{½NO}$ are influenced by the composition of the bacterial flora. The maximum values are reached when no competing substrate is present. Thus:

$k_{½NO} = c.k_{½NO,max}$, in which c (< 1) is a reduction factor expressing the presence of heterotrophic bacteria next to the nitrifiers.
Harremoës(1984) found values of $k_{½BO,max} = 4.1$ and $k_{½NO,max} = 3.8\ (g\ O_2)^{½}.m^{-½}.d^{-1}$.
If the BOD conversion is limited by the oxygen diffusion and DO-concentration is 3 $mg.l^{-1}$ the maximum BOD conversion rate (r_{aB}) is: 12.3 g O_2. $m^{-2}.d^{-1}$ and r_{aN} = 2.85 g NH_4-N. $m^{-2}.d^{-1}$.

5.3. Trickling filters

5.3.1. Method description

In our laboratory Heydeman (1979) was the first to study the treatment of UASB-effluent from domestic origin with a one-stage trickling filter. The filter had a filterbed height of 1.15 m. The filter media consisted of pebbles. No settler was installed, no effluent recirculation was applied. At a (low) hydraulic loading rate of 0.1 $m^3.m^{-3}.h^{-1}$, an organic loading rate of 0.54 kg $COD.m^{-3}.d^{-1}$ and 29° C 36% COD_{fil}-removal was obtained.

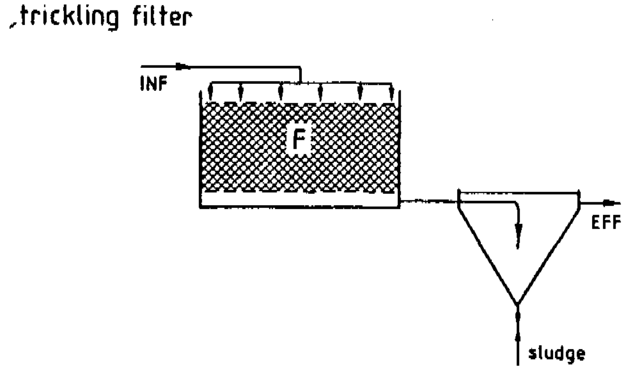


Figure.8. Single stage trickling filter with settler.

Bovendeur & Klapwijk (1986) used a trickling filter consisting of two columns in series provided with a final settler. By mounting an intermediate settler the installation could be transformed into a two stage system. The two columns had a square cross-sectional area of 0.16 m^2 and heights of 4.3 m and 2.3 m respectively. The effective filter volume of the columns was 0.58 m^3 and 0.29 m^3. The UASB-effluent flow rate amounted to 0.225 $m^3.h^{-1}$. It was mixed with recirculated settled filter effluent and added on the top of the first filter.
Filterpack CR 50 was used as a filter medium. This material consists of PVC cylinders. The specific surface area is approximately 200 $m^2.m^{-3}$. The pore volume is 96%.
The filters were aerated by a forced downflow air/oxygen stream of 4.7 $m^3.h^{-1}$.

5.3.2. Treatment efficiencies of trickling filters

In the work of Bovendeur & Klapwijk (1986) COD-loading rates varied between 1 and 4 kg $COD.m^{-3}.d^{-1}$. The variations were due to UASB-effluent COD fluctuations. Assuming the 200 $m^2.m^{-3}$ surface of the filtermedium this corresponds to biofilm loading rates of ~~5 - 20 g COD.m^{-2}.d^{-1}.~~
Irrespective of the applied loading rates the COD_{tot}-removal amounted to 50 - 60%. COD_{diss}-removal rates were usually somewhat lower, but high concentrations of suspended COD did not necessarily produce high COD removal efficiencies. Poor treatment efficiencies were found occasionally.
Since a trickling filter is a plug flow reactor the highest COD removal rates are found in the top regions of the filter.
Here, the COD_{fil} removal rate amounts to about 6 gr $COD_{fil}.m^{-2}.d^{-1}$. It was shown that the conversion process was substrate limited.
Kjeldahl-nitrogen loading rates varied between 100 and 600 gr $N_{kj}.m^{-3}.d^{-1}$. corresponding to biofilm loading rates of 0.5 - 3 gr $N_{kj}.m^{-2}.d^{-1}$.
N_{kj} removal rates varied between 50 and 200 $gr.m^{-3}.d^{-1}$ and treatment efficiencies were between 10 and 50% and showed little correspondence to the Kjeldahl-N loading rates. The average Kjeldahl-N oxydation rate was calculated to be 0.32 g. $m^{-2}.d^{-1}$. In the trickling filter a considerable denitrifying activity was found in the top region as the nitrate containing effluent was recirculated to enhance the hydraulic loading rate. The presence of a settler after the first stage column did not affect the N_{kj} removal efficiency in the second stage.

De Man (1985) worked with the same installation and found an average ammonia nitrogen removal of 20% at an average COD loading rate of 1.10 kg $COD_{fil}.m^{-3}.d^{-1}$ (5.5 g $COD.m^{-2}.d^{-1}$).
The excess sludge from trickling filters has excellent settling characteristics (SV = 60 $ml.l^{-1}$). The production is low: approximately 0.1 gr sludge COD per gr COD removed.

HL ($m^3.m^{-3}.h^{-1}$)	Organic LR (kg $COD.m^{-3}.d^{-1}$)	Treatment efficiencies (%) COD	NH3-N	N_{kj}	Author(s)
0.1(S1)	0.54	36*	-	-	Heydeman,79
0.26(S1+2)	1.20*	49*	20	-	De Man, 85
0.26(S1+2)	1 - 4	50$	-	10 - 50	Bovendeur,89

Table.11. Treatment of UASB-effluent using trickling filters (lab and pilot plant scale), S1= single stage, S1+2= two stages (without an intermediate settler), HL = Hydraulic loading rate based on wastewater flow rate = reciprocal value of hydraulic retention time.

* COD removal was determined from filtered influent and effluent samples.
$ COD removal based on unfiltered UASB effluent and unfiltered effluent from a final settler.

5.3.3. Discussion

COD_{tot} and COD_{fil} removal amount to approximately 50%, and ammonia and Kjeldahl nitrogen removal are between 10 and 50% in a well recirculated trickling filter irrespective of the organic loading rate. Fecal coliform removal was not recorded in this work. Treatment efficiencies were low a single stage filter (Haskoning, 1985) and 90-95 % as a secondary treatment method (Feachem et al., 1980)

5.4. Biorotors as a posttreatment method for UASB-effluent

5.4.1. System description

In the study of biorotors (or rotating biological contactors) a two stage biorotor system was used. The system consisted of two identical units connected in series with settling tanks following each unit. Each of the biorotor units contained 10 polystyrene foam discs of 0.59 m diameter and 0.02 m thick, mounted on a steel shaft. The total disc surface per unit was 5.2 m^2. 35% of the disc surface was submerged. The trough volume is 0.055 m^3 filled with 0.036 m^3 water.
The speed of disk rotation was 18 r.p.m. in the first and 12 r.p.m. in the second stage.

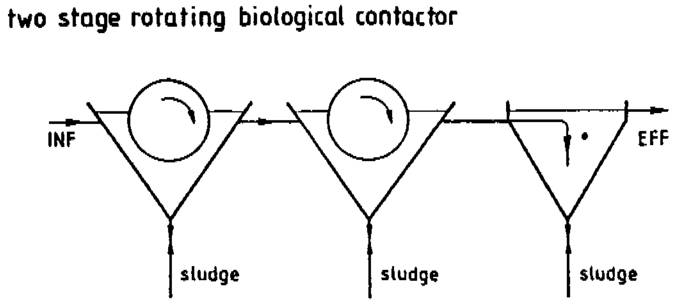


Figure 9. Two stage rotating biological contactor

Hack and Klapwijk (1984) studied UASB-effluent treatment in this system.
During their work temperatures varied between 13 and 21° C. The first biorotor unit was relatively highly loaded using HRT values between 0.51 and 1.56 h. The speed of disk rotation was 18 r.p.m. The second stage was meant for nitrification and had HRT values between 2.8 and 4 h.

Bovendeur & Klapwijk (1986) applied HRT values of 0.24 in the first and 2.76 hr in the second disk unit.
Due to fluctuations of the COD concentration of the UASB-effluent COD loading rates varied between 30 and 400 $gr.m^{-2}.d^{-1}$ or
4.33 - 58 kg $COD.m^{-3}.d^{-1}$ in the first stage rotating biological contactor. N_{kj} loading rates varied between 1.5 and 4 $gr.m^{-2}.d^{-1}$ in the second stage. Temperatures varied between 12 - 20° C.

Postma (1987) did posttreatment experiments with the first stage biorotor only followed by a sedimentation tank.
The hydraulic retention time in the trough was 2.4 h.
Water temperatures varied between 8 and 20° C. Disc rotation speed was set at 4 and later 10 rpm.

Haas and De Gooyer (1984) measured the removal of several indicators of pathogenic organisms in the two stage biorotor installation at unknown residence times.

The faecal coliform removal from the effluent of an EGSB reactor (HRT = 1.5 h) in each of the biorotor stages was determined separately at hydraulic retention times of 1.75 h in the first and 1.7 h in the second unit.

5.4.2. Treatment efficiencies of biorotors as a postreatment system.

HRT (h)	Organic LR (kg $COD.m^{-3}.d^{-1}$)	Surface LR (kg $COD.m^{-2}.d^{-1}$)	Treatment efficiencies (%) SS	COD_{tot}	COD_{fil}	NH_4-N
2.4 (S1)	4.27	0.04	66	67	-	35
1.56(S1)	-	0.036	-	53	50	-
0.51(S1)	-	0.097	-	38	40	-
0.24(S1)	4 - 58	0.03 - 0.4	-	40-80	20-40	0
2.76(S2)	0.57-2	0.004 - 0.014	-	-	-	70-90
3.5(S1+2)	-	-	-	72	-	73

Table 12. Treatment of UASB-effluent in one and two stage biorotor installations at various loading rates (Hack & Klapwijk (1984), Bovendeur & Klapwijk (1986), Postma (1987)).
S1 = one stage system,
S2 = second stage of a two stage system,
S1+ 2 = two stage system.

The results of experiments on pathogen indicator removal are given in table 13.

Organism	Mean removal (%)
E. coli(S1+S2)	95
E. coli(S1)	0(*)
E.coli(S2)	51(*)
Streptococcus faecalis(S1+2)	94
Clostridium(S1+2)	79
Coliphages(S1+2)	52

Table: 13. Efficiency of removal of several pathogen indicator from UASB-effluent in a two stage biorotor unit, residence time unknown, Temperature: 13 - 21°C.
* EGSB-effluent at HRT-values of 1.7 h.

5.4.3. Discussion

Biorotors including sedimentation tanks appear to be a relatively effective method of UASB-effluent posttreatment.
An effective COD removal is possible at small hydraulic retention times in a one stage system. Bovendeur & Klapwijk (1986) demonstrated the significant influence of the suspended matter concentration on the COD-removal rate.
When COD_{tot}/COD_{fil} was < 2 and the concentration of suspended solids was low COD_{tot}-removal rates reached a maximum of about 40 gr $COD.m^{-2}.d^{-1}$.

at a loading rate of 80 gr COD.$m^{-2}.d^{-1}$. This means that up to this loading rate the COD_{tot} removal is about 50% but decreases at higher loading rates. COD_{tot}/COD_{fil} ratios < 2 are usual in UASB-effluent.

At high suspended solids concentrations however the removal rate (gr COD. $m^{-2}.d^{-1}$ removed) does not reach a constant level but increases linearly with the loading rate. Up to high COD loading rates (200 g.$m^{-2}.d^{-1}$) a 70% removal of COD_{tot} can be obtained. This means that suspended solids are effectively adsorbed by the bacterial mass on the disks.

In table 12 the beneficial effect of a two stage system on the nitrification is clearly demonstrated. While the single stage system exhibits an average nitrification of 35% (N_{Kj} LR = 3.1 gr.$m^{-2}.d^{-1}$, N_{Kj}RR = 0.35 x 3.1 = 1.1 g.$m^{-2}.d^{-1}$), the two stage system is able to bring about at least 70% N_{Kj} oxydation at a HRT of about 3 hrs at the same or even higher loading rates in the second stage reactor. Here the N_{Kj} RR amounts to 2.4 g.$m^{-2}.d^{-1}$. This is brought about by the elimination of COD in the first stage, so that on the disks in the second unit the ratio of nitrifiers/heterotrophs is higher.

The pathogen removal from UASB-effluent in biorotor installations has not yet been extensively studied. Sagy and Kott (1990) studied the treatment of settled sewage in a single stage biorotor installation at temperatures between 12 and 29° C. Faecal coliform treatment efficiencies were about 90% (residence time unspecified). Efficiencies of 99 - 99.9% were obtained at longer retention times.
As biorotors seem to be able to absorb much suspended matter (to which category bacteria and higher pathogenic organisms belong) it is expected that the system could perform well in pathogenic organism removal.
The data gathered so far are not conclusive.
The work of Haas & DeGooijer(1984) showed results corresponding to the ones found by Sagy & Kott (1990). Other measurements shown in table 13 demonstrated that removal at short retention times can be poor.

The excess sludge production was determined by the COD loading rate. Hack & Klapwijk (1984) found a mean value of the net sludge yield (= excess sludge COD/(excess sludge COD + oxydized COD)) of 0.57 g sludge-COD per g COD removed was found.

5.5. Submerged filters

5.5.1. System description

Submerged filters (also called packed bed reactors) consist of an aerated tank filled with a submerged support material on which the aerobic sludge is growing. The aerated tank is followed by a sedimentation tank. No sludge is recycled. Attention should be paid to the flow conditions in the filter which may be impaired by sludge accumulation. In this case backwashing should be applied.
Schlegel (1988) has described several modifications and applications of submerged filters. Submerged filters are claimed to be useful as a nitrification step after preliminary organic matter removal.

In our laboratory Ancher (1990) used a one-stage submerged upflow filter (tank volume = 116 l, height 0.60 m) filled with PVC rings (specific surface: 180 $m^{-2}.m^{-3}$). The filter was aerated using two aeration disks. The air flow amounted to 10 m^3 air/ m^3 reactor.
The submerged filter was followed by a 120 l settler.
The filter was fed with UASB effluent. The temperatures varied between 19 and 25° C. The filter was studied at HRT-values of 10.3 h and 5.9 h. No filter backwashing was applied.

submerged filter (upflow)

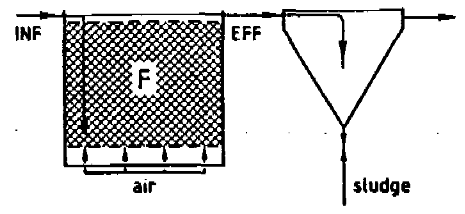


Figure 10. Single stage submerged filter with settling tank.

5.5.2. Treatment efficiencies

HRT (h)	Organic LR (kg $COD.m^{-3}.d^{-1}$)	Surface LR (g $COD.m^{-2}.d^{-1}$)	Treatment efficiency (%) COD_{tot}	COD_{fil}	NH_4-N	FC
5.9	0.88	4.9	-	-	44	96
10.3	0.50	2.8	61	53	75	98

Table 14. Treatment of UASB-effluent with a submerged filter (Ancher, 1990)

The suspended solids concentrations in the SF effluent were low (COD_{ss} = 14 $mg.l^{-1}$). Therefore, a settler was not really useful during normal operation, but it is necessary when filter backwashing is applied.
Effluent DO - values were always higher than 5 $mg.l^{-1}$.

5.5.3. Discussion

As a consequence of the very low sludge discharge from the filter a gradual accumulation of sludge in the filter takes place. This eventually causes filter clogging and anaerobic zones. Regular filter cleaning by an enhanced air or water flow should take place.
From the data presented above it can be concluded that the submerged filter produced a rather poor COD (50-60%) but a good FC removal. COD_{diss} SRR amounted to 1.04 $g.m^{-2}.d^{-1}$. Ammonia removal was 0.56 $g.m^{-2}.d^{-1}$ both at 5.9 and 10.3 h retention time. The nitrification is limited by oxygen availability and the small fraction of nitrifiers in the film. It can be improved by applying a two stage system in which the first stage specializes on COD removal and the second on NH_4-N conversion. By recirculating the effluent a considerable denitrification in the first stage might be achieved.
Rusten (1984) treated settled municipal wastewater with a single stage submerged filter and obtained a 70% COD removal at a COD loading rate of 20 $g.m^{-2}.d^{-1}$. At low organic loading rates nitrification rates of 1.3 g NH_3-N removed/m^{-2}.d were found. This may be regarded as a high value as BOD values in the reactor were not particularly low: 10 - 30 $mg.l^{-1}$.
In the submerged filter as in the suspended growth systems protozoa seem to play an important role in the conversion of sludge particles. At a retention time of 5.9 h several

types of ciliates and amoebae could be distinguished. The concentrations were 10^4 - 10^5/100 ml. These organisms could play an important role in the removal of free-floating (pathogenic) bacteria.

5.6. Cascades as a posttreatment method for UASB-effluent

5.6.1. System description

The only known experiment so far was done by Heydeman (1979).
The cascades consisted of a 1.5 m plexiglass 'ladder' with some liquid hold up on each step. It was followed by a settling tank. No sludge recirculation was practiced. The surface of the cascade gradually gets covered with sludge. The angle of inclination was varied to control the liquid hold-up on the steps of the cascade.
The temperatures of the experiments mentioned were between 21 and 26 ° C.

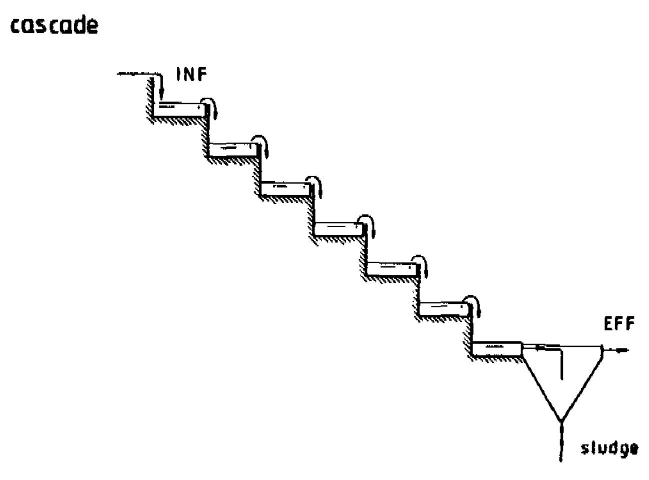


Figure 11. Cascade.

5.6.2. Treatment efficiencies of the cascades as a posttreatment system

Slope	HRT (h)	Treatment efficiency Liquid holdup (ml)	COD_{fil} (%)
30°	< 1 min	400	14
45°	10 sec	250	7

Table 15. Treatment efficiency of a cascade for posttreatment of UASB effluent (Heydeman, 1979)

5.6.3. Discussion on cascades

As the retention time is very short no great treatment results may be expected. Moreover, the flow in cascades tends to follow certain channels through the bacterial mass, so that the contact surface is relatively small. A smaller slope of the cascade produces a lower flow rate and somewhat higher treatment efficiencies. In some specific situations natural slopes may be used for a kind of overland flow.

5.7. Discussion on attached growth methods

In order to make a comparison of the studied attached growth systems the COD and N removal rates are summarized in table 16.

	HRT (h)	OLR ($kg.m^{-3}.d^{-1}$)	COD_{diss} SLR ($g.m^{-2}.d^{-1}$)	COD_{diss} SRR	N SLR ($g.m^{-2}.d^{-1}$)	N SRR
TF(S1+2)	3.8	1-4	5-20	2-10	0.5-3	0.32
RBC(S1)	0.24	4-58	40	20	12	0
RBC(S1)	2.4	4.3	40.5	26.8!	3.1	1.1
RBC(S2)	2.8	0.57-2	5	2	3	2.4
SF(S1)	5.9	0.88	-	-	1.30	0.56
SF(S1)	10.3	0.50	1.96	1.04	0.74	0.56

Table 16. Characteristic values of the surface loading and removal rates of the attached growth systems used for UASB-effluent posttreatment.(! = COD^{tot}).

It can be concluded that RBC installations are more effectively removing organic matter than TF and SF. At HL values of approximately 0.3 h^{-1} (HRT = 3.3 h) TF and SF remove 50% of COD_{tot} but RBC remove about 70%. Even at much smaller retention times (0.24 h) RBC show 40 - 80% COD_{tot} removal (See table 12). This is probably due to a better contact between the UASB-effluent and the biofilm and higher oxygen concentrations (oxygen limitation) in the film of RBC.
The higher efficiency of the RBC probably also holds for nitrification. RBC is more effective than TF: at HRT = 2.8 h in the second stage of the RBC a surface N-removal rate of 2.4 $g.m^{-2}.d^{-1}$ and a 70 - 90% NH_4-N removal was found. In the two stage trickling filter a much lower surface removal rate and treatment efficiency for NH_4-N were observed. The above data of the single stage SF are not conclusive as to the efficiency of nitrification. The NH_4-N surface removal rate at HRT 5.9 and 10.3 h are equal which indicates oxygen limitation. Schlegel (1988) found 1 - 3 $g.m^{-2}.d^{-1}$ N_{Kj} removal during treatment of an industrial secondary effluent at concentrations (in the reactor) of BOD < 5, COD < 80 and N_{Kj} < 20 $mg.l^{-1}$.
The need to first dissolve oxygen by bubble aeration of the mixed liquor is a draw-back of the submerged filter as compared to the TF and the RBC.

6. Synopsis of intensive methods for the posttreatment of UASB-effluent

In order to review some current methods for posttreatment of UASB-effluent and to assess their efficiency and general usefulness the data from preceding sections are summarized in table 17.

			Treatment efficiencies				
Method	HRT (h)	OLR $kg.m^{-3}.d^{-1}$	SS (%)	COD_{tot} (%)	NH_4-N (%)	N_{kj} (%)	FC (%)
Sediment	0.25	-	53	31	-	-	-
	0.5	-	51	30	-	-	-
	1.0	-	61	30	-	-	-
AF	6.7	0.74	44	27	low	low	low
	9.7	0.66	72	31	low	low	low
	24	-	-	-	-	-	60
PBC	2.4	2.7	42	15	-	-	low
AS	3.6	-	55	59	45	55	-
	5.4	0.8	37	46	88	79	-
	10.5	-	96	73	73	-	-
TF(S1+2)	3.8	1-4	-	50	-	30	low
RBC(S1)	0.24	4-58	-	40-80	0	-	0
RBC(S1)	2.4	4.3	66	67	35	-	-
RBC(S2)	2.8	0.57-2	-	-	70-90	-	-
RBC(S1+2)	-	-	-	-	-	-	90-95
RBC(S1+2)	3.5	-	-	72	73	60	-
SF(S1)	10.3	0.50	-	61	75	-	95-99
SF(S1)	5.9	0.88	-	-	44	-	95-97
Cascade	very sh.	-	-	14(**)	-	-	-
AL	6	0.87	-	38(**)	3	0	-
	24	0.22	-	33(**)	23	17	-
	96	0.05	-	20(**)	63	58	-

Table 17. Synopsis of treatment efficiencies of various treatment methods applied to UASB-effluent. Treatment efficiencies are given as a function of hydraulic retention time and organic loading rate.

Treatment efficiency data are based on unfiltered influent and effluent samples from final settlers unless otherwise mentioned.

(*) Experiment conducted with septic tank effluent.
(**) based on analysis of filtered samples.
very sh. = very short (only some minutes).
AF = anaerobic upflow filter (incl. final settler)
AS = activated sludge suspended growth system (incl. final settler)
AL = aerated lagoon (completely mixed) without settler (data borrowed from M. Alam, 1991).
TF = trickling filter (incl. final settler)
RBC= rotating biological contactor installation (incl final settler)
SF = submerged aerated filter (without final settler)

The indicated treatment efficiencies are related to:
S1 = one stage posttreatment or the first stage of a two stage system;
S2 = the second stage of a two stage system;
S1+2 = a two stage system.

Discussion on the synopsis of treatment methods

Removal of suspended solids
The SS content of UASB-effluent may vary much.
Under normal circumstances (TSS = appr. 60 $mg.l^{-1}$) 15-30 % of the UASB-effluent TSS may be removed by a simple sedimentation process. When much higher TSS concentrations are found, as is the case during incidental sludge wash-out from the UASB-reactor, much higher removal efficiencies are found. When the process of sedimentation is combined with soluble COD removal in anaerobic upflow filtration applying hydraulic retention times of 2 - 10 h the TSS-removal efficiencies are enhanced to 40 - 70 %.

Removal of COD
It was shown that anaerobic processes do not effectively treat UASB-effluent.
An advanced COD-removal and partial nitrification can be brought about by aerobic activated sludge processes, such as the suspended growth activated sludge installation, the trickling filter, the submerged aerated filter and the biorotor.
Comparing the suspended and attached growth processes it turns out that the conventional activated sludge process and the biorotors are the most effective in COD removal and nitrification. In an activated suspended growth process an 80% N_{KJ}-oxydation can be achieved at sludge loading rates of 0.2 g COD/g sludge d.m..day. This corresponds to a hydraulic retention time of 5 -6 h. A two stage biorotor installation shows 70% COD_{tot} and 80% NH_4-N removal at a total HRT of about 3 h at a COD-SLR of 40 $g.m^{-2}.d^{-1}$ in the first stage and a N-SLR of 3 $g.m^{-2}.d^{-1}$ in the second stage.
Suspended growth processes as compared to attached growth processes have the disadvantage of bulking sludge problems. Especially under tropical conditions denitrification in the secondary sedimentation tank may impair the sludge settling.
The treatment efficiencies of trickling filters as a posttreatment method for UASB-effluent in both simple single pass systems (COD) and in the recirculated systems used by Bovendeur & Klapwijk are rather low as compared to biorotors (see subsection 5.3.).

NH_4-N removal
The main NH_4-N removal process is nitrification. Efficient nitrification of UASB-effluent can be obtained in activated sludge installations at a relatively low COD loading rate (< 0.3 g COD/g sludge d.m..day) and a sludge age higher than about 10 days.
Of the attached growth systems tested the two stage biorotor turned out to be more effective than the trickling filter. More data should be obtained about the sewage treatment in combinations of a UASB-reactor and a submerged filters.

Disinfection
Fecal coliform removal from wastewater is mainly brought about by adsorption in biomass and by processes of bacterial competition and protozoal action. Further, FC removal needs sufficient contact time between wastewater and the active bacterial mass.
Posttreatment methods lacking these conditions, such as the sedimentation and anaerobic filtration (low adsorption, bacterial competition and protozoal activity), trickling filters and cascades (low contact time) show relatively poor FC-removal.
Aerobic processes having sufficient contact time and an actively adsorbing and growing flora show good FC-elimination.
The removal of pathogenic bacteria and viruses in various sewage treatment processes has been reviewed by Feachem et al.(1983). Aerobic suspended growth systems generally show a good removal (90-99%). This study showed similar results for the submerged filter (95-99%). The performance of the two stage rotating biological contactor is less satisfactory. As for the attached growth methods the collection of more data on pathogen removal is required.

7. Appropriate sewage treatment in tropical countries

7.1. The appraisal of sewage treatment methods

The treatment efficiencies of several combinations of anaerobic pretreatment and aerobic posttreatment were presented above.
As anaerobic treatment of domestic wastewater will be especially useful in tropical climates an effective posttreatment method should be appropriate to the conditions of tropical and often underdeveloped countries as well. This means that these methods should be characterized by low investment and maintenance costs, high treatment efficiencies especially for pathogenic organisms and easy process handling.

In table 18 a qualitative appraisal is given of various combinations of anaerobic and aerobic methods with special reference to the factors of importance to developing countries.

Legend to the first row of table 18:

a. Hydraulic retention time
b. Organic matter removal
c. Nitrification
d. Fecal E. coli removal
e. Saving land surface
f. Technical simplicity
g. Resistance against sludge bulking
h. Simplicity of surveillance
i. Investment cost
j. Operation cost

	a.	b.	c.	d.	e.	f.	g.	h.	i.	j.
UASB	6h	!	--	--	++	+	+	+	++	+
UASB + lagoon	30h	+/-	--	-	+/-	+	+	+	+	+
UASB + lagoons	15d	+(*)	+/-	+	--	+	+	+	--	+
UASB + aer. lagoon	24h	+/-	--	-	+/-	+/-	+	+/-	+	+/-
UASB + conv. AS	10h	++	+	+/-	+	-	-	-	-	-
UASB + Bardenpho	12h	++	++	+/-	+	-	-	--	--	--
UASB + 2 stage TF	12h	+	--	-	+	-	+	-	--	-
UASB + 2 stage RBC	9h	++	+/-	+/-	++	+/-	+	+/-	-	+/-
UASB + 2 stage SF	12h	++	+/-	+/-	+	-	+	-	--	-

Table 18. Qualitative appraisal of various sewage treatment methods starting from UASB-pretreatment.
(!) UASB $COD_{tot,eff}$ = ca. 200 $mg.l^{-1}$.
(*) occasionally high $COD_{tot,eff}$ values occur as a consequence of algal bloom.

An explanation of the marks (++,+, +/-,- and --) is given in tables 19 and 20.

Mark	Org. Matt. ($COD_{tot,eff}$) ($mg.l^{-1}$)	Nitrification (% NH_4-N_{eff} converted)	Nutrients ($N_{tot,eff}$) ($mg.l^{-1}$)	FC (n/100 ml in eff.)
++	< 50	> 90	< 5	$< 10^3$
+	$50 \leq c < 100$	$80 \leq c < 90$	$5 \leq c < 10$	$10^3 \leq c < 10^4$
+/-	$100 \leq c < 150$	$70 < c < 80$	$10 \leq c < 15$	$10^4 \leq c < 10^5$
-	$150 \leq c < 200$	$60 < c < 70$	$15 \leq c < 20$	$10^5 \leq c < 10^6$
--	>200	<60	≥ 20	$> 10^6$

Table 19. Legend to the appraisal of treatment efficiencies

Mark	Technical simplicity	Ease of surveillance
++	no grit and sludge removal nor pumps, aerators etc.	unskilled labour
+	grit and sludge removal only	on the job training of unskilled labour
+/-	grit and sludge removal aerators or pumps facultatively	on the job training of labour with basic mechanical skills
-	grit and sludge removal aerators and pumps, sludge recirculation	labour with specialized training in wastewater treatment
--	grit and sludge removal pumps, aerators, dosage of chemicals	labour with advanced specialized training in wastewater treatment

Table 20. Legend to the appraisal of technical qualities of the treatment methods

Mark	Land saving (HRT, h)	Investment cost (x cost UASB)	Operation cost (x cost UASB)
++	<10	≤1x UASB	≤1x UASB
+	$10 \leq c < 20$	$1 < c < 1.5$ x	$1 \leq c < 1.5$ x
+/-	$20 \leq c < 40$	$1.5 \leq c < 2$ x	$1.5 \leq c < 2$ x
-	$40 \leq c < 80$	$2 \leq c < 2.5$ x	$2 \leq c < 2.5$ x
--	>80	>2.5 x	>2.5 x

Table 21. Explanation of appraisal of land saving and costs.
Land saving is related to hydraulic retention time:land is saved at small HRT.
Investment and operation costs of the total treatment installation are related to the costs of an UASB-reactor.

All combinations of treatment methods mentioned in table 18 could be applied under the following conditions:

* Off-site sewage treatment
* Preliminary treatment by means of a screen and grit removal
* (Sub)tropical climate (Temp. > 15° C)
* Application of some form of excess sludge treatment

7.2. Conclusions on appropriate sewage treatment methods

Table 18 offers an appraisal of the relative use value of the UASB reactor and eight methods of UASB pretreatment and aerobic posttreatment.
Among these methods three groups may be distinguished.
First the UASB reactor could be followed by a lagoon system having a short (24 h) or relatively long hydraulic retention time (15 days).
Aerated lagoons do not seem to have an essentially higher treatment efficiency than lagoons without aerators (Alam, 1991). Secondly, an UASB reactor may be followed by a suspended growth activated sludge system; this may be a conventional system having an HRT of 4 to 5 h and providing nitrification as described above in subsection 4 or a multicell Bardenpho system for nitrification and denitrification.
The third group consists of an UASB reactor followed by an attached growth system. The attached growth systems distinguished are trickling filters, rotating biological contactors and submerged filters. The attached growth systems considered are two stage systems having a first stage for organic matter removal and a second stage for nitrification.
All systems except the UASB plus lagoon system having a long HRT produce excess sludge which should be treated on sludge drying beds or recycled to the UASB reactor.
In the first three columns the treatment efficiencies of the nine systems are compared. This comparison was made on the basis of literature (Van Buuren, Specker, 1988) and data collected in table 17.

Treatment efficiencies
It can be concluded that all UASB plus posttreatment systems produce a satisfactory (+/-, + or ++) removal of organic matter i.e. at least 70% COD_{tot} removal. In lagoon systems especially at retention times longer than 1 day algae growth occurs and the $COD_{tot,eff}$ may attain higher values.
Nitrification and pathogen removal are reasonable (+ and +/-) in the suspended growth and attached growth systems except in the trickling filters which usually give less satisfactory results probably due to hydraulic problems. High levels (99 - 99.9% FC removal) of fecal E. coli removal are only achieved using multicell lagoon systems having long hydraulic retention times. High levels (++) of nutrient removal (N and P) may be expected only in the Bardenpho process which is provided with anoxic cells for denitrification.

Technical simplity
Technical simplicity of a treatment station is related to the need (occasionally or continuously) of mechanical devices like pumps , aerators, and scrapers in sedimentation tanks. In the judgment of technical simplicity the behaviour of the treatment station at failure of pumps, aerators etc. should play a role. An aerated lagoon is regarded more simple than a trickling filter as the former system still purifies on aerator failure, but for all more advanced systems the pumps and aerators are indispensable.

Grit and sludge removal are seen as technically simple (+) as they can be carried out by manual labour or using gravity (in the case of UASB reactors). For sludge removal occasionally pumps may be used.
The use of aerators in order to enhance treatment efficiency as is the case in aerated lagoons is regarded as moderately simple (+/-). Simplicity is less (- and --) in the case of all more advanced treatment methods which make use of aerators and pumps for influent supply, effluent drainage, backwashing or recirculation.

The conventional activated sludge process and the Bardenpho process are marked by (-) as a consequence of the need of aeration, sedimentation and sludge recirculation. Contrary to trickling filters submerged filters can be operated using gravity flow but they need regular filter backwashing. Both methods are judged as technically rather complicated (-) because of indispendable use of aerators and pumps.
The use of RBC is technically simple. RBC can be operated by gravity flow, they do not need pumps nor aerators. In some cases aerators are mounted below the rotating contactors in order to 'blow off' excess sludge. It seems that the construction of big scale RBC is not easy. Special measures should be taken to prevent clogging and overloading of the contactors by excessive sludge. The final judgment for RBC therefore is (+/-).

Operating skills
The skills of employees for routine maintenance on the wastewater treatment station are appraised under the heading of 'ease of surveillance'.
Special skills for starting-up of reactors are not included here.
It is shown that routine maintenance of UASB-reactors and UASB reactors followed by lagoons does not require specialized labour.
The suspended and attached growth systems except the RBC do require more specialized labour to take care of sludge and effluent recirculation, filter backwashing, control of bulking sludge, etc. RBC seem more easy to maintain than most other methods as they do not include sludge or effluent recycling, but care should be taken to avoid contactor overloading.

Sludge bulking
Resistance to bulking sludge problems are indicated by (+) for all methods which do not depend on sludge recirculation and by (-) for the suspended growth methods in which sludge bulking may occcur and interfere with the purification process.

Land space
The use of land space is related to hydraulic retention time of the treatment method.

Investment and operation costs
The investment and operation cost of the various methods are all related to the costs of an UASB-reactor.
The per capita capital costs of an UASB reactor depend on the capacity and the volumetric load.
The capital costs in Brazil are 10 US \$/capita for a 120 m^3 installation at a volumetric load of 200 litres/capita.d and HRT 4 h, corresponding to 3,600 p.e. (Vieira, 1988). The cost of a 40,000 p.e. installation in Colombia is estimated at US \$ 375,000, i.e. US \$ 113/ m^3 reactor volume and 9.4 US \$/capita at a daily load of 390 litre/capita (Rincon and Collazos, pers. comm., 1990).
The relatively high costs of an UASB plus lagoon system (15 days HRT) is due to the land acquisition costs which are assumed to be 10 US \$/m^2.

The operation costs of an UASB reactor are estimated 0.4 US \$/capita.yr (Vieira,1988)

Appropriateness of treatment methods
Table 18 may help a discussion on appropriate combinations of anaerobic (UASB) pretreatment and aerobic posttreatment methods. As mentioned above only few full-scale experiments of the various treatment methods have been carried out. One should be warned, therefore, that the appraisal is only indicative. Further research and development may hopefully give us more appropriate sewage treatment techniques.

The appropriateness of a technical system depends on a score of local factors. The required effluent quality is a starting point in the selection of a posttreatment method. When e.g. only 90% BOD removal at little cost is required an UASB plus small posttreatment lagoon is the appropriate technology choice.
When more than 99% faecal E. coli removal is stricly required (+ and ++) large posttreatment lagoons or an suspended growth activated sludge system could be chosen.

The application of lagoon systems having long retention time requires relatively large surfaces of flat land. One might argue that if sufficient flat land is available the use of a relatively expensive UASB reactor as a first treatment stage is unnecessary and the treatment system of choice would be a conventional lagoon system.

When land acquisition costs are high or flat land is not available intensive activated sludge systems remain the only possible choice when nitrification and pathogen removal are required.
Suspended growth activated sludge systems and more specifically the more complicated nitrification/denitrification system (Bardenpho) may give the required effluent quality but the system may on the other hand be inappropriate in most situations in Third World countries as a consequence of high costs, the need of skilled operators and technical problems to be expected. The attached growth systems discussed all have strong and weak points. More demonstration and full-scale experiments are needed to well assess the applicability of attached growth systems.

Table 18 shows that aerated lagoons and rotating biological contactors are treatment methods having a high average degree of appropriateness. They combine relatively good treatment efficiencies, technical simplicity and acceptable costs. Aerated lagoons have the draw-back of limited nutrient removal.
The Rotating Biological Contactors seem sturdy devices with a high treatment efficiency and limited operation costs. The construction however, especially of big units, does not seem simple and costs may be prohibitively high, especially in developing countries.

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9. List of abbreviations

AF	Anaerobic Filter	
AL	Aearated Lagoon	
AS	Activated suspeded growth process	
B_x	Sludge Loading Rate	($kg.kg^{-1}.d^{-1}$)
B_{COD}	COD sludge loading rate	($kg.kg^{-1}.d^{-1}$)
B_N	Ammonia sludge loading rate	($kg.kg^{-1}.d^{-1}$)
BOD	Biochemical Oxygen Demand	($mg.l^{-1}$)
DO	Dissolved oxygen	($mg.l^{-1}$)
COD	Chemical Oxygen Demand	($mg.l^{-1}$)
DCOD	Direct COD	($mg.l^{-1}$)
EC	Electrical Conductivity	(mmho)
EGSB	Expanded Granular Sludge Bed	
FC	Faecal Coliform count	($n.(100\ ml)^{-1}$)
HL	Hydraulic Load	($m^3.m^3.d^{-1}$)
HRT	theoretical Hydraulic Retention Time HRT was calculated by dividing the real volume of the posttreatment reactor(s) (excluding settling tank(s) by the influent flow rate. HRT does not always express the real residence time of the liquid.	(d)
HSLR	Hydraulic Surface Loading Rate	($m^3.m^{-2}.h^{-1}$)
MLSS	Mixed Liquor Suspended Solids	($mg.l^{-1}$)
MLVSS	Mixed Liquor Volatile Suspended Solids	($mg.l^{-1}$)
OC	Oxygenation Capacity	($mg.l^{-1}.h^{-1}$)
OLR	Organic Loading Rate OLR is based on COD_{tot}, unless otherwise mentioned	(kg $COD.m^{-3}.d^{-1}$)
PBC	Pebble Bed Clarifier	
RBC	Rotating Biological Contactor	
SF	Submerged Filter	
SLR	Surface Loading Rate	($g.m^{-2}.d^{-1}$)
SRR	Surface Removal Rate	($g.m^{-2}.d^{-1}$)
SV	Sludge Volume	($ml.l^{-1}$)
SVI	Sludge Volume Index	($ml.g^{-1}$)
TF	Trickling Filter	
TOC	Total Organic Carbon	($mg.l^{-1}$)
TSS	Total suspended solids	($mg.l^{-1}$)
UASB	Upflow Anaerobic Sludge Blanket	
VFA	Volatile Fatty Acids	($mg.l^{-1}$)