

Combination of Anaerobic Treatment and Nutrient Removal of Wastewater in Brazil

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ABSTRACT: The limits of the elimination of nitrogen after an anaerobic wastewater treatment are evaluated. First the suitability of conventional batch tests for the determination of the easily degradable COD and for the determination of nitrification and denitrification rates are analysed for anaerobic pre-treated wastewater. In none of the 4 examined different anaerobic reactors provoked an inhibition of nitrification after the anaerobic pre-treatment. The denitrification rate depended significantly on the efficiency of the organic reduction in the reactors, but it was reduced significantly. For analysis of denitrification capacities the determination of easily degradable COD by an oxygen consumption test turned out to be of less significance than a characterisation of the COD composition by the effluent the denitrification test. The second part of the paper discusses alternatives for Ammonia and Nitrogen removal, based on an investment cost analysis and a comparison of operational aspects and costs under Brazilian conditions.

KEY WORDS: anaerobic treatment, nitrification, denitrification, investment costs, operational costs

1 INTRODUCTION

Aim of this paper is a discussion of the possibilities and the needs for Ammonia and Nitrogen removal in warm climates. The paper is the result of a joint venture project between a Brazilian and a German University, the target of which to combine the experiences of anaerobic wastewater treatment in Brazil with experiences of nutrient removal in Germany. Cost and operational aspects have been developed by a Brazilian-German consultancy company. In warm climates the anaerobic treatment of wastewater has proven to be an economic solution for the removal of organic substances (HAANDEL & LETTINGA, 1994) and is a common method as a single step treatment for municipal wastewater in Brazil. Due to the environmental problems caused by insufficiently cleaned wastewater there is an increasing demand for advanced treatment including nutrient removal. In cases of a combined anaerobic - aerobic treatment, the efficiency of the anaerobic reactor influences significantly the design of the aerobic stage. Especially for effluent standards which require nitrification and denitrification the existing experiences are very limited. Therefore the effluent of existing anaerobic WWTP's (UASB, RALF, Lagoon) is evaluated in respect of a nitrogen removal (N, DN). Target of the investigations was to establish a suitable batch-test for characterisation of anaerobic treated wastewater. From aerobic post-treatment after an anaerobic stage conclusions are drawn from an economic and operational view.

2 METHODS

Raw wastewater and wastewater after an anaerobic treatment was analysed in comparison in batch-test. Domestic wastewater of a UASB (1.000 p.e., high loaded, detention time 3,5 h) a RALF (Brazilian variation of a UASB) (10.000 p.e, low loaded, detention time 12 h) as well as an anaerobic lagoon (84.000 p.e, detention time 18 d) and a septic tank (University, detention time 2 d) was used. All batch tests were done in laboratory by 20°C. Always the same activated sludge of a local activated sludge plant with aerobic sludge stabilisation was used. The plant provides nitrification and denitrification.

A special problem was the large distances under subtropical conditions in which the samples had to be transported. Due to the high temperatures of the wastewater the setup of the experiments had to be changed to execution of the batch tests close to the treatment plants. Otherwise even a cooling of the samples and immediate analysis after the transport led to wrong results due to high COD reduction rates during transport. For the analysis of the BOD₅ and TKN it was necessary to freeze the samples. For the characterisation and differentiation of the COD several methods were applied:

- Oxygen consumption test (according to MARAIS & EKAMA, 1976; HULSBEEK,1995) for the determination of highly biodegradable COD = COD_{high deg.} (fig. 1) or S_s
- Denitrification-Test (according to HULSBEEK,1995) for the determination of part of the COD which is accessible for denitrification purposes or COD_{DN}
- Determination of BOD₅ (Respiratory test)

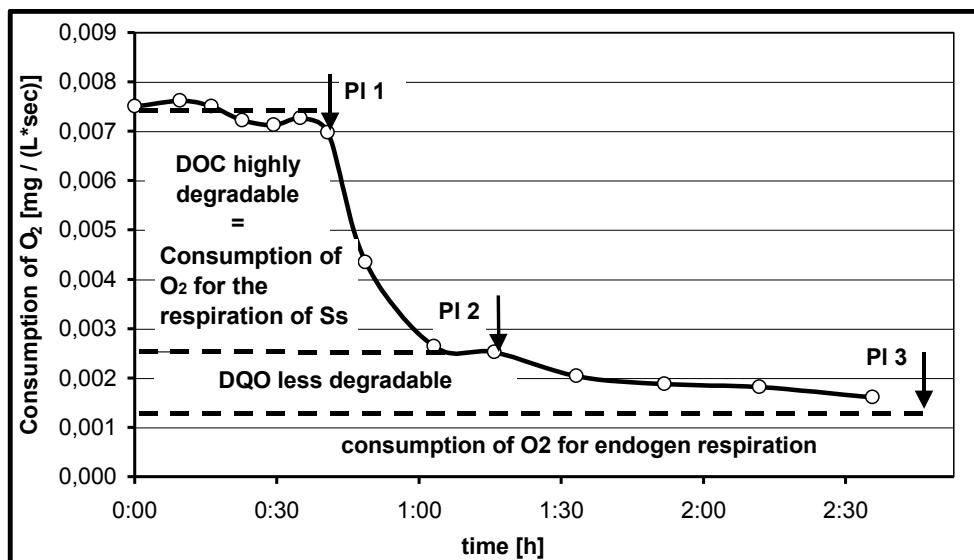


Figure 1: Example for an oxygen consumption test with raw wastewater for determination the part of COD highly degradable (S_s)

S_s = $(1/(1-y)) \cdot \text{Area of } O_{2-SS} \cdot ((V_{\text{wastewater}} + V_{\text{sludge}})/V_{\text{wastewater}})$ with

Y = (Yield) choose according BORNEMANN ET AL. 1998: $0,6 \text{ gCOD}_{\text{Biomass}} / \text{gCOD}_{\text{Wastewater}}$

The activity of nitrificants has to be inhibited, adding nitrification inhibitors. The F/M (food by micro-organism) at the beginning was 0.4-0.6 gCOD /gTSS. For anaerobic pre-treated wastewater with COD concentrations around 200 mg/l only a very small activated sludge quantity could be used, in order to guarantee the necessary F/M value. Applying this method for the effluent, the results were not interpretable with frequency, thereby proving that the oxygen consumption test is not applicable after anaerobic treatment.

The test of denitrification velocity is used for the comparison of the denitrification capacity before and after anaerobic treatment. A solution of KNO₃ was used to raise the Nitrate concentration. Best results were achieved with an initial concentration of at least 50 mg NO₃-N / L. However in some cases this led to partially high NO₂-N concentrations, as al-

ready reported by HULSBEEK (1995). This has to be considered in the calculation. The F/M was adjusted to 0,1 g COD / g TSS. The calculation was done using the following equation:

$$S_s = \frac{1}{1-y} \cdot (2,86 \cdot \Delta \text{NO}_3\text{-N} - 1,72 \cdot \Delta \text{NO}_2\text{-N}) \cdot \frac{(V_{\text{wastewater}} + V_{\text{sludge}})}{V_{\text{wastewater}}}$$

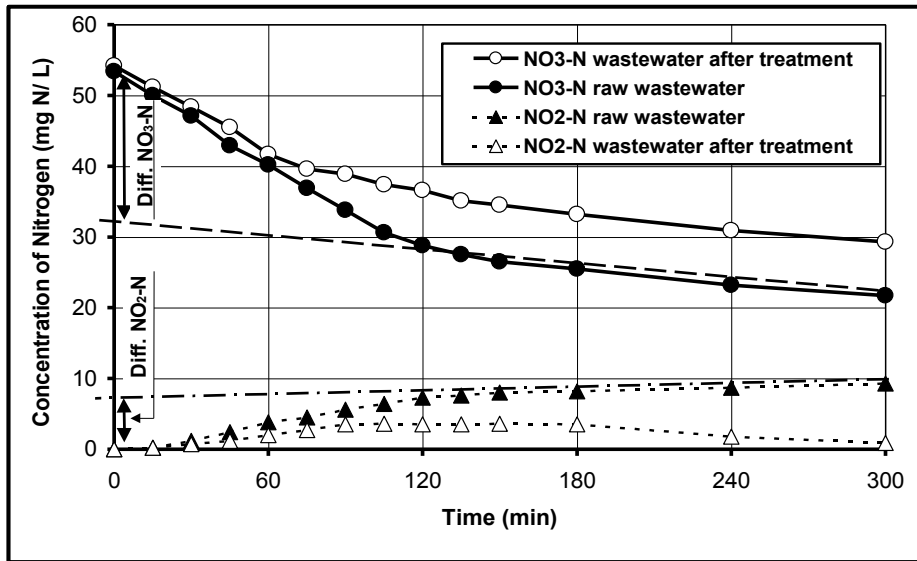
$$Y = (\text{Yield}) \text{ according BORNEMANN ET AL. 1998: } 0,5 \text{ gCOD}_{\text{Biomass}} / \text{gCOD}_{\text{Wastewater}}$$


Figure 2: Example for two tests of denitrification velocity, comparison of denitrification with raw wastewater and wastewater after an anaerobic treatment

In order to identify eventual problems due to inhibition of nitrification after an anaerobic treatment a test of nitrification velocity (HULSBEEK, 1995) was used to compare nitrification activities before and after anaerobic treatment.

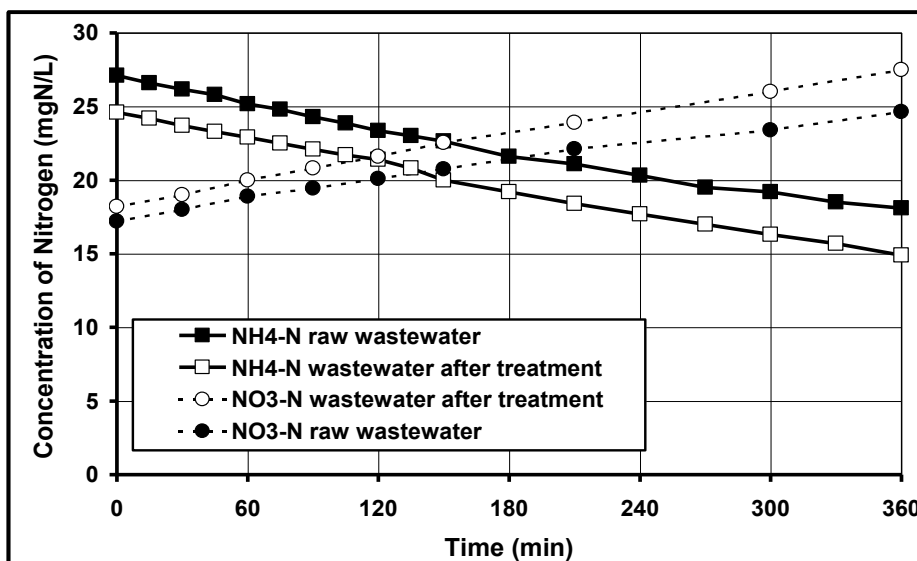


Figure 3: Example for two tests of nitrification velocity; comparison of nitrification with raw wastewater and wastewater after an anaerobic treatment

To obtain nearly the same Ammonia concentration at the beginning of the nitrification tests a NH_4CL -solution was dosed. An initial concentration of 25-30 mg $\text{NH}_4\text{-N/L}$ was found to show best results. The pH value had to be controlled between pH 7.6-7.8, since otherwise it inhibits the nitrification. For this purpose a lime solution was added when nitrification started. The natural buffer capacity both the raw wastewater and the anaerobically treated wastewater was found not to be sufficient for all examined wastewaters. To check the nitrification the increase of the $\text{NO}_3\text{-N}$ concentration was measured. $\text{NO}_2\text{-N}$ did not occur during these experiments, that means, that the nitrification ran completely.

3 RESULTS

Raw wastewater and wastewater after an anaerobic treatment of four different anaerobic treatments was analysed about 3 month. Table 1 shows the results of the different tests.

Table 1: Results of analyses e tests with raw wastewater and anaerobically pre-treated wastewater

Anaerobic treatment parameter	Anaerob. lagoon		UASB		RALF		Septic tank	
	Infl.	Effl.	Infl.	Effl.	Infl.	Effl.	Infl.	Effl.
COD	379	250	699	164	372	267	720	465
BOD ₅	218	78	504	88	188	78	442	240
TKN	48	45	66	61	53	48	22	20
COD _{Ss} aerobic test	35	7	12	6	54	26	94	90
COD _{DN} anoxic Test	120	30	75	36	158	33	280	200
Nitrification Velocity (mg N/gTSS/h)	0,29	0,35	0,44	0,35	0,24	0,24	0,31	0,27
Denitrification Velocity (mg N/gTSS/h) (max)	1,36	0,87	0,69	0,59	1,44	1,01	1,62	1,46

- Depending on the design and the load of the actual plant an efficiency of 30-75% COD_{total} removal is found while the BOD₅ efficiency is about 45-80%. The efficiency of the anaerobic treatment plant with low loadings (RALF, lagoon) is clearly below the data of other publications (HAANDEL & LETTINGA, 1994). Probably due to a very high quantity of infiltration of groundwater, leading to low influent concentrations.

- The fraction of the easily degradable COD (S_s) in raw wastewater is situated in the usual range of 10-15% of the total COD, with exception of the influent to the UASB, where it is only about 2%. The extremely short sewer system in this case might be an explanation for this observation.

- The fraction of the easily degradable COD in the effluent of anaerobic treatment in the under-loaded systems (RALF, lagoon) was 3-4% of the total COD, but in the overloaded UASB it was nearly 10% and in septic tank it rises up to 20%.

- TKN elimination by the anaerobic treatment is always about 6-8%, which is nearly the same as the elimination by mechanical pre-treatment.

As a next step the suitability of the anaerobic effluent for nitrification and denitrification purposes was investigated in batch-tests (table 1). The composition of the wastewater before and after the anaerobic treatment was compared.

- Both, the nitrification rates (reference temperature 20°C) as well as the denitrification rates are low, probably because of the low concentration of active biomass in the activated sludge. The sludge concentration of the treatment plant was very high (7-8 g TSS/L) during the time of investigation leading to a very high sludge age.

- In contrast to former published experiences with sewage of the anaerobic sludge treatment (HOFFMANN, 2000) inhibition problems concerning nitrification after anaerobic treatment could not be found. In each case, the nitrification velocity of the anaerobically treated wastewater was higher due to the lower organic concentrations. The effluent of the UASB showed reduced nitrification velocities in some cases, but an inhibition of nitrification did not take place.

- The maximum denitrification velocity decreases after anaerobic treatment. Compared with raw wastewater RALF and anaerobic lagoon still achieve about 65-70% of the initial velocity. The high loaded UASB and septic tank however achieve 85-90%.

- The maximum denitrification velocity is not sufficient to express the capacity of denitrification after an anaerobic pre-treatment. With a mixing proportion of 1:1 activated sludge:

wastewater, the maximal velocity of denitrification of the pre-treated wastewater already ended after 45-60 min, while denitrification with maximum velocity of the raw wastewater took place over 90-180 min (fig. 2). Therefore the tests, which indicated a clear change from nitrate respiration of the easy-degradable COD to endogenous respiration, were used to determine the COD available for the denitrification (COD_{DN}).

- The fraction of the COD immediately available for denitrification (COD_{DN}) determined with the anoxic test was much higher than the fraction of easily degradable COD (COD_{SS}) determined with aerobic O_2 consumption test. The part of COD immediately available for denitrification in the raw wastewater varies between 30-40% and again only the influent to the UASB showed an exception with only 10%. A comparison of the total values after anaerobic treatment shows that for both low loaded systems, the lagoon and the RALF, the COD_{DN} was reduced by 75-80%, but in the high loaded UASB it was reduced only by about 50% and in the septic tank about 30%.

- If the BOD_5 is set into relation to the COD_{DN} , it can be observed, that the effluent concentration COD_{DN} in three reactors is about 40% of the BOD_5 , while it varies in the influent between 15-85%. In the septic tank it rose from 63% in the influent to 83% in the effluent.

- The very common oxygen consumption test seems not to be appropriate for fractionating the low concentration of rest-COD after anaerobic treatment. A direct determination of denitrification capacity (COD_{DN}) has shown to be more suitable. Very important is a clearly interpretable graph, which was not achieved applying low loads (1 part of activated sludge : 1 part of wastewater). Only with F/M values of 0,1 g COD/ gTSS these curves were able to be used. An addition of the nitrate to up to 100 mg NO_3-N /L facilitates the test, however the nitrite development has to be considered. We defined the part of COD available for denitrification as COD_{DN} in difference to the COD_{SS} of the aerobic O_2 -consumption test.

4 COMPARISON OF INVESTMENT AND OPERATION COSTS

A comparison of combined anaerobic-aerobic wastewater treatment to a conventional aerobic treatment as a single step under the aspect of investment and operation costs and operational aspects was carried out. The experiences were gained in projects for five small cities in Santa Catarina, Brazil (PLATZER ET AL., 1999). The size of the plants ranged from 15.000 – 50.000 p.e.. The data are based on the costs of a treatment plant for 20.000 p.e. currently under construction and the referencial costs of CASAN, a state company for basic sanitation in the state of Santa Catarina, Brasil. The database for prices is constantly updated. In order to be able to compare the results more easily, all results were transformed to a 20.000 p.e. plant equivalent. The costs were compared with typical investment costs of wastewater treatment plants in Brazil.

As Brazil is a country with a warm climate and low possibilities for large investments or high operational costs, it seems more than logic to apply anaerobic processes as much as possible. On the other hand one has to see given parameters by the Brazilian law. The law CONAMA (1986) an Ammonia-N concentration of 5 mg/l in the effluent. Even though up to now the requirement for Ammonia removal is not followed, it has to be expected that this parameter is the next parameter to be followed. In some cases state laws define more strict effluent standards, i.ex. Santa Catarina has defined an effluent concentration of 10 mg N_{tot} for discharge in coastal waters (CONSEMA-SC, 1981). Depending on the demands of nitrification and denitrification the advantages and disadvantages the treatment strategies must be balanced carefully.

For the basic requirements by CONAMA (1986) the best treatment technology is a combination of a USAB and a aerobic treatment for the rest BOD_5 and nitrification. Good opportunities in this case are:

- trickling filters and rotating disc reactors,
- constructed wetlands (vertical flow beds - VFB)
- and the activated sludge process.

These combinations offer an almost complete nitrification, and as it has been shown in the previous chapters, the nitrification is not influenced negatively by the anaerobic treatment. All of the combinations meet perfectly the Brazilian effluent requirements given by CONAMA (1986).

The combinations are compared with a conventional activated sludge plant with aerobic stabilization of the sludge. All alternatives are equipped with a automatic screener, grid removal. Sludge mineralization beds are used as final sludge treatment. This form of sludge treatment offers about 7 years of storing capacity. Due to this the organic compounds are significantly reduced, resulting in a total reduction of the mass of dry sludge by one third. The basic dimensioning criteria for the processes were:

For the activated sludge plant: aerobic reactor - sludge age 20 days, content of total solids 3,5 kg/m³, sludge production 1 kg TSS/ kg BOD₅, aeration with rubber membranes, (reactor volume 6.171 m³); secondary clarifier – sludge volume index 130 ml/l, sludge volume surface loading 450 l/m²,h or sludge surface loading of 30 kg TSS/m²,d

For the UASB and activated sludge plant: Equalisation tank of 400 m³, UASB – hydraulic retention time of 8 hours, sludge production 0,45 kg TSS/ kg BOD₅ (raw wastewater influent), digestion of 1/3 of secondary sludge mass, (reactor volume 950 m³); aerobic reactor - sludge age 8 days, content of total solids 3,5 kg/m³, sludge production 0,8 kg TSS/kg BOD₅, aeration with rubber membranes, (reactor volume 600 m³); secondary clarifier – same as described above.

For the UASB and trickling filter: Equalisation tank and UASB – same as described above without co-treatment of the secondary sludge; trickling filter – filling material of trickling filter 150 m²/m³, organic loading 0,3 kg BOD₅/m³,d, (total volume 1.100 m³); secondary clarifier – detention time 2,5 h, hydraulic load 1,0 m/h.

For the UASB and Vertical Flow Bed (VFB): Equalisation tank and UASB – same as described above; storage tank for intermittent loading; Constructed wetland as Vertical Flow Bed – Organic loading 12,5 g BOD₅/m²,d, Filter depth 0,6 m, intermittent pulse loading every 4 hours, division of the bed in four compartments.

Table 2 shows the results. The conventional activated sludge plant is about 35% more expensive than the other alternatives. This is due to the large volume necessary for the aerobic treatment of the total organic load, because the anaerobic treatment offers an initial elimination of about 70% of the organic load. The other processes are about equal in investment costs, presenting a slight advantage for the combination UASB and trickling filter. The costs for the combination of UASB and constructed wetland as vertical flow bed depend very much on local conditions.

Table 2: Comparison of Investment costs of four treatment alternatives (all costs in US\$)

	Activated Sludge Plant	UASB+ activated sludge	UASB+ trickling filter	UASB+ vertical flow bed
Civil construction	383.085,91	257.855,02	331.783,75	370.968,24
Additional equipment	101.653,47	107.572,52	94.504,91	152.401,39
Technical equipment	318.679,20	222.160,12	158.578,25	89.013,03
General aspects	35.428,72	33.314,09	33.732,62	32.450,97
Total costs	838.846,87	620.901,31	618.599,53	644.833,20
Cost per p.e.	41,94	31,05	30,93	32,24

Typical costs for wastewater treatment plants in Brazil range from 25,00 U\$ (lagoons) to 50 U\$ (activated sludge plant with nutrient removal and disinfection) (ANA, 2001). An advantage which is not expressed in the costs is the simplicity in construction and operation of the two alternatives with UASB and trickling filter or constructed wetlands. This is an advantage which is important for proper functioning of the treatment systems. Comparing

operational aspects, not only the costs can be seen, as these differ largely from region to region. Therefore we expressed the costs together with some basic operational data in table 3. Determining parameters are the specific energy consumption, the specific sludge production, the number of operators necessary and the technical maintenance costs. The technical maintenance costs were calculated with 2% per year of the technical equipment costs (table 3). Under Brazilian circumstances the process with the lowest operational costs is the UASB in combination with the constructed wetland VFB. This due to the lowest energy consumption and very low costs for technical equipment. The sludge production is low as well, because the process only produces primary sludge in the UASB.

Table 3: Comparison of basic operational data and operational costs for 4 different treatment alternatives

	Activated Sludge Plant	UASB + activated sludge	UASB + trickling filter	UASB+vertical flow bed
Energy consumption (kWh/p.e, year)	36	16	3	0,5
Sludge production (kg/p.e, year)	13,1	8,1	8,1	5,9
Operators	2,5	3,5	2,5	2,3
Operational costs (US\$)	66.592	38.463	17.178	10.784
US\$/p.e	3,3	1,9	0,9	0,5
<i>* including a 30% reduction of total mass by the sludged mineralisation beds</i>				

It has to be seen clearly that the three alternatives with an anaerobic pre-treatment do show disadvantages considering nitrogen removal. Especially the two solutions with low energy consumption and more simple construction requirements, which are the UASB in combination with a trickling filter or a constructed wetland, are almost impossible to be applied in cases of nitrogen removal requirements.

In the exceptional cases where a nitrogen removal down to 10 mg N_{tot}/l, we would currently recommend the UASB in combination with an activated sludge process and the addition of an external carbon source. The difference in investment costs and operational costs between the two possible solutions (conventional activated sludge plant or UASB in combination with activated sludge) for nitrogen elimination show, that the additional costs for an external carbon source and an additional reactor could be easily absorbed by the solution with anaerobic pre-treatment. According to our orientating results presented above, a bypassing of 30 % of the raw wastewater, as recommended by HAANDEL & LETTINGA (1994) would not be sufficient for an enhanced nitrogen removal. Although each plant should be equipped with a bypass of the UASB for operational reasons.

An other way, still to be researched under these climatic conditions, could be an anaerobic fixed film reactor as i.ex. trickling filter as described by DORIAS (1996).

5 CONCLUSION

in almost all aspects the conventional activated sludge plant is not recommendable in comparison to the processes with an anaerobic pre-treatment. It is not understandable why in a warm country like Brazil the activated sludge process is still one of the most applied process for wastewater treatment. Current research and plant design is leading towards a more appropriate use of technology (HAANDEL & LETTINGA, 1994; GONÇALVES ET AL., 1998; PLATZER ET AL., 1999; SPERLING ET AL., 2001; AISSE ET AL., 2001), but still a lot of projects are being developed leaving out these experiences.

Beside the costs the energy factor should be considered in a country which suffered lately from a lack of energy. The "low energy" solutions consume about 1,5 to 8% of the energy necessary for a conventional activated sludge plant. Another fact to be considered is the final sludge use or disposal, which in a lot of cases is not properly managed. In these cases smaller quantities of sludge would harmless.

Therefore we recommend to use systems which present low energy consumption and sludge disposal.

The batch tests showed that in cases of necessity of nitrogen removal the addition of an external carbon source is uninevitable, but we question in most cases the necessity of an enhanced nitrogen removal in a country where only 35 % of the wastewater is treated especially regarding the aspect that a post-denitrification would be possible to be introduced later.

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