Combined Anaerobic/Aerobic Secondary Municipal Wastewater Treatment: Pilot-Plant Demonstration of the UASB/Aerobic Solids Contact System

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Abstract: Anaerobic pretreatment followed by aerobic posttreatment of municipal wastewater is being used more frequently. Recent investigations in this field using an anaerobic fluidized bed reactor/aerobic solids contact combination demonstrated the technical feasibility of this process. The investigation presented herein describes the use of a combined upflow anaerobic sludge bed (UASB)/aerobic solids contact system for the treatment of municipal wastewater and attempts to demonstrate the technical feasibility of using the UASB process as both a pretreatment unit and a waste activated sludge digestion system. The results indicate that the UASB reactor has a total chemical oxygen demand removal efficiency of 34%, and a total suspended solids removal efficiency of about 36%. Of the solids removed by the unit, 33% were degraded by the action of microorganisms, and 4.6% accumulated in the reactor. This low solids accumulation rate allowed operating the UASB reactor for three months without sludge wasting. The long solids retention time in this unit is comparable to the one normally used in conventional sludge digestion units, thus allowing the stabilization of the waste activated sludge returned to the UASB reactor. Particle flocculation was very poor in the UASB reactor, and therefore, it required postaeration periods of at least 100 min to proceed successfully in the aerobic unit. Polymer generation, which is necessary for efficient biological flocculation, was practically nonexistent in the anaerobic unit; therefore, it was necessary to maintain dissolved oxygen levels greater than 1.5 mg/L in the aerobic solids contact chamber for polymer generation to proceed at optimum levels. Once these conditions were attained, the quality of the settled solids contact chamber effluent always met the 30 mg BOD/L, 30 mg SS/L secondary effluent guidelines.

DOI: 10.1061/(ASCE)0733-9372(2007)133:4(397)

CE Database subject headings: Wastewater management; Anaerobic treatment; Aerobic treatment.

Introduction and Background

Different anaerobic technologies have been applied for the treatment of domestic wastewater, providing good efficiencies at low hydraulic retention times (HRTs). One technology is the upflow anaerobic sludge bed (UASB), which is the most frequently used reactor in full-scale installations for the anaerobic treatment of industrial and domestic wastewater (Field 2003). According to Field (2003), in a recent survey conducted by Franklin (2001), 1,215 full-scale high rate anaerobic reactors have been built for the treatment of industrial effluents since the 1970s throughout the world. An overwhelming majority (72% of all plants) of the existing full-scale plants are based on the UASB or expanded granular sludge bed design concept developed by Lettinga et al. (1980) in The Netherlands.

Unfortunately, anaerobic biological treatment alone cannot achieve the performance levels required for direct discharge in receiving streams. However, it can be employed as a cost-effective pretreatment ahead of aerobic treatment. The marriage of these processes brings two advantages: Simple design technology and minimization of sludge production (Jenicek et al. 1999).

Van Haandel and Lettinga (1994) suggested the combination of the UASB process with the conventional activated sludge as a polishing unit for the treatment of domestic sewage, and cite several advantages of this technology, including the fact that there is no need to have separate sludge digestion for the waste activated sludge.

Alem Sobrinho et al. (1995) investigated an UASB-activated sludge system, receiving an influent composed of 90% of industrial wastewater. The UASB reactor achieved removal efficiencies around 70% for chemical oxygen demand (COD) and 80% for biochemical oxygen demand (BOD). The aerobic system was unstable and subjected to filamentous bulking. On stable periods, the removal efficiencies of the activated sludge system alone averaged 42% for COD and 63% for BOD. Alem Sobrinho attributed the instability to the high percentage of industrial wastewater flow.

Pontes et al. (2003) studied the performance of an UASB reactor used for combined treatment of domestic sewage and excess sludge from a trickling filter. No adverse effects on the performance of the UASB reactor due to the return of the aerobic sludge produced in the trickling filter were reported. On the contrary, the COD results indicated better removal efficiencies.

Research performed at the University of New Orleans has determined that in the New Orleans Metropolitan area most of the organic matters present in domestic sewage is particulate material

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Note. Discussion open until September 1, 2007. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on June 24, 2005; approved on August 28, 2006. This paper is part of the *Journal of Environmental Engineering*, Vol. 133, No. 4, April 1, 2007. ©ASCE, ISSN 0733-9372/2007/4-397–403/$25.00.

that can be removed by flocculation (La Motta et al. 2004). In the case of Jefferson Parish, La., more than 80% of the total chemical oxygen demand (TCOD) is in the form of organic particles, while less than 20% is truly dissolved organic material. Therefore, removal of colloidal COD and suspended solids cannot be achieved unless there is successful biological flocculation and sedimentation of the well-formed floc particles (La Motta et al. 2004). Unfortunately, as demonstrated by Luque (2005), due to the low or insignificant extracellular polymer production by anaerobic bacteria, flocculation efficiency is generally poor in anaerobic environments and, unless there is good granular sludge formation, the effluent of anaerobic units such as the AFBR and the UASB, when processing domestic sewage, is loaded with suspended solids. Therefore, additional treatment, involving flocculation of this large amount of solids becomes necessary.

Corzo (2001) and Bustillos (2002) demonstrated the possibility of using short aeration times to promote biological flocculation of the effluent from anaerobic units, such as the AFBR and the UASB, processing domestic sewage. Based on the idea originally proposed by Van Haandel and Lettinga (1994) and de Sousa and Foresti (1996), that was later tested by Goncalves et al. (1999), the waste activated sludge was returned to the anaerobic reactor for solids digestion.

The investigation presented herein describes the use of a combined UASB/aerobic solids contact (UASB/ASC) system for secondary treatment of municipal wastewater. In order to compete favorably with the conventional activated sludge process, the proposed treatment train would include the following units: grit chamber, fine screen, short-HRT UASB, short-HRT aerobic solids contact unit, final clarifier, sludge recycling to the aerobic unit, and waste activated sludge digestion in the UASB reactor. The primary clarifier and the conventional sludge digestion unit, therefore, would be eliminated. No attempt would be made to optimize the UASB performance by using the long HRTs usually recommended (6 or more hours). As long as the final effluent from the ASC chamber meets secondary effluent criteria, and as long the waste activated sludge can be digested in the UASB unit, the treatment objective would be achieved.

A pilot-plant comparison of the AFBR and the UASB technologies for the anaerobic pretreatment and waste activated sludge digestion will be presented in a forthcoming paper including a quantification of the SS removal and accumulation in the system, and determination of the degree of stabilization of solids in the anaerobic unit. This comparison demonstrated that the UASB reactor offers significant advantages over the AFBR, such as much lower energy consumption for bed fluidization, much higher sludge stabilization rates, and lower solids accumulation in the sludge bed. For this reason, the UASB/ASC combination was selected for this research, as described in the following.

Experimental Setup and Design

The University of New Orleans Schlieder Urban Environmental Systems Center has been conducting important research aimed at determining the feasibility and efficiency of combined anaerobic/aerobic treatment of domestic wastewater. The University of New Orleans pilot-plant facility was located at the Marrero Wastewater Treatment Plant, Marrero, La.

The pilot plant processed 10 m^3/day of effluent coming from the Marrero full-scale plant grit chamber effluent splitter box. This total flow rate was divided into two streams, one feeding the aerobic pilot plant (La Motta et al. 2004), and the other one feeding the anaerobic/aerobic pilot plant discussed herein. The anaerobic/aerobic pilot plant consisted of the following components: Rotating screen, UASB reactor or AFBR reactor, aerated solids contact chamber, and secondary clarifier. The excess sludge was pumped from the clarifier to the UASB unit. A schematic representation of the treatment unit train, when using the UASB reactor, can be seen in Fig. 1. A similar investigation was carried out using an AFBR instead of the UASB reactor, and will be reported in a forthcoming paper.

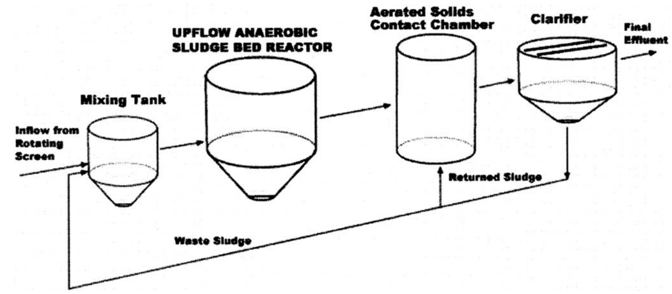

Fig. 1. Pilot-plant diagram

The pilot-plant influent was taken from the full-scale grit chamber effluent splitter box to a 0.5 mm gap wedge wire fine screen. As the wastewater entered the rotating cylindrical screen, solids larger than 0.5 mm rode over the top of the screen and were removed by a blade assembly located along its upper part.

The effluent from the rotating screen was pumped out to a distribution tank located on the roof of the pilot plant, provided with an electrical mixer. The screened effluent flowed by gravity from the distribution tank into a conical-bottom tank with two inlets, one from the distribution tank with the screened wastewater, and the other from the clarifier with the waste sludge. A submersible pump located inside the conical tank mixed the two water streams.

A diaphragm pump fed the anaerobic reactor with the liquid mixture. The mixing tank fed either the AFBR or the UASB reactor at a flow rate of 125 L/h ± 10% with individual diaphragm pumps. The effluent of the anaerobic units was either fed to the aeration unit or it was discarded, depending on the respective experimental phase.

The UASB reactor was a cylindrical polyethylene tank with a 60° conical bottom. The tank had a diameter of 0.86 m and a total height (conical section plus cylindrical section) of 1.16 m. The volume of the 0.52 m high frustum of a cone was 0.129 m^3, and the useful volume of the cylindrical section, 0.46 m high, was 0.267 m^3. Five ports are arranged along the UASB reactor height allowing samples to be taken. Fig. 2 shows a schematic diagram of the UASB reactor.

The UASB unit is fed from the bottom of the reactor to ensure fluidization of the sludge bed. An internal recirculation system was used to maintain an upflow velocity of around 1 m/h in the cylindrical section.

The aeration chamber was fed by gravity either from the AFBR, when the HRT was from 20 to 100 min (Bustillos 2002), or from the UASB, when the HRT was 120 and 180 min (Luque 2005). The aeration chamber consisted of several different polyethylene tanks equipped with a fine bubble diffuser system at the bottom. The volume of the tank used for 40 and 20 min retention times was 114 L, and the volume of the second tank, used to study the effects of 100, 80, and 60 min hydraulic retention times, was 202 L (Bustillos 2002). The operating volume of these two

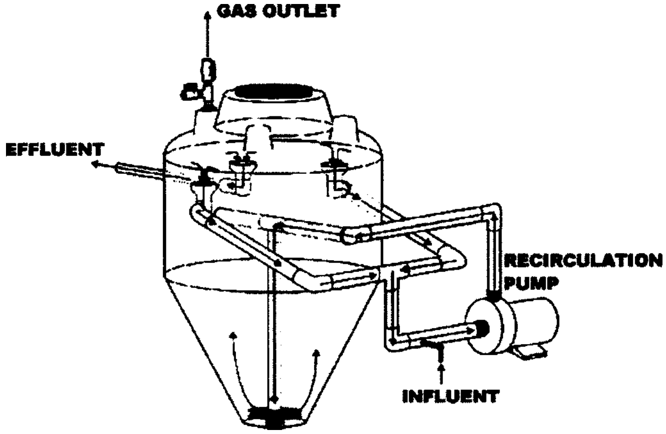

Fig. 2. Upflow anaerobic sludge bed reactor

ASC chambers was adjusted by using an effluent overflow weir while keeping a constant flow rate to observe the effects of the HRT on the monitoring parameters, total COD, filtered COD, and total suspended solids of the mixed liquor supernatant after 30 min settling. The tank volume used to get a 120 min detention time was 240 L, and the reactor volume for the 180 min HRT was 360 L (Luque 2005). Table 1 summarizes the operating characteristics of the ASC chamber.

The blower provided air to maintain an optimum dissolved oxygen concentration and velocity gradient for uniform mixing and flocculation. The airflow rate was varied from 36.8 to 45.3 L/min to maintain a constant velocity gradient around 25 s^{-1}. The recycle flow rate was frequently adjusted between 0.5 and 1.0 of the influent flow rate in order to maintain average MLSS concentrations between 3,800 and 4,100 mg/L.

The clarifier unit consisted of a 280 L polyethylene tank with a conical bottom section. The water from the aeration chamber was discharged into the clarifier tangentially into a center well in order to reduce the inflow energy and to provide optimum condition for flocculation. Additionally, a rotary arm moved by a 1 rpm gear motor scraped the bottom of the conical section to prevent the sludge from compacting.

The clarified effluent left the unit through three PVC pipes located radially along the top of the clarifier, and was discharged into the final effluent line. Part of the sludge retained at the bottom of the unit was recycled to both the ASCC and the screen effluent mixing tank by centrifugal pumps driven by cycle timers.

Table 1. Operational Parameters of the Aerated Solids Contact Chamber

Parameter		Value	
Influent flow rate (L/h)		125±10%	
Mean temperature (°C)		20	
Mean cell residence time (days)		1.5–2.0	
Reactor volume (L)	HRT (min)	MLSS (mg/L)	F/M (kg COD/ day kg VSS)
41.7	20	3,788	8.1
83.3	40	3,952	3.9
125.0	60	3,839	2.7
166.7	80	3,805	2.0
208.3	100	4,134	1.5
240.0	120	2,980	1.5
360.0	180	3,295	0.9

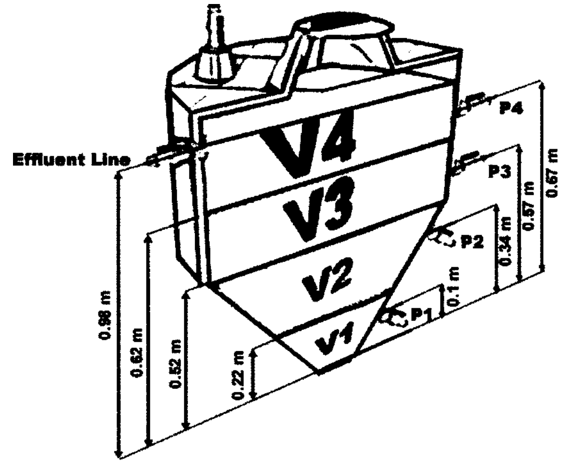

Fig. 3. Location of the sampling ports

The sampling phase was initiated in January 2004 and lasted through November 2004. Samples were taken as often as possible depending on weather and plant operating conditions. After collection, samples were taken to the Environmental Laboratory at the University of New Orleans to be analyzed.

Water samples were collected from four different points: mixing tank (influent), anaerobic reactor effluent, clarifier effluent, and sludge recycle line. The intermittent discharge of sludge from the clarifier to the mixed tank along with a continuous variation of the pilot-plant influent, made it necessary to collect 24 h composite samples instead of grab samples. Two automatic composite wastewater samplers from Global Water were used to collect the water samples (150 mL of sample every 60 min). In order to preserve the samples, sulfuric acid was added to the collection tank to ensure the final pH was maintained below 2 (APHA 1999).

Samples of the anaerobic unit effluent and their supernatants were stored and analyzed separately. The samples were stored in glass bottles of 500 mL each.

Three parameters were measured, namely, TCOD, total suspended solids (TSS), and volatile suspended solids (VSS). The analyses were performed in the Environmental Engineering Laboratories located at the Center for Energy Resources Management (CERM), University of New Orleans.

The TCOD was performed according to Method 5220D of the Standard Methods (APHA 1999). The TSS was run using Method 2040D of the Standard Methods (APHA 1999), and the VSS test was performed using Method 2540E of the Standard Methods (APHA 1999).

The methane concentration, which is an important indicator of the anaerobic activity inside the reactor, was measured during the experimental phase. A portable gas analyzer model LMS manufactured by CEA Instruments, Inc. was used to monitor the methane concentration in the biogas.

Results and Discussion

After several weeks of unsuccessful operation of the UASB reactor, the unit was emptied and cleaned. The reactor was inoculated again with 132.5 L of anaerobic sludge from the digester units of Terrace Avenue Wastewater Treatment Plant, Slidell, La. The rest of the reactor volume was filled with raw wastewater. After two months of continuous operation the plant performance was deemed stable, and sampling commenced (Fig. 3).

The influent to the UASB reactor had an average total COD of 341 mg/L, a TSS concentration of 189 mg/L, and a VSS concentration of 162 mg/L. The UASB reactor was fed at a constant flow rate of 125 L/h ± 10%. The hydraulic retention time (HRT) of the reactor, based on a 0.187 m^3 sludge bed volume, was 1.5 h; the HRT based on the total liquid volume (0.396 m^3) was 3.2 h; the average organic load applied, based on an average influent COD of 341 mg/L and a total liquid volume of 0.396 m^3, was 2.6 kg TCOD/(m^3) (day); the average organic load applied, based on a sludge bed volume of 0.187 m^3, was 5.5 kg TCOD/(m^3) (day); the average solids load, based on an average influent TSS concentration of 189 mg/L and the total liquid volume was 1.4 kg TSS/(m^3) (day); and the average solid load, based on the sludge bed volume, was 3.0 kg TSS/(m^3) (day).

In order to evaluate the performance of the UASB reactor, the UASB mixed effluent TCOD (mg/L) was plotted against the influent TCOD (mg/L). Fig. 4 shows a linear relationship with a coefficient of determination (R^2) of 0.79. A linear regression analysis, forcing the intercept through the origin, generated the following:

$$\mathrm{TCOD}_{\mathrm{Mixed\ Effluent}} = 0.66 \times \mathrm{TCOD}_{\mathrm{Influent}} \quad (1)$$

This equation yields a removal of 34%. This TCOD removed corresponds to organic matter that is converted to CH_4 and CO_2.

A similar performance was observed with regard to the removal of TSS in the UASB unit. Fig. 5 shows a linear relationship between TSS in the influent and mixed effluent of the UASB, with a coefficient of determination (R^2) equal to 0.69. The linear regression equation, forced through the origin, is

$$\mathrm{TSS}_{\mathrm{Effluent}} = 0.64 \times \mathrm{TSS}_{\mathrm{Influent}} \quad (2)$$

This equation yields a TSS removal of 36%.

As presented in Table 2, the average percent removals of TCOD, TSS, and VSS obtained in the UASB based on a completely mixed effluent were 34, 36, and 37, respectively. These results indicate a low performance of the UASB unit compared to typical values reported for UASB reactors treating municipal wastewater using HRT between 6 and 8 h. No efforts were made to optimize the UASB performance because it would have affected the performance of the aeration chamber. As indicated before, the overall treatment objectives were to achieve a final effluent that meets secondary effluent criteria, and to obtain waste activated sludge digestion in the UASB unit.

The average percent removals of TCOD, TSS, and VSS obtained in the UASB based on the settled effluent were 50, 78, and 69%, respectively, and demonstrate that a significant fraction of the effluent TCOD is due to TSS.

Table 2. Average Performance of the UASB

Parameter	No. obs.	Mean	Std. dev.
Influent TSS (mg/L)	40	189	69
Mixed effluent TSS (mg/L)	40	144	74
Settled effluent TSS (mg/L)	42	57	24
Influent VSS (mg/L)	41	162	36
Mixed effluent VSS (mg/L)	42	125	61
Settled effluent VSS (mg/L)	42	50	22
Influent TCOD (mg/L)	40	341	85
Mixed effluent TCOD (mg/L)	42	273	124
Settled effluent TCOD (mg/L)	42	162	70
Fraction of TSS degraded		0.36[a]	
Fraction of TSS removed after settling		0.78[a]	
Fraction of VSS degraded		0.37[a]	
Fraction of VSS removed after settling		0.69[a]	
Fraction of TCOD degraded		0.34[a]	
Fraction of TCOD removed after settling		0.50[a]	
Organic loading, (kg TCOD/m^3 day)		5.5	
Temperature range (°C)		15–30.5	
Mean temperature (°C)		20	
Volume of gas (mL CH_4/L sewage), at 25°C (1 atm)		27.7	
Biogas composition (% methane)		60	

[a]Calculated by doing a linear regression of all observations, forced through the origin, between effluent and influent water quality parameter.

UASB Biogas

Vieira and Sonia (1987) reported that the biogas produced in a full-scale UASB reactor treating domestic sewage had an average composition of 70% methane, 22% nitrogen, and 8% carbon dioxide. Typically, the biogas in an anaerobic reactor treating domestic sewage is about 70–80% methane, and the remainder is made up of a mixture of carbon dioxide, nitrogen, hydrogen, water vapor, and a small fraction of hydrogen sulfide (Van Haandel and Lettinga 1994). In the present research, the biogas produced had an average of 59.8% of methane.

According to Yoda et al. (1985), given the partial pressure of methane in the overlaying gas phase, the amount of methane dissolved in the effluent can be calculated using Henry's law.

Unfortunately, only a few points could be recorded during September and August 2004 on biogas production. Total produc-

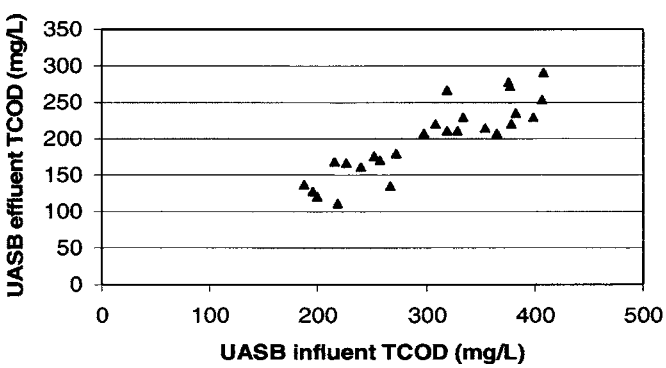

Fig. 4. Relationship between UASB effluent TCOD and influent TCOD

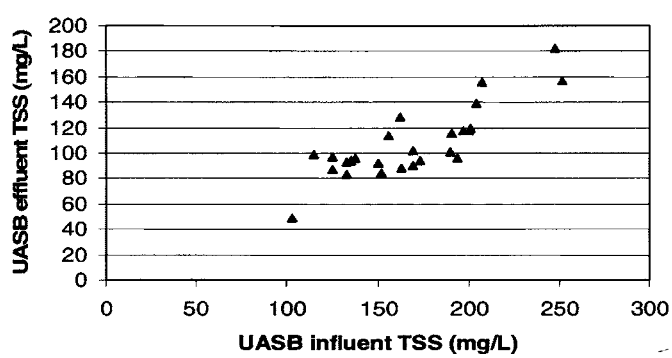

Fig. 5. Relationship between UASB effluent TSS and influent TSS

Table 3. TSS and VSS Concentration Profiles in the UASB (mg/L)

Port	Volume served by port (m^3)	April 7, 2004	May 27, 2004	June 26, 2004	July 22, 2004
1	0.021	36,016 mg TSS/L	41,667 mg TSS/L	38,970 mg TSS/L	36,899 mg TSS/L
		24,390 mg VSS/L	27,955 mg VSS/L	25,576 mg VSS/L	23,411 mg VSS/L
2	0.108	34,340 mg TSS/L	44,764 mg TSS/L	35,014 mg TSS/L	31,054 mg TSS/L
		24,906 mg VSS/L	29,663 mg VSS/L	22,145 mg VSS/L	20,817 mg VSS/L
3	0.058	13,115 mg TSS/L	3,511 mg TSS/L	28,098 mg TSS/L	28,656 mg TSS/L
		9,180 mg VSS/L	2,835 mg VSS/L	18,133 mg VSS/L	18,540 mg VSS/L
4	0.209	724 mg TSS/L	57 mg TSS/L	108 mg TSS/L	8,788 mg TSS/L
		495 mg VSS/L	47 mg VSS/L	86 mg VSS/L	5,636 mg VSS/L
Total mass		5.37 kg TSS	5.93 kg TSS	6.25 kg TSS	7.63 kg TSS
		3.84 kg VSS	3.96 kg VSS	4.00 kg VSS	4.99 kg VSS

tion of CH_4 (temperature range, 25.5–28.5°C, and one atmosphere of pressure) varied from 25.3 to 29.1 mL CH_4/L sewage, of which, as measured in the field, between 5 and 8.7 mL CH_4/L sewage were released to the atmosphere.

Sludge Concentration and Accumulation in the Reactor

The behavior of the sludge bed was analyzed by taking sludge samples from ports at different elevations. Four sampling ports were arranged over the UASB reactor height: $P_1=0.1$ m, $P_2=0.34$ m, $P_3=0.57$ m, and $P_4=0.67$ above the bottom. As shown in Fig. 3, Port P_5 collected the effluent from the UASB reactor.

The results, presented in Table 3, show that in the first solids profile test (April 7, 2004), the concentration of particles in the bed decreases gradually from P_1 to P_4. The second test (May 27, 2004) shows a different distribution of concentrations, with an accumulation of solids in P_2, and a relatively low concentration in P_3. The results of the third test (June 26, 2004) show a homogeneous distribution of the sludge blanket among Ports 1, 2, and 3. The relatively high concentration found in P_3 seems to indicate that the height of the sludge bed was increasing, and the low concentration found in P_4 indicates that the boundary of the sludge bed was somewhere between P_3 and P_4. The results of the last test (July 22, 2004) indicate that the top of the sludge bed was reaching Port 4.

To estimate the sludge hold-up of the reactor, the sludge concentration at each section was assumed to be equal to the concentration found at its port. Table 3 presents the results and shows that between the first and the last solids profile tests (106 days), 2,252 g of TSS and 1,155 g of VSS accumulated in the reactor. It is important to mention that during the 106 days between the first and the last profile, no sludge was removed from the reactor except for sampling. Using the information presented in Tables 2 and 3, based on an average sludge mass of 6.3 kg TSS, and an average TSS concentration of 144 mg/L coming out of the UASB reactor, the solids retention time in this unit was 14.6 days, which is within the recommended range for conventional anaerobic sludge digestion (Metcalf and Eddy 2003).

Mass Balance on Solids in the UASB

To establish a consumption rate and determine the amount of solids degraded inside the UASB reactor a mass balance on solids was performed. Table 4 shows the results: 33% of the TSS fed was degraded by the action of microorganisms and 4.63% accumulated in the UASB reactor. This yields an accumulation rate of 21.25 g/day and degradation rate of 151.6 g/day. Therefore, at the applied solids load of 1.15 kg TSS/m^3 day, 0.38 kg/m^3 day were consumed, and 0.054 kg/m^3 day accumulated in the unit.

Effect of the HRT on the Performance of the Aerated Solids Contact Chamber

As indicated previously, the ASC received AFBR effluent when the HRT varied from 20 through 100 min (Bustillos 2002), and processed UASB effluent when the HRT was 120 and 180 min (Luque 2005). Table 5, and Figs. 6 and 7 summarize the performance data of the aerated solids contact chamber. The dependent variable selected for the plots is the ratio between the outflow stream concentration and the influent stream concentration, to consider the variability of the concentrations in the inflow stream.

As shown in Figs. 6 and 7, there is a slight tendency for the supernatant TCOD and TSS to decrease with increasing HRT values. When the HRT was less than 100 min, the performance of the ASC was erratic, a poor flocculation could be visually observed, and this resulted in a high dispersion of the data. When the HRT was 100 min, the performance of the aerobic unit remained stable, with the effluent TCOD being generally less than 58 mg/L, thus indicating that this HRT should be considered as a minimum value to obtain an effluent of acceptable quality.

When the HRT was around 120 min, the supernatant TCOD ranged from 21 to 61 mg/L (average=46 mg/L), and the supernatant TSS ranged from 7 to 33 mg/L (average=16 mg/L). There were several events of sludge bulking due to growth of filamentous microorganisms, especially when the DO levels in the aeration chamber were less than 1.5 mg/L. Bulking was controlled by adding chlorine to the mixed liquor and by keeping the DO concentration higher than 1.5 mg/L. Additional operating problems, such as power failures, and solids washout during heavy rain events, led to unstable performance of the aerobic reactor at this HRT.

Table 4. Mass Balance Results Based on 106 Days of Reactor Performance without Sludge Wasting

TSS fed to the UASB reactor	48,540 g
TSS accumulated in the UASB reactor	2,252 g
TSS degraded in the UASB reactor	16,065 g
TSS recycled from the clarifier	18,166 g

Table 5. Summary of Experimental Performance Data of the ASC

HRT (min)	Mixed liquor (mg/L) SS	Mixed liquor (mg/L) VSS	ASC influent (mg/L) TSS	ASC influent (mg/L) VSS	ASC influent (mg/L) TCOD	Supernatant (mg/L) TSS	Supernatant (mg/L) VSS	Supernatant (mg/L) TCOD
20	3,560–4,080	1,740–3,840	83–120	77–107	138–224	17–42	17–37	64–128
40	3,960–4,200	2,843–3,810	68–111	59–94	122–269	5–12	4–11	48–190
60	3,771–3,970	2,980–3,713	79–148	76–133	173–282	7–39	4–25	35–261
80	3,563–4,000	3,325–4,440	76–298	66–286	122–282	4–23	3–20	38–264
100	2,680–6,220	2,150–4,380	63–280	51–240	128–256	4–12	3–12	38–67
120	2,108–4,036	1,644–2,930	71–248	54–188	145–449	7–33	5–25	21–61
180	1,994–4,148	1,552–3,034	106–260	90–115	172–344	8–18	6–11	40–59

When the HRT was around 180 min, the performance of the ASC was significantly more stable. The supernatant TCOD was between 40 and 59 mg/L (average=46 mg/L), and the supernatant TSS was between 8 and 18 mg/L (average=12 mg/L).

From these observations, it can be concluded that the minimum HRT for the ASC chamber in the UASB/ASC system should be 100 min. A more stable operation, including better particle flocculation and better sludge settling characteristics, was observed at HRT=180 min.

It is important to state that while the different HRTs were being studied, the properties of the water treated in the aeration basin changed. As an example, when the system operated at high retention times (higher than 100 min) the color of the water was light brown. As the HRT was decreased, the color of the mixed liquid turned to a darker color, although the dissolved oxygen levels were kept greater than 1.5 mg/L. Also observed in the field were significant changes in the settling properties of the flocs. It was observed that when the HRT was changed to 60 min and, subsequently, to 40 and 20 min, the settling velocity of the flocs decreased significantly. This resulted in a high turbidity observed in the supernatants and a low sludge settleability, thus leading to the conclusion that at low retention times the flocculating ability of the bacteria decreases significantly.

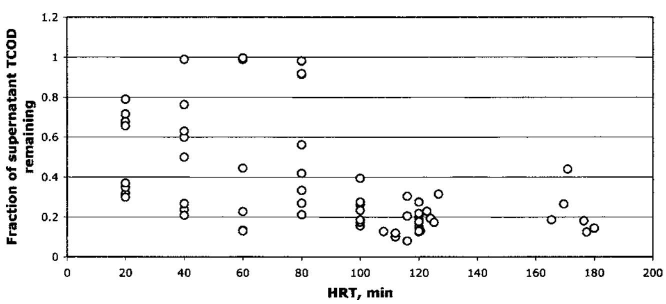

Fig. 6. Fraction of supernatant TCOD remaining in the ASCC versus HRT

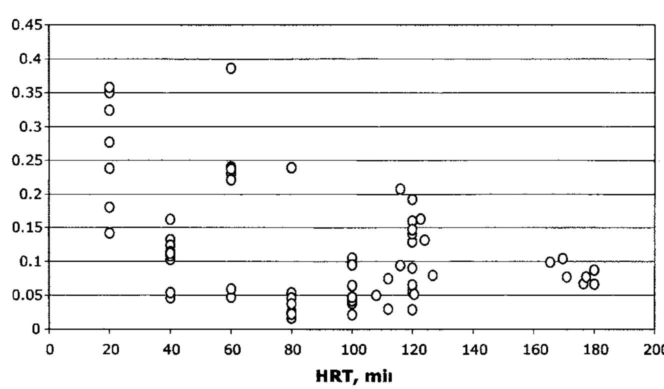

Fig. 7. Fraction of supernatant TSS remaining in the ASCC versus HRT

Conclusions

The following conclusions can be drawn from the research reported herein:

1. The UASB/ASC system is an attractive alternative for municipal wastewater treatment because it can eliminate the need for a separate sludge stabilization unit.
2. Although the UASB reactor had low TSS and TCOD removal efficiencies, the overall UASB/ASC system was capable of meeting secondary-effluent water quality requirements with an overall HRT of at least 5 h.
3. Of the solids removed by the UASB unit, 33% were degraded by the action of microorganisms, and 4.6% accumulated in the reactor.
4. An accumulation rate of 21.25 g/day and degradation rate of 151.6 g/day were observed in the UASB unit. Therefore, at the applied solids load of 1.15 kg TSS/m^3 day, 0.38 kg/m^3 day are consumed, and 0.054 kg/m^3 day are accumulated in the unit. This low accumulation rate would allow operating the UASB reactor without sludge wasting for extended periods (around 3 months)
5. The UASB produces methane gas at an average rate of 6.47 mL of CH_4 per liter of sewage treated. Potentially, this energy could be reused.

Acknowledgments

Financial support for the research reported herein was provided by the University of New Orleans Schlieder Urban Environmental Systems Center through a grant from the U.S. Environmental Protection Agency, and by the Jefferson Parish Department of Sewerage.

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