Institut Agronomique et Vétérinaire Hassan II

Anaerobic Reactor High-Rate Pond Combined technology

for sewage treatment in small communities

Implementation, operation and performance

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Abbreviations and acronyms

ARHPCT	anaerobic reactor high rate pond combined technology
BOD_5	biochemical oxygen demand
CH_4	methane
CODs	soluble chemical oxygen demand
CODst	chemical oxygen demand after 30 min settling period
CODt	total chemical oxygen demand
DO	dissolved oxygen
FC	faecal coliforms
HRAP	high rate algal pond
HRT	hydraulic retention time
IAV	institut agronomique et vétérinaire Hassan II, Rabat.
k_{20}°C	first-order reaction rate constant at 20°C
KTN	Kjeldhal total nitrogen
LAR	land area requirement
Log Unit	logarithmic units
MENA	countries of the Middle East and North Africa region
MP	maturation pond
$N\text{-}NH_4^+$	ammonia nitrogen
O & M	operation and maintenance
p.e.	person equivalent
PO_4^{3-}	orthophosphates
RR	removal rate
SGSS	small bore gravity sewer system
SRT	solid retention time
SVI	sludge velocity Index
SS	suspended solids
TSUAR	two-step upflow anaerobic reactor
UASB	upflow anaerobic sludge blanket reactor
VFA	volatile fatty acids
VSS	volatile suspended solids
WHO	world health organization
WSP	wastewater stabilization ponds

Preface

Countries of the Middle East and North Africa (MENA) region share similar climate conditions and socio-economic features. For example, sanitation and wastewater treatment in rural areas are very poor or inexistent. In addition, recent concepts such as sustainable decentralized sanitation and wastewater and nutrients recycling are ignored. Therefore, any successful experience addressing these kinds of issues in one of the MENA counties could benefit to the whole region.

Impoverishment of the rural populations following repetitive draught episodes in the region has caused an important migration to “urbanised” villages or suburban settlements. To cope with this phenomenon, central and local governments have made efforts in providing drinking water, electricity and other services (for example, the provision for drinking water to the rural population in Morocco jumped from 14% in 1994 to more than 50% in 2004). But, it is still widely admitted that sanitation and wastewater investments have not kept pace with the urbanisation rate in the MENA region.

Development of sustainable and low-cost technologies for the adequate collection and treatment of wastewater could help fill the gap. In this respect, practical alternatives to meet increasing capacities and relatively larger communities (several hundreds or thousands of homes) must be urgently implemented where common individual or small cluster of houses approaches showed their limits.

However, any alternative system must meet fundamental prerequisites when addressing low-income communities to guarantee its sustainability. The success of any sanitation project is to be analysed in the context of limited funding capabilities, increasing resource depletion and greater environmental protection measures. The main requirements to be fulfilled by any chosen system are:

i. Low investment cost (avoiding equipment purchase and import).
ii. Low land area requirement.
iii. Simplicity of construction and operation.
iv. Minimization of sludge production.
v. Transformation of organic matter into useful energy.
vi. Recycling of nutrients for crop production.
vii. Water conservation through agricultural reuse and/or urban purposes.
viii. Minimization of wastewater collection and conveying costs.

This document describes the development of a low-cost technology to treat wastewater for small communities in Morocco. This technology could be suitable for most of the MENA counties. However, adoption of a low-cost and well-adapted technology is far from fulfilling all conditions listed above. Requirements vi), vii) and viii) could only be attained if sustainable concepts for water management are adopted, particularly by considering wastewater as part of the community's water budget.

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The development of the Anaerobic Reactor High rate Pond Combined Technology (ARHPCT) could not have happened without the support of former Director of the IAV, Dr M'Hamed Sedrati, and Mr. Tayeb Benchokroun from the Administration Générale du Génie Rural, Ministry of Agriculture. Help and support from the IAV Director and staff are acknowledged by the author.

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Introduction

The idea of developing the “Anaerobic Reactor High-rate Pond Combined Technology (ARHPCT)” evolved some fifteen years ago when a research project was started at the IAV, Rabat to adapt Oswald’s high rate algal pond (HRAP) for treating domestic sewage in Morocco.

Soon, the necessity of adding an anaerobic pond to the HRAP became obvious and led to the construction of an experimental unit including an open anaerobic pond. Later, the system was completed by adding a maturation pond for disinfecting the effluent.

In 1992, the HRAP system was scaled up and implemented, side by side to a WSP train for comparison, in Ouarzazate beyond the Atlas Mountains in the south of Morocco (Actes, MOR 86/018; El Hamouri *et al.,* 1995). The higher performance of the HRAP system over the WSP system was demonstrated and allowed the reduction of the net land area requirement by at least 40% (El Hamouri *et al.,* 2003).

This result incited the author to develop further the work on the HRAP system. But due to the bad public acceptability of the project, particularly because of persistent offensive odor emanations, the system was modified by adopting covered anaerobic reactors instead of open anaerobic ponds resulting in a new setup called ARHPCT that was implemented at the IAV campus, Rabat in 1996.

The central concept in the development of the ARHPCT could be summarized as a combination of high rate anaerobic and aerobic complementary reactors, in which main biological functions requested for waste degradation were more or less segregated and confined in limited spaces to avoid negative feedbacks and dilution effects.

From the hydrodynamics perspective, the approach consisted of choosing treatment components that could favor plug flow patterns to enhance the treatment performance and to reduce reactor size.

The set up of the ARHPCT did finally result in the adoption of two consecutive treatment units:

i) A pre-treatment unit including the two-step upflow anaerobic reactor (TSUAR) followed by a settling tank and a gravel filter.

ii) A post-treatment unit consisting of a HRAP followed by one maturation pond.

The pre-treatment had the role of removing suspended solids and soluble organic components while minimizing sludge production and handling. The post-treatment unit, which relied on solar energy, removed soluble nutrients and contributed to pathogen die-off.

The results obtained at the IAV treatment plant following a five-year sampling and analysis program, showed that the ARHPCT could offer excellent performance and remarkable stability in time.

The salient features of the ARHPCT are: i) the low-cost, ii) the small land area requirement, iii) the ability to controll offensive odor emanation and iv) the simplicity of construction and operation, particularly for sludge withdrawal and handling.

The ARHPCT is suitable for small communities and tourist installations located outside an urban area with no possibilities of a connection to existing sewage networks. It could also fit in decentralized sanitation approaches, where small plants can be constructed inside the urban area. This could favor *in-situ* effluent reuse for landscaping with minimal conveyance and cost and could, in some cases, help postponing or avoiding existing plant extensions.

1. Rationality behind the development of the ARHPCT

1.1. Limits of the open anaerobic pond

The wide practice of putting an open anaerobic pond at the head of the treatment line is common in WSP design. However, this "necessary" unit has three main drawbacks:

I) Low performance in sewage treatment. We may confirm the most reported values for BOD removal gathered from several plants in Morocco, which never exceed 50 to 60% removal. This is mainly due to poor basin hydrodynamics (Pena and Mara, 2004), large day/night temperature changes (Ouazzani, 1998) and thermocline formation in the summer.

ii) The necessity of constructing two similar anaerobic ponds in parallel at the head of the ponds train; one being under operation while the second is emptied for sludge thickening before its removal.

Recall that accumulated sludge in anaerobic ponds has to be withdrawn every four to five years (WHO, 1987). This operation represents one of the largest expenses in WSP operation cost. Moreover, when PVC films or compacted clay are adopted for waterproofing the ponds, the use of mechanical means to scrape the bottom and to withdraw sludge often lead to large cuts or cracks, which are rarely mended.

iii) Emanation of offensive odor, particularly in areas where the plants cannot be constructed down stream of the prevailing winds (this happens when topography and prevailing wind are not compatible).

1.2. Limits of the HRAP operated as a secondary/tertiary unit

The HRAP is a photosynthetic reactor, in which microscopic, photosynthetic algae are living together with heterotrophic bacteria. This pond is carrousel-shaped, is shallow, from 35 to 50 cm, and is continuously mixed by a paddle wheel (Oswald and Gotaas 1957; Oswald, 1988).

In this system, saturating concentrations of dissolved oxygen (DO) and high pH values prevail in the pond during the daytime, preventing anaerobic bacteria from colonizing the pond (provided a sludge layer build up could take place in the bottom of the pond).

The HRAP has a high capacity for solar energy capture that forces algae cells to evolve a maximum of oxygen for waste degradation by aerobic bacteria. In return, nitrogen, phosphorus and CO_2 resulting from the accelerated waste degradation are taken up by algae to sustain their growth in the pond. Such a co-habitation is often called a symbiosis. It represented the central idea of Oswald's concept using the HRAP, as a combined secondary/tertiary system, for sewage treatment (figure 1).

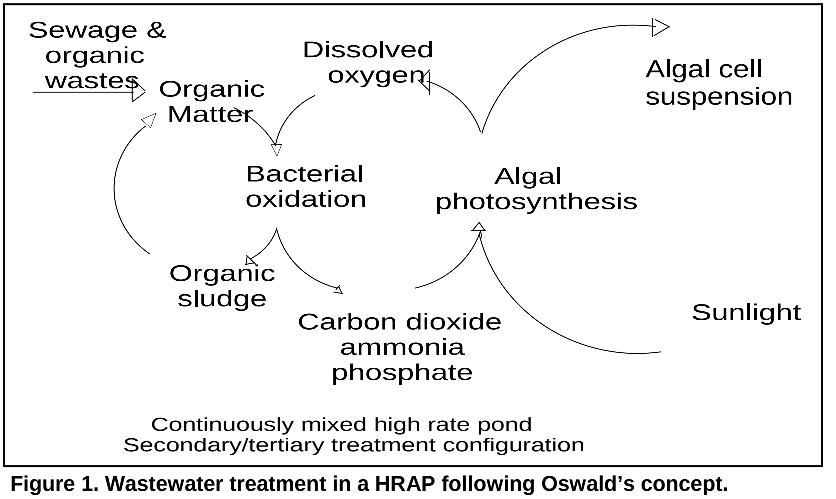

Figure 1. Wastewater treatment in a HRAP following Oswald's concept.

However, we have learned from our own experience working on HRAPs in Morocco for fifteen years that the HRAP system only works within limits. The adoption of a reliable pretreatment unit to reduce sewage BOD and TSS content before feeding the HRAP is a fundamental condition for sustainability of this "ecosystem". In the absence of such a unit, "Oswald's symbiosis" simply cannot take place. Light penetration through the water column is also essential in the HRAP. High concentrations of biodegradable organic matter favor bacterial growth at the expense of algae. If light is scattered by both bacterial cells and TSS and is prevented from penetrating in the pond, the HRAP could not become green!

Keeping the bacteria from taking over and to prevent algae from growing beyond the acceptable limits for such an ecosystem gives an insight on how difficult it is to operate a HRAP following Oswald's secondary/tertiary treatment concept.

Green and Oswald wrote in 1993 "the strategy [in operating a HRAP] is to collect sufficient solar energy in the form of algal biomass to oxygenate the waste, but not to grow more algae than is required for oxygen production"

This dilemma is resolved in *Chlorella* farms, where the Oswald's HRAP is used for algae production (high-added value food) on fresh water. In these units, nutrients and CO_2 are supplied and algae are harvested at regular intervals following a reformulated Green and Oswald's statement: "the strategy is to collect as much solar energy in the form of algal biomass as the supplied CO_2 and nutrients could allow it"; that is to maximize the green dry matter production.

Oswald and his co-workers (see Oswald, 1988) also investigated the possibility of valorizing sewage-grown algae for animal feed or for biogas production. These studies quickly showed the limits of such an approach due to the cost of the harvesting equipment and supplies.

Moreover, algae cells have the capacity of concentrating chemical components found in municipal sewage inside their cells, particularly heavy metals and other industrial organics, This introduces another limit for their valorization in animal feed.

This prevented income generation for the economical viability of the system constituting therefore the major obstacle that faced Oswald and others in applying the "regular algae harvest strategy" to keep the HRAP under the control of the operator. Such an intervention is essential in similar processes. For example, in an activated sludge system, in which the growth of bacteria is pushed to the maximum by supplying the cell with the necessary oxygen and eventually nutrients, the cell concentration in the aeration tank is continuously kept within optimal ranges using the harvest/recycle strategy.

1.3. Approach adopted to develop the ARHPCT

1.3.1. Basic principles

The basic research work to develop the ARHPCT was based on the following principles:

i) Segregate, as much as possible, biological activities taking place in complex systems. Indeed, when two or more biological activities, for example anaerobic and aerobic waste degradation, are taking place simultaneously in the same space and they are depending one on the other, the system will obey the limiting factor principle making the overall speed of the process be dictated by the slowest activity.

Thus to avoid negative feedbacks of one activity on the other, we adopted "specialized" components that exhibit narrow biological activities and placed them in series in a complementary way.

ii) Adopt geometrical shapes that help reducing the LAR, avoiding short-circuiting and thermal stratification, tending towards plug flow hydraulic pattern and allowing anaerobic units to be completely covered.

1.3.2. Option for high-rate anaerobic reactors

In anaerobic ponds, primary sludge sedimentation, complex organics hydrolysis and acetate transformation into methane[1] do take place in the same space and the same time partially explaining the low performance of this system. BOD removal rates not exceeding 50 to 60% were reported for a HRT of 3 to 4 days under Moroccan climate (see § 3.2.)

The adoption of upflow reactors proved to be an excellent approach in sewage digestion. It increases the contact between the incoming wastes and anaerobic bacteria leading to higher performance and to shorter HRT i.e. low LAR. Also, an important fact was highlighted by the research team of the Agricultural University of Wageningen in the Netherlands, which showed that particulate COD was highly concentrated in urban sewage (between 40 and 60% of total COD) and that a settling and hydrolysis step was required upstream of the methanization step, particularly at low temperatures. They recommended adopting a two-step rather than a one-step reactor (Wang, 1994).

The anaerobic reactor adopted in Rabat was a two-step upflow anaerobic reactor (TSUAR). In this system, the choice not to manually remove any excess sludge for operation simplicity, was deliberate. This option distinguishes the TSUAR from the widely known UASB (upflow anaerobic sludge blanket), in which the operator, to keep in permanence an optimal sludge concentration in the reactor, has to decide on how much sludge to remove, from which depth of the reactor and at what frequency.

In the TSUAR, the option not to manually remove sludge forces the reactors to work on the “maximum sludge hold up” mode (van Haandel & Lettinga, 1994). The advantage of adopting this mode is the simplicity and the ease of the sludge management inside the plant (see § 4 operation and maintenance).

1.3.3. Option for a HRAP operated as a tertiary unit

At the IAV plant, the HRAP was operated as a post-treatment unit being placed behind a TSUAR, a pre-treatment unit, which removed 80% of total COD (CODt) and 90% of SS. First-order reaction rate constants, $k_{20°C}$ for the removal of CODt, nitrogen and phosphorus were determined for the HRAP operated under this configuration (table 1).

[1] The fundamentals of sewage anaerobic digestion can be found in Malina, 1962, and McCarty, 1964.

Table 1 shows that instead of removing organic carbon (positive $k_{20°C}$ value), new material was added to the effluent during its treatment in the HRAP. The HRAP changed from a carbon degradation unit to another that was building up new biological material. Algae cells constituted the essential of the produced material and algae growth became the main activity in the HRAP. Also in order to utilize the available nitrogen and phosphorus, the ecosystem started importing CO_2 from the atmosphere therefore explaining the occurrence of a negative $k_{20°C}$ value.

The TSUAR removed most of biodegradable wastes up stream. Doing this, the HRAP looses its secondary/tertiary function to become a strictly tertiary treatment unit explaining the large positive $k_{20°C}$ values found for nitrogen and phosphorus removals (table 1).

Table 1. First-order reaction rate constants, $k_{20°C}$ for the removals of total COD, total nitrogen and total P obtained in the HRAP of the IAV plant.

	CODt	Total nitrogen	Total phosphorus
$k_{20°C}$ (d^{-1})	-0.245	0.653	0.249

Operation of the HRAP under a tertiary mode implied that atmospheric CO_2 was used to produce *de novo* algae biomass. Doing this, the HRAP started removing high amounts of N and P and immobilizing them in algae cells.

Therefore, because of the limited availability of atmospheric CO_2, carbon concentration in the HRAP became the limiting factor that helped keep the algae cell concentration within compatible limits (figure 2) preventing the large changes in the pond content and performance observed when the HRAP was operated as a secondary/tertiary treatment unit.

The consequence of controlling the algae growth is seen on the DO concentration recorded in the HRAP. The average of a one-week DO continuous recording in the HRAP of the IAV plant during the coldest period of the year 2004 (average temperature around 15°C) is shown in figure 3. This corresponds to the most non-favourable period in the Mediterranean climate.

The high concentrations of DO obtained at mid-day as well as the fact that no anoxic period was observed in the night indicate that photosynthetic activity produces enough oxygen during the light period to overcome the oxygen needs of algal respiration.

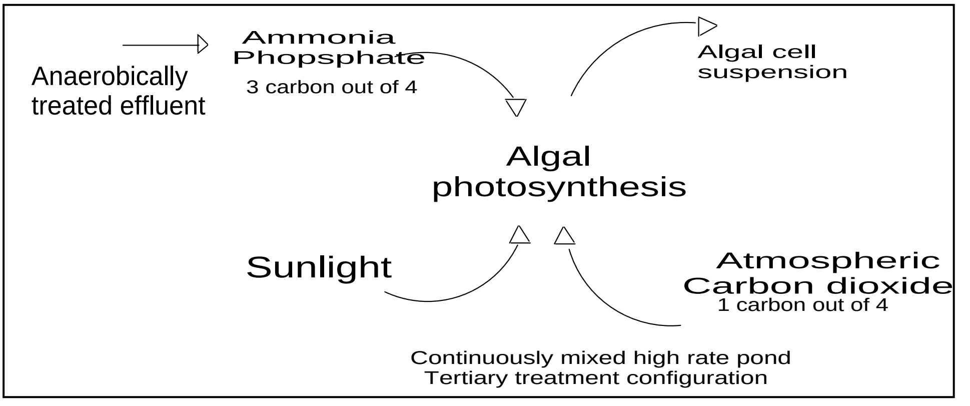

Figure 2. Function of a HRAP operated as a tertiary unit.

Table 2 summarizes the main operation parameters of the HRAP used as a strictly tertiary treatment unit.

Table 2. Recommended parameters to operate a HRAP as a tertiary treatment unit.

Parameter	Value
Organic loading rate (kg CODt ha^{-1} d^{-1})	80
HRT (d)	3
Water depth (m)	0.35
Chlorophyll-a (mg/l)	0.6
Algae cell counts (10^6/ml)	0.8

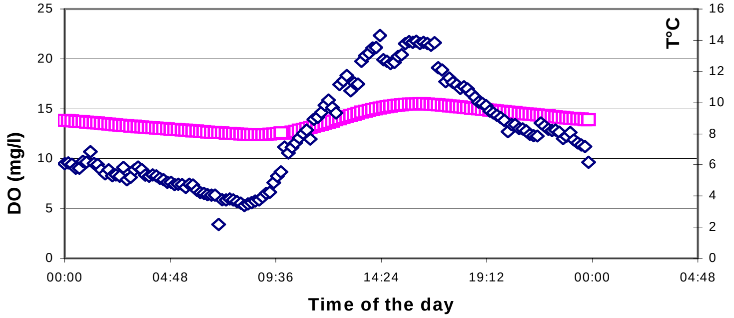

Figure 3. One-week continuous recording of diurnal water, temperature and dissolved oxygen in the HRAP during the coldest days of the year.

Under the secondary/tertiary mode also tested at the IAV Plant, algal cell concentrations were so high at some periods that all available oxygen was depleted within the first hours of the night leading to long anoxic periods in the HRAP (see El Ouarghi *et al.,* 2000).

2. Implementation of the ARHPCT at the IAV campus

The first plant based on the concepts described above was constructed in the campus of the Institute of Agronomy and Veterinary Sciences (IAV) (1,500 students) in an urban environment (see album) and put into service in December 1996.

The city of Rabat, capital of Morocco, is located in the North-West of the country (latitude 30°03' N, longitude 6°46' W). Its altitude is 73 m above sea level. The average temperature in the site is 14°C in the cold season and 24°C in the hot season. The facility at the IAV campus receives wastewater mainly from the students residence and restaurant.

The plant occupies 1,200 m^2 and receives a daily flow of 63 m^3. It includes a preliminary treatment (screening and grit removal) followed by a TSUAR, for pre-treatment, and then by a post-treatment line, which includes a HRAP flanked with one maturation pond (MP) (figure 4). Design details and dimensions are reported in annex 1 "Example of drawings". All the components are constructed in reinforced concrete, but approaches using local material could be adopted.

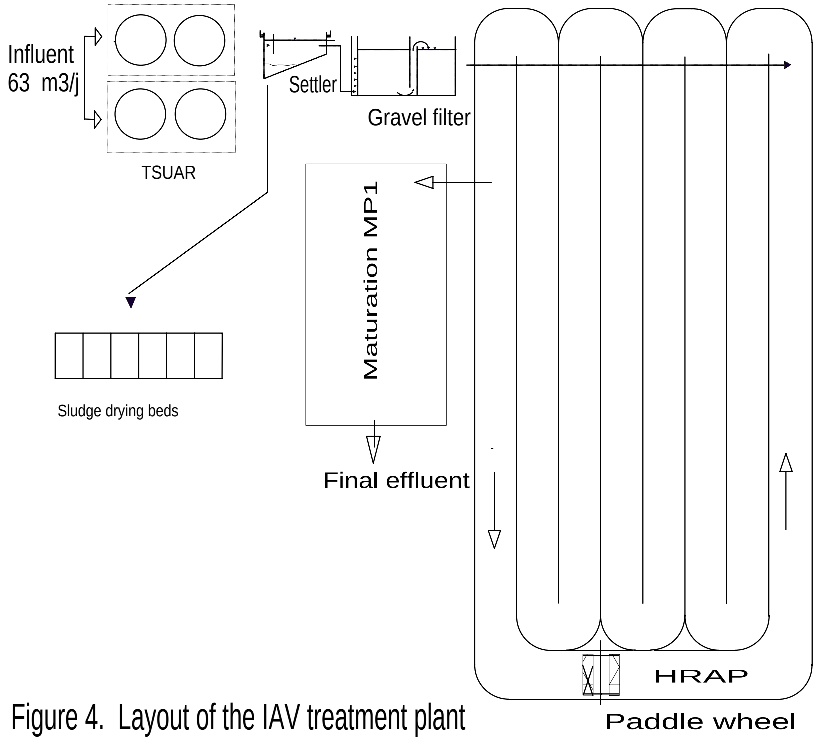

Figure 4. Layout of the IAV treatment plant

2.1. The Pre-treatment unit or the two-step upflow anaerobic reactor

High-rate anaerobic systems are often called "pre-treatment" while units placed downstream are called "post-treatment". Post-treatment units are in charge of nutrients and pathogens removals particularly if the effluent is to be reused for agriculture purposes (Lettinga *et al.,* 1997; Zeeman and Lettinga, 1999). BOD and TSS removals are also called secondary treatment while nutrient removal and disinfection are called tertiary treatment.

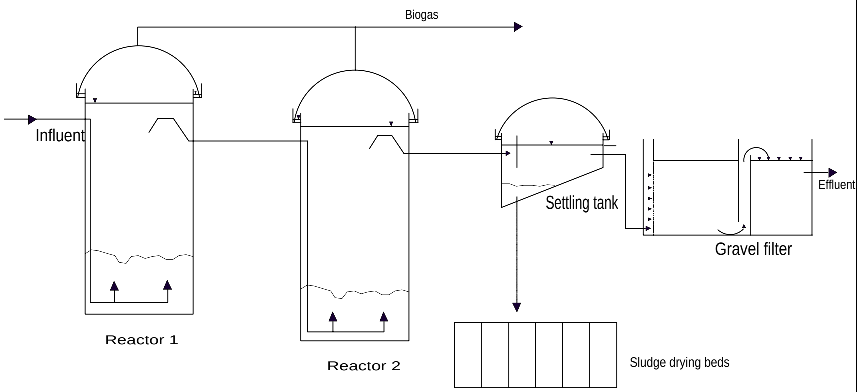

Figure 5. The pretreatment unit of the IAV treatment plant

2.1.1. Reactors R_1 and R_2

Reactors R_1 and R_2 are cylindrical with a diameter of 3 m. They are respectively 5.30 and 5.00 m deep. The part constructed above ground is 2.50 m for R_1 and 2.00 m for R_2 (figure 5 and table 3).

Table 3. Dimensions and operating parameters for reactors R_1 and R_2 of the TSUAR.

	Reactor R_1	Reactor R_2
Depth (m)	5.30	5.00
Area (m^2)	7.06	7.06
Diameter (m)	3.0	3.0
Volume (m^3)	33	31
Average HRT (h)	24	23
Average Solid retention time (d)	32	32
Over flow (m h^{-1})	0.1- 0.6	0.1 – 0.6
Number of inlets	2	2
Number of outlets	1	1
Average HLR (kg COD m^{-3} d^{-1})	0.76	0.40

HLR: hydraulic loading rate

In both reactors, upflow velocity was maintained in the range of 0.1 to 0.6 m h^{-1} depending on the admitted flow, which varies during the day.

Three main features distinguish the TSUAR from the widely known, UASB reactor

i) Absence of any in-built three phase separator (liquid/solid/gas)
ii) No manual sludge withdrawal from the reactors
iii) Longer HRT (48 h instead of 6 to 14 h in the UASB).

2.1.2. Biogas collection and anti-odour system

Biogas is collected from the reactors using hard, external cupola-shaped covers made of acid-resistant polyester material. The base of the covers is inserted into a channel surrounding the reactors external wall with dimensions of 0.40 m width and 0.40 m depth (figures 4 and 5).

This channel is filled with effluent from the HRAP to act as a water seal preventing biogas and offensive odours from escaping the reactor. The water of the seal channel is replaced by freshly treated effluent every week.
The IAV plant produces between 4 and 10 m^3 of biogas per day. The lowest production (4–6 m^3) coincides with the coldest period of the year, from December to February, where the average air temperature was around 15°C. The TSUAR biogas specific production was found to be 0.25 m^3 kg^{-1} removed COD, which corresponds to 0.19 m^3 CH_4 kg^{-1} removed COD.

The main components of the IAV biogas and their relative proportions were determined using gas chromatography analysis. The main components were methane, CH_4 (77%); nitrogen, N_2 (14%); oxygen, O_2 (4%); carbon dioxide, CO_2 (2%) and hydrogen sulphide, H_2S, which was found in traces. The presence of nitrogen at such a proportion in an anaerobic system is intriguing. Concordant evidences assign the occurrence of nitrogen in biogas to a new biochemical pathway, which associates the reduction of sulfate with ammonia oxidation under anoxic conditions. This anaerobic denitrification process was described by Mulder *et al.* (1995) and by Fdz-Polanco *et al.* (2001) and called ANAMMOX.

The possibility of burning biogas to produce electrical power was tested using a pilot-scale unit including a 7.5 KVA electric generator powered by a combined Diesel-biogas engine. The air admission device of the engine was modified such that biogas could be admitted with air to be burned inside the engine (see album). Assuming that 1 m^3 of methane is equivalent to 2.8 kWh, then the facility at the IAV might generate 21.5 kWh/day or 0.34 kWh/m^3 of treated wastewater.

United Nations Environment Program (UNEP) is funding projects aiming at the reduction of methane production. Experts are recommending to wastewater treatment plant owners to collect and valorize methane to reduce the green house effect. When valorization options cannot be implemented, the experts are simply recommending to burn methane to reduce the green house effect.

2.1.3. Settling tank

One of the main features of the TSUAR concept is the deliberate choice of constructing a settling tank located outside the reactor rather than adopting an integrated settling unit, as this is the case with the UASB reactors. A rectangular shaped settling tank was designed for a two-time normal flow and an overflow rate of 1.5 mh^{-1}. The unit was covered to avoid offensive odors emanation. Design of the settling tank, construction details and example of drawings are reproduced in annex 1.

2.1.4. Gravel filter

Reactor R_2 effluent was analysed for particle size distribution. The bulk of the sludge found in the effluent was made of two types of particles: i) reticulated particles with diameter between 100 and 350 μm having a sludge velocity index (SVI) of 20 mg/l and ii) low-density particles with a diameter of 60 μm and a SVI of 35 mg/l.

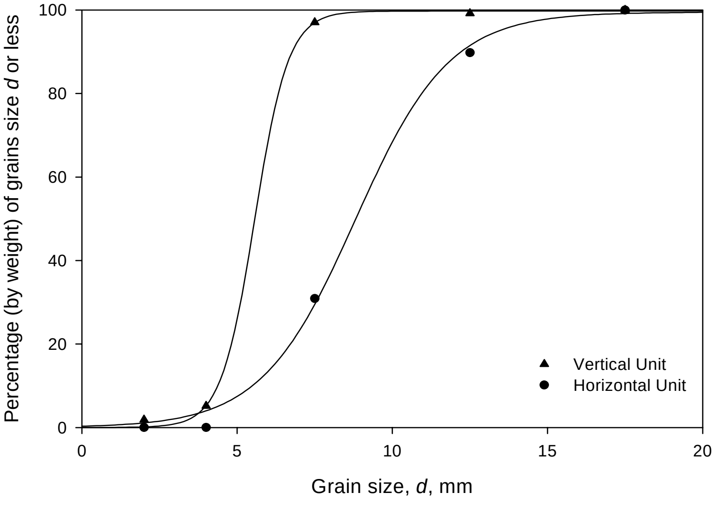

Figure 6. Grain size distribution of the filling medium of the gravel filter

Particles can escape the settling tank and be carried away to the photosynthetic post-treatment unit, with the immediate consequence of reducing the light penetration through the water column. They also contribute in the build up of sediment, which becomes a place for anaerobic waste degradation. This reduces the rate of oxygen evolution, nitrogen and phosphorus assimilation and prevents pH from climbing to values that are lethal for faecal coliforms (El Hafiane *et al.,* 2003; El Hafiane & El Hamouri*,* 2004).

The low-density troublesome particles are stopped using a gravel filter consisting of two steps; a horizontal unit (HU) followed by a vertical unit (VU). The filter has a rectangular shaped basin. Both Hu and VU have dimensions of 2 m width and 2.5 m length. The depth of the filtering medium was 0.8 and 0.6 m respectively for the HU and the VU and the hydraulic loading rate was 12.6 m d^{-1}.

<u>Main medium characteristics are</u>: (figure 6)

Horizontal flow unit (VU)	Vertical flow unit (VU)
Porosity = 47%	Porosity = 47.4%
d_{10}= 5.5 mm	d_{10}= 4.4 mm
d_{60} = 9.4 mm	d_{60} = 5.8 mm
Uniformity coefficient, U = 1.72	Uniformity coefficient, U = 1.32
Conductivity, K= 5,016 md^{-1}	Conductivity, K = 1,588 md^{-1}

2.2. Post-treatment unit:
The post-treatment unit included one HRAP followed by one maturation pond.

2.2.1. High rate algal pond
The HRAP of the IAV Plant has an area of 790 m^2 and a depth of 0.30 m. Water leaves the pond from structures equipped with rectangular weirs that control water depth in the pond.

A tracer study, using Rhodamine WT, was performed on this unit in 1998 (El Ouarghi *et al.,* 2000) and concluded that the hydraulic pattern was a plug flow with recirculation and small amount of dispersion. The study also showed that the dye concentration was homogenously distributed 17 h after injection and that the recirculation time was 78 min. From this study, the mean HRT was calculated and found to be 126 h. The flow at the period of the test was $50m^3/d$.

The tracer study also showed that the rate of short-circuiting in the unit was 30%. This deficiency came from the assumption made in the original design of 1996, which was based on the assumption that the stream created by the paddle wheel was strong enough to prevent short-circuiting. This is why the two pipes were placed side-by-side with the inlet pipe located downstream. Inlet and outlet pipes were modified in 1999 and set in the positions shown on figure 4.

2.2.2. Maturation ponds
The original design of the plant did adopt two maturation ponds in series. However, the follow up of the performance later showed that MP2 did not contribute significantly and was then by-passed. Therefore, the final design will recommend one MP (figure 4) with dimensions of 17 m x 7 m, depth of 1 m and a retention time of 1.5 days (see annex 1 for design and drawings).

2.3. Land area requirement of the ARHPCT

The ARHPCT needs approximately 1m^2 per capita. This area is to be multiplied by a factor of 1.20 to take account of walkways and outbuildings. Implementation for small communities does not require large space for trucks circulation or other vehicles. At the IAV plant, the width of the walkways left between treatment units does not exceed 1 m.

3. Performance of the IAV plant

Wastewater treatment performance presented here was obtained following a five-year sampling and analysis program (generally a sampling every 15 days). Figure 7 shows the average temperature of the influent and effluent recorded during the follow up period.

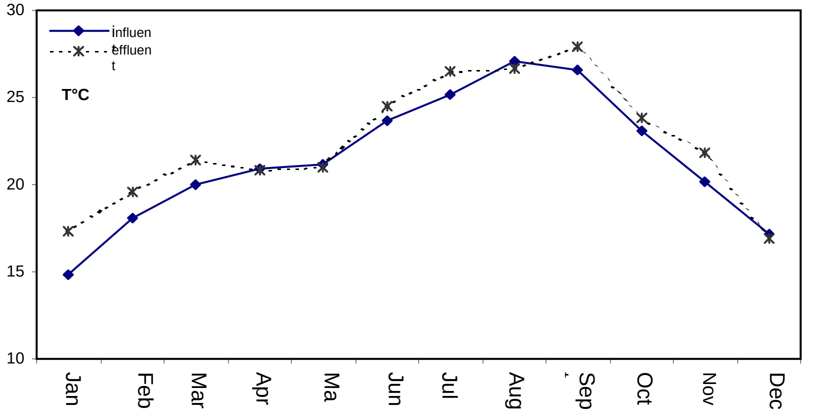

Figure 7. Average temperature of influent and effluent.

3.1. Performance of the pre-treatment unit

The pre-treatment means here: the two-step upflow anaerobic reactor (TSUAR) followed by the external settling tank and the adoption of a gravel filter later.

3.1.1. Stability and homogeneity of the reactors content

Diurnal temperatures in the reactors were stable and did not fall below 15°C in wintertime even when lower air temperatures were recorded (the temperature at the depth of 3 m is shown as an example in figure 4). In anaerobic ponds, diurnal amplitudes of temperatures in the range of 4 to 10°C were reported (Ouazzani, 1998).

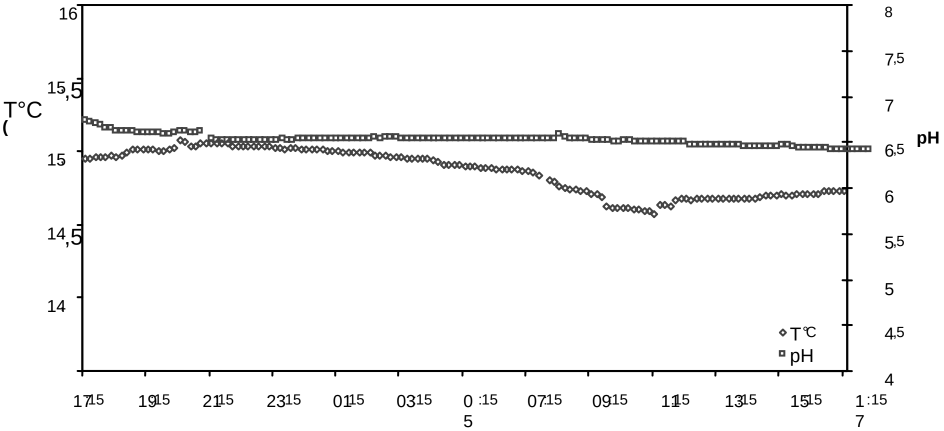

Figure 8. Winter water temperature and pH at the of 3 m-depth in reactor R_1.

3.1.2. Reactors performance

Empirical equations were established relating COD removal rate to the HRT in anaerobic reactors following the general expression of equation (1).

$$E=1 - Se/Si = 1 - C2\ (HRT)^{-C1} \qquad (1)$$

Where, Se and Si stand for COD concentration in the effluent and the influent respectively; E is the COD removal efficiency in % and C_1 and C_2 are empirical constants.

The stability of physico-chemical conditions were confirmed by diurnal pH values also shown in figure 8 while the homogeneity of the reactors is demonstrated by the pH profile shown in figure 9.

For an anaerobic pond, the constants C_1 and C_2 were found to be 0.50 and 2.4 respectively. Van Handel and Lettinga (1994) deduced these figures from several research papers on anaerobic ponds. Corresponding values for C_1 and C_2 were obtained for the TSUAR at the IAV and found to be 0.56 and 1.82 respectively (equation 2).

$$E = 1- 1.82\ (HRT)^{0.56} \qquad (2)$$

The HRT required for 80% efficiency in the TSUAR was 45 hours (less than 2 days) and six days (144 hours) in a classical anaerobic pond demonstrating how the approach adopted to reduce the LAR was successful.

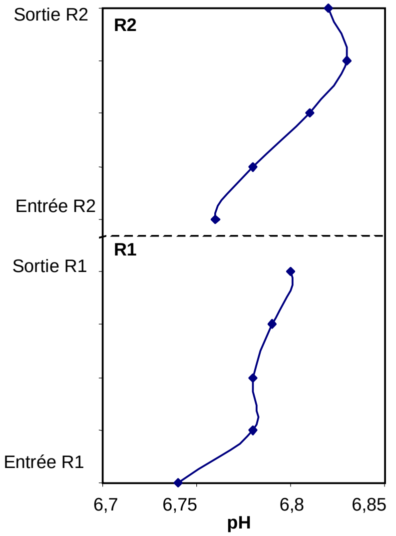

Figure 9. Average annual pH profile in reactors R1 and R2.

3.1.2.1. The ways to express reactors performance

COD removal rate (RR) for anaerobic reactors is usually calculated using equation (3) ; where, CODt and CODst stand for total and settled COD respectively ; *in* and *ef* stand for influent and effluent respectively.

$$RR_{COD} = (CODt\ in - CODst\ ef)/CODt\ in \qquad (3)$$

Equation (3) expresses the actual performance of reactors since it discards the contribution of the sludge particles to COD.

3.1.2.2. Removal of sludge particles improves reactor performance

Comparison of tables 4 and 5 shows that the construction of a gravel filter behind the settling tank improved the TSUAR effluent quality. Sludge particles accompanying the effluent were stopped at this level. Such an action is not only important for the assessment of the reactors performance but also for protecting the post-treatment unit from being submerged with sludge and suspended matter.

COD removal rates achieved by the TSUAR during the five consecutive years of monitoring at the IAV plant are shown in figure 6. They demonstrate the stability and the consistency of the performance of the TSUAR as a reliable pre-treatment system.

On the other hand, nutrient removal by TSUAR was very small if not nil. In many instances, ammonia and orthophosphate concentrations increased in the effluent confirming previous reports on anaerobic systems.

Table 4. Average performance of the line R1 + R2 + Settling tank.

	Influent	Reactor 1		Reactor 2		Settling tank		Global
	Value	Value	RR	Value	RR	Value	RR	RR
CODt (mg/l)	800	530	34	380	28	310	18	60
CODst		285		159	44	59	-	
CODs (mg/l)	420	270	36	120	56	120	-	70
BOD_5 (mg/l)	390	200	49	150	25	120	20	70
SS (mg/l)	330	300	9	280	7	230	18	30
VSS (mg/l)	190	150	21	160	-	105	34	45
TKN (mg/l)	72	60	17	66	-	65	2	10
$N\text{-}NH_4^+$ (mg/l)	46	48	-	50	-	50	-	-
Total P (mg/l)	8.2	8	2	8	-	8	-	-
PO_4^{3-} (mg/l)	5.7	6	-	6	-	5	15	-
FC	3.6 E7					7.1 E5		
Helminths egg/l	13					0	100	100

CODt : total COD ; CODst : COD after 30 min settling period ; CODs : soluble COD ; KTN : Kjeldhal (total) nitrogen ; $N\text{-}NH_4^+$: ammonia nitrogen ; PO_4^{3-} orthophosphates ; FC :faecal coliforms removal in log10 ; RR : removal rate .

Table 5. Effect of the gravel filter on the quality of the final effluent of the TSUAR.

	Settling tank effluent	Gravel filter effluent	Global Pre-treatment Removal rate (%)
CODt (mg/l)	310	110	86
BOD_5 (mg/l)	120	70	82
SS (mg/l)	230	15	95
VSS (mg/l)	105	5	97
TKN (mg/l)	65	61	15
$N\text{-}NH_4^+$ (mg/l)	50	49	-
Total P (mg/l)	8	8	-
PO_4^{3-} (mg/l)	5	6	-
FC (log_{10}/100 ml)	7.1 E5	7.1 E5	1.7*
Helminth (egg/l)	0	0	

For health related parameters, the performance achieved by the TSUAR confirmed earlier reports on the performance of anaerobic system for the removal of pathogens. At the IAV plant, helminth eggs were not found at the outlet of the gravel filter. Faecal coliform concentrations were also reduced in the system. Regularly, 1.7 logarithmic units were removed by the TSUAR.

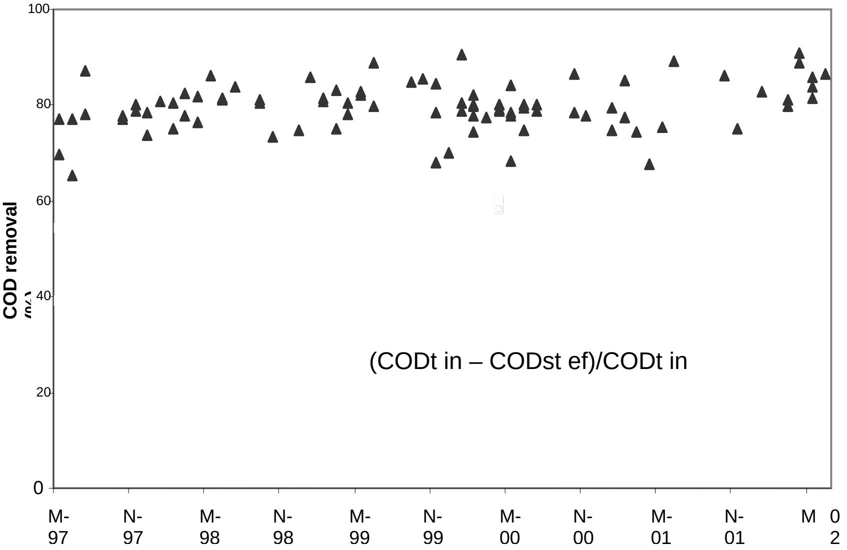

Figure 10. COD removal rates achieved by the TSUAR during the follow up period.

3.1.3. Specific role of reactors R_1 and R_2

Both reactors R_1 and R_2 were producing methane. They achieved similar soluble COD removal. However, the two reactors were working in series and consequently R_2 was only receiving 2/3 of R_1 organic load (figure 11) indicating that soluble COD removal rate in R_2 was 1.5 times higher than in R_1.

Two assumptions can be made to explain this difference:

i) Reactors R_1 and R_2 might be colonized by the same methanogenic group, but unfavourable conditions, such as inhibiting concentrations of VFA or low pH values, could prevail in R_1 but not in R_2

ii) Methanogenic bacteria colonising reactors R_1 and R_2 might belong to different groups, but those found in R_2 could have a higher activity.

The first assumption is unlikely. Indeed even though VFA concentrations were always higher in R_1, they never reached their inhibition level which is higher than 500 mg/l (Haskoning, 1994). Also, pH values in both reactors were closer and much higher than the value of pH 5, which was reported to inhibit acetotrophic bacteria (Haskoning, 1994) (see table 6).

In return, assumption ii) might be not far from reality, particularly in light of the results obtained in methane measurement testing. This laboratory test is done on sludge samples using acetate as the sole substrate. The methane produced is then proportional to the population of acetotrophic groups present in the sample.

Following this test, methane produced, expressed as mg COD mg^{-1} VSS, was 0.03 and 0.22 respectively for R_1 and R_2. This indicates that acetotrophic groups are either more active or were in larger numbers in R_2 than in R_1 and that the methane produced in R_2 likely originated from acetotrophic groups while that obtained in R_1 could have been mainly produced by hydrogenotrophic groups.

Based on this, a tentative function for each of the two reactors is proposed:

– R_1 would mainly function as a trap for particulate COD and as a digester harbouring acidogen groups and hydrogenotrophic methanogens.
– R_2 would mainly functions as a digester transforming acetate into methane with an apparent domination of acetotrophic methanogen groups.

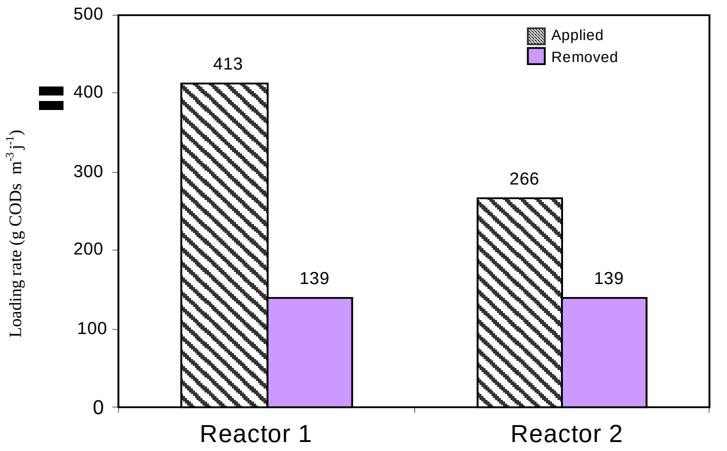

Figure 11. Applied and removed soluble COD in reactors R_1 and R_2

Table 6. Physicochemical characteristics for anaerobic digestion in reactors R_1 and R_2

	Influent	Reactor R_1	Reactor R_2	Settling tank
pH	6.9	6.6	6.8	6.8
Temperature (°C)	19.5	20	21.5	21
EC (µS/cm)	1,290	1,400	1,415	1,420
VFA (mg/l)	120	170	70	-
Alkalinity (mg $CaCO_3$/l)	120	164	204	-

VFA: volatile fatty acids

3.2. Post-treatment performance

3.2.1. Nutrient removal

The HRAP removed 89% of ammonia ($N\text{-}NH_4^+$) and 59% of orthophosphates ($P\text{-}PO_4^{3-}$) with residual concentrations of 7 and 2.4 mg/l respectively (table 7).

At the same time, TSS and VSS almost doubled in the HRAP with the bulk of VSS being made, for more than 95%, of algae cells. Nutrient removal and algae growth are then positively correlated. Algae cell uptake nutrients using solar energy to produce new cells whose composition and behavior are of course different from those of the sewage wastes.

Coincidently, the mechanisms behind N and P removal and algae growth are also at the origin of the exacerbation of pathogens lethal conditions. Both mechanisms are consequences of the algal photosynthetic activity.

Table 7. Treatment performance of the HRAP.

Parameter	Influent	Effluent	R R (%)	Increase (%)
pH	7.2	8.9		
CODt (mg/l)	110	250		66
BOD_5 (mg/l)	45	35	22	
SS (mg/l)	15	115		95
VSS (mg/l)	5	85		98
TKN (mg/l)	61	8.3	86	
Ammonia ($N\text{-}NH_4^+$) (mg/l)	49	7	86	
Total P (mg/l)	8	2.7	66	
Orthophosphate (PO_4^{3-})(mg/l)	5.8	2.4	59	
FC (log_{10}/100 ml)	4.6 E5	2.7 E4	1.23*	

* RR: removal rate.

Figure 12 shows the nitrogen mass balance in the HRAP. The major component in the influent is ammonia, which accounted for 95% of total N. Such a high ammonia concentration was due to the organic nitrogen mineralization taking place in the reactors.

At the effluent side, ammonia proportion was reduced to 8%. The comparison with the influent shows that 39% of the nitrogen mass was immobilized as new algae material while 46% was lost to the atmosphere by ammonia stripping.

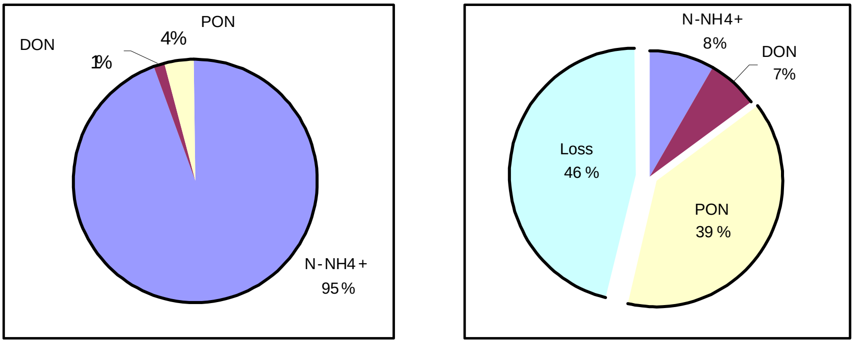

Figure 12. Nitrogen mass balance in the HRAP

(DON and PON stand for dissolved and particulate organic Nitrogen respectively)

Nitrogen stripping took place when ammonia (NH_3) was the dominating species in the medium. The transformation from NH_4^+ to NH_3 is governed by both pH and temperature in the pond. The species NH_4^+ is dominant at pH lower than 8 while almost all nitrogen is transformed into NH_3 at pH 11 (Minocha & Prabhakar Rao, 1988; Picot, *et al.,* 1991; Nurdogan & Oswald, 1995). The dependence of this process on temperature and on pH is shown by equation (4) which indicates that the NH_3 concentration is multiplied by 10 for one unit pH increase or by 2 for an increase in the temperature, equal to 10°C.

$$NH_3/ NH_4^+ = 10^{(10 - pH - 0.03 T)} \qquad (4)$$

The follow up of the pH in the HRAP (Figure 13) showed that NH_3 concentration was dominating and explained the nitrogen mass losses observed on figure 12.

For the fate of phosphorus, it was found that 90% of P in the influent was in PO_4^{3-} and that the effluent still contained 37% of PO_4^{3-} (figure 14).

Removal efficiency of P in the HRAP was much lower than for N. Algae uptake was only 25% while almost another 23% were lost by precipitation of phosphate salts under the effect of high pH (figure 15) see also (Nurdogan & Oswald, 1995; Moutin & *al.*, 1992; Mesplé & *al*., 1995; El Hafiane et al., 2003).

Tracing the fate of P in the HRAP, one may conclude that ¼ of the admitted P mass is lost by precipitation and another ¼ is assimilated by algae.

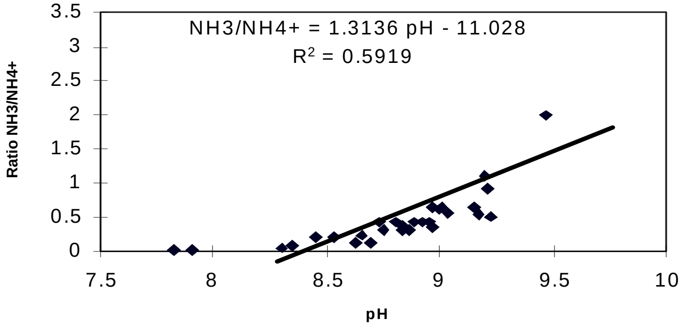

Figure 13. NH_3/NH_4^+ ratio for pH values recorded on the HRAP of the IAV.

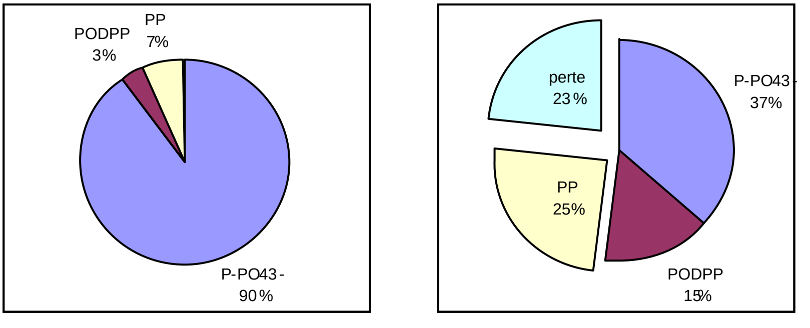

Figure 14. Phosphorus mass balance in the HRAP operated inside as a tertiary unit. (PP: particulate P; PODPP: dissolved organic P and polyphosphates)

Table 8. shows that the maturation pond also has an important role in the treatment. This pond acted as a polishing step in which almost 30% of BOD_5, COD and total Nitrogen were removed. Settling of algae at this stage of the treatment seems to be the main mechanism of such a polishing effect.

On the other hand, the increase in algae concentration shown in table 7 does not impair the effluent quality. BOD concentration decreases in the effluent of the HRAP while unfiltered or total COD doubled. The discrepancy between BOD and COD occurs because algae are oxidized in the COD test while they have no effect on the BOD as far as the samples are kept in darkness during the test.

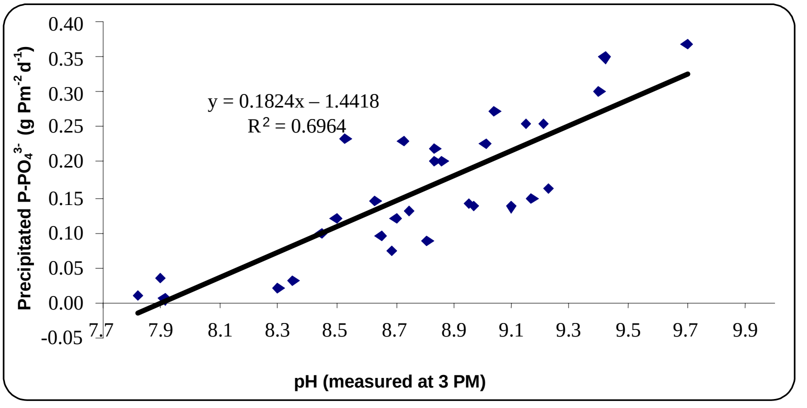

Figure 15. Predictable precipitated mass of orthophosphates in the HRAP of IAV

The presence of reasonable concentrations of algae in the effluent has no pollution impact on receiving media. Upon their disposal, algae will operate as immediate oxygen suppliers in the receiving water body and will constitute an available food for the protozoa and algae consuming fish.

This is why CODt measurements are meaningless in this case. Soluble or filtered COD better describes the situation. In Europe, algae contribution is subtracted from CODt for WSP plants effluents before checking their compliance with disposal standards.

Table 8. Role and performance of the maturation pond.

Parameter	**Influent**	**Effluent**	**RR (%)**
COD (mg/l)	250	170	32
BOD5 (mg/l)	35	25	28.5
SS (mg/l)	115	115	
TNK (mg/l)	8.3	6.0	28
Total P (mg/l)	2.7	2.4	10
CF (log10/100 ml)	2.7E+04	2.4E+03	1.05

*Removal rates in log Unit.

If the effluent is to be used for irrigation then this biological material is a humus source to improve the soil characteristics and texture. If advanced irrigation systems are to be adopted, our experience at the IAV shows that placing a sand filter and a screen behind the pressure pump is sufficient to allow a sprinkler system to operate without major troubleshooting.

3.2.2. Pathogens removal

3.2.2.1. Helminth egg removal

Helminth eggs are removed in the pre-treatment stage. They are mostly trapped in the reactors sludge blanket. After the gravel filter unit, helminth eggs were not detected in the samples.

3.2.2.2. Faecal coliforms removal

Faecal coliform (FC) removal occurs in all the components of the system cumulating a removal rate of 3.93 logarithmic units. The TSUAR, including the gravel filter, removes 1.7, the HRAP 1.23 and the maturation pond 1 Log unit.

It is of importance to highlight the salient performance of the maturation pond. This pond removes 1 Log Unit while the FC loading was much lower than those applied to the TSUAR and to the HRAP indicating that the maturation pond is playing a key-role in the disinfection phenomenon taking place in the ARHRPCT.

The settling and trapping (adsorption, flocculation, etc.) of bacterial cells or flocs explain most FC removal in the TSUAR. In the HRAP and in the maturation pond, similar mechanisms are at the origin of FC die-off. In these two ponds, strong variations of pH and DO concentrations between extreme values are governed by algal photosynthesis (high in the day, absent in the night). These sharp variations play a key-role in FC die-off (Pearson et al. 1987; Quin et al. 1991; Fernandez et al. 1992; El Hamouri et al., 1994).

4. Design and operation

4.1. Design of the reactors

4.1.1. Reactor volume

The reactor volume is the main design parameter. It controls the HRT and hence the efficiency of organic digestion in the reactor and has a direct effect on the cost of the project. TSUAR design should be based on a reactor volume allowing a minimum retention time of 45 hours or roughly 2 days.

4.1.2. Geometric shape and construction material

TSUAR reactors may have a rectangular or a circular shape. Circular shapes offer more stability and are more economical. Circular shape could be recommended for small reactors while for large and multi-reactors units the rectangular shape is advantageous. Construction material is reinforced concrete.

Care must be given to coat the upper part (1m) with protecting and anti-acid paint. Although oxygen concentration inside the reactor would be low, there would be still enough O_2 to support the bacterial oxidation of H_2S to sulphuric acid, which attacks the concrete or any other material. This type of protection is also necessary for the upper metre of the settling tank.

4.1.3. Reactor depth

As for every system, avoiding pumping is of common economical sense. If topographic conditions do not permit feeding by gravity, then it is recommended to try to minimize pumping by changing the ratio of the part to be constructed over ground and that to be buried. Partial burying of the reactor reduces the cost, but care must be given for this approach when groundwater is near the surface (buoying).

The depth (or height) is related to the reactor performance, the deeper the reactor the better the performances. The depth controls the upflow velocity (equation 5). High values are beneficial; they permit a better contact of sludge anaerobic bacteria with the incoming influent (substrate). However, excessive upflow velocity values may lead to washout and reduction of the sludge concentration in the reactor.

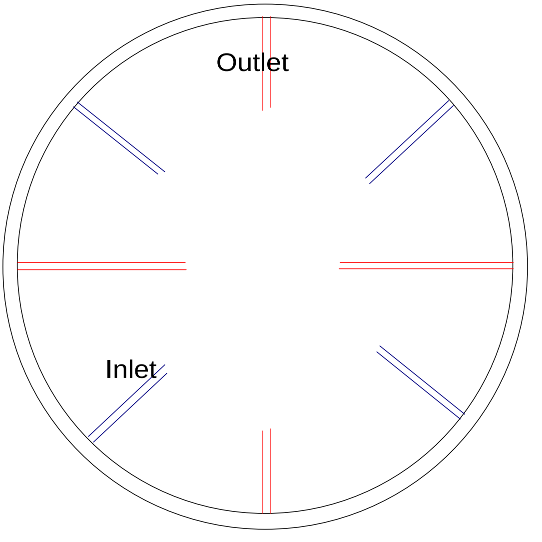

figure 16. Influent distribution and effleunt withdrawal pipes

Recall that the reactor sludge content controls the rate of digestion in the reactor. The average daily upflow velocity should not exceed 1 m h^{-1}

V = Qi/A = Vr/HRT*A = H/HRT (5)

V: Liquid upflow velocity (m/hour); Qi: The average daily flow (m^3/d) A: surface area of reactor (m^2); Vr: The reactor volume (m^3); H: height (depth) of the reactor (m); HRT; hydraulic retention time (hours).

4.1.4. Influent distribution and effluent collection devices

Homogenous distribution of the influent at the bottom of the reactor is an essential element in the design. Design of inlets and outlets obeys basic hydraulic open channel rules and flow-splitting devices are used to have equal feeding rates.

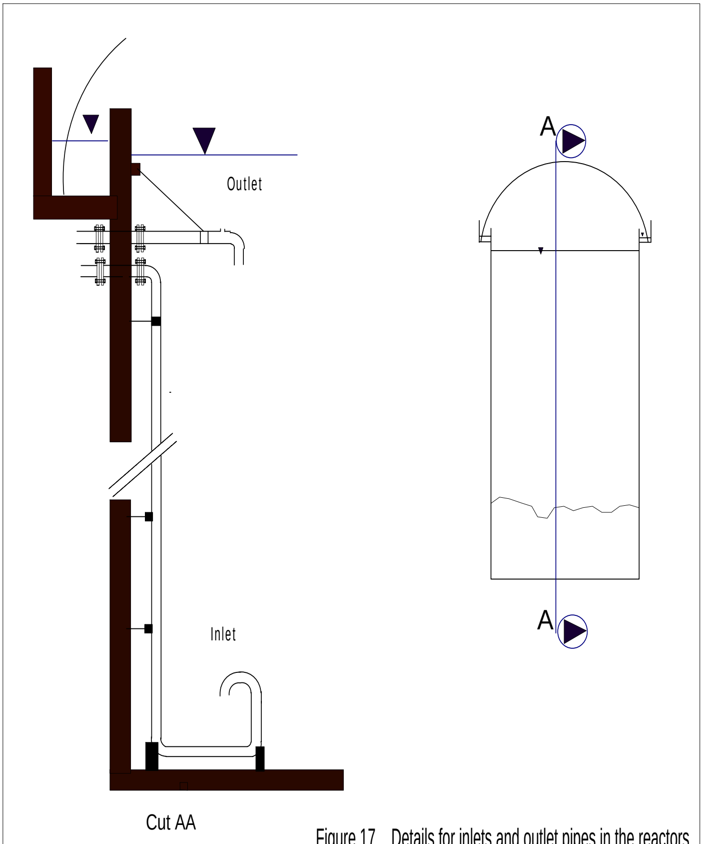

Figure 17. Details for inlets and outlet pipes in the reactors

The effluent discharge at the top of the reactor is also important for keeping the water level at the top of the reactor and for avoiding short-circuiting in the reactor (figure 16 and 17). Design of influent distribution and effluent withdrawal devices is easy for small units but may become a headache for larger units.

The channel was filled with the effluent from the post-treatment unit to act as a water seal to prevent biogas and offensive odours escaping from the reactor. The water in the seal was replaced with freshly treated effluent every week.

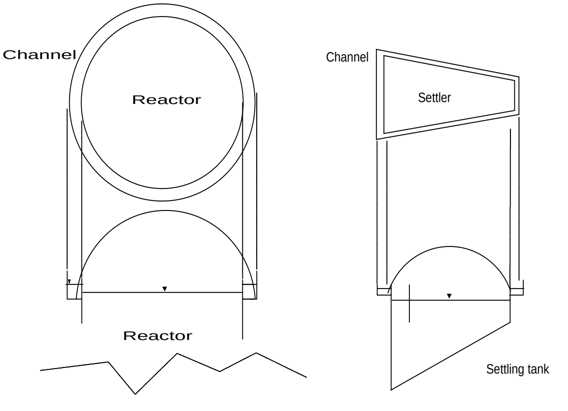

Figure 18. Water-seal devices of the TSUAR and reactor covers designed to collect biogas and to prevent offensive odours emanations.

4.1.5. Biogas collection and anti-odour system

Biogas is collected from the reactors using hard, external cupola-shaped covers made of acid-resistant polyester material. The base of the covers is inserted into a (0.40 m width x 0.40 m depth) channel surrounding the reactors' external wall (figure 17 and 18).

4.2. Design of the HRAP

The carrousel-like pond called the High-rate algal pond (HRAP) is a photosynthetic reactor, in which microscopic, photosynthetic algae are living together with heterotrophic bacteria.

Water inside the pond is kept in a closed circuit and is gently mixed by a paddle wheel (6 to 8 RPM) using an electrical motor affixed to a gearbox that moves water at a surface speed of 0.2 m/s. The main object of this mixing is to prevent algae from settling in the pond giving them the opportunity to be exposed to sun light at regular intervals.

The HRAP is an efficient photosynthetic reactor where a high amount of oxygen is evolved leading to concentrations of dissolved oxygen (DO) as high as three times the pure water saturation value. Parallel to this, intensive uptake of CO_2 increases the pH values as high as 8.5 to 9. DO and pH curves show parallel trends. Both parameters usually reach their maximum around 14:00 (El Hamouri *et al.*, 1994; El Hamouri *et al.*, 1995).

The use of the HRAP as a tertiary treatment unit is argued in section 1. The design of the HRAP under a tertiary configuration is based on an average HRT of 3 days maximum. If disposal limits for N and P are mandatory, then the design should be based on the kinetics of N and P removals.

The first order reaction rate constants, k 20°C for the removals of N and P were determined on the HRAP of the IAV (table 1). They allow calculation of the HRT requested to meet the disposal concentrations following the plug flow equation (equations 6 and 7) which we have shown as the most likely to describe removal kinetics in the HRAP (El Hamouri *et al,* 2003) (see also annex 1)

$$Se/Si = e^{-kt} \qquad (6)$$

With, Se and Si : concentrations of TKN in the effluent and in the influent respectively; k : first order reaction rate constant for N or de P removal and t retention time in days. $k_{20°C}$ can be calculated using equation (7)

$$k_{T°C} = k_{20°C}\ \theta^{(T°C\ -20)} \qquad (7)$$

Where, $k_{T°C}$ and $k_{20°C}$ are k values at T°C an at 20°C respectively and θ, Arrhenius constant whose value is 1,047.

4.2.1. Geometrical shape

The shape of the HRAP, its depth and the width of the raceways are well known. The reader could find details on this matter in Oswald's publications (Oswald, 1998).

4.2.2. Paddle wheel

The paddle could be made of used conveyer belt purchased from industrial second hand stock. (figure 19). Details of the construction are given in annex 6. The electric motor and the gearbox for speed reducing unit used at the IAV plant have the following characteristics:

Motor; 380/400 V; cos f = 0.75; power: 0.37 kW; 1380 RPM at 50 Hz.
Gear box for speed reduction unit: i= 100; speed = 13.8 rounds per minute (RPM).

Choice of the chain sprocket was to give a final speed of 5 to 6 RPM.
The motor and the speed reducing unit (see picture) used were purchased from industrial second hand stock and adapted by the local machining shop

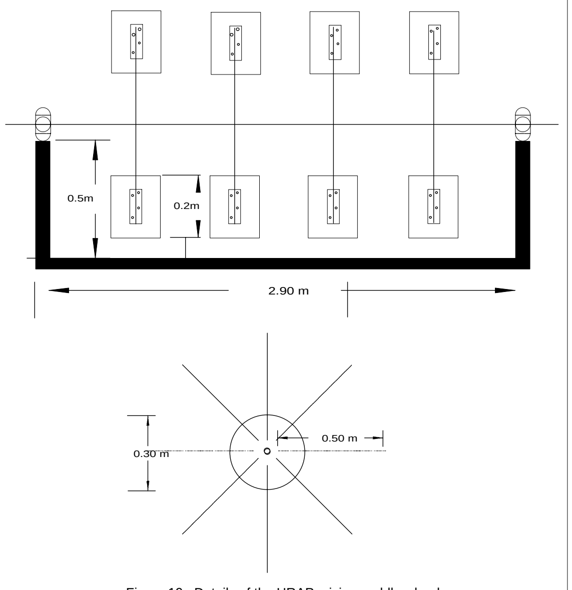

Figure 19. Details of the HRAP mixing paddle wheel

4.3. Design of the maturation pond

Design of the maturation pond is to be based on the Marais's equation for faecal coliform removal. This design method is very common and is found in the literature (see annex 1 for design example).

4.4. Operating parameters

This part refers to the experience gained from operating the IAV plant in Rabat, Morocco for more than 5 year continuously.

4.4.1. Sewage characteristics and volumetric loading rate

The strength of wastewater collected from the IAV campus was moderately concentrated. The volumetric loading rate was in the range of 0.5 to 0.7 kg COD $m^{-3}d^{-1}$. Loading rates exceeding 1 kg COD $m^{-3}d^{-1}$ were even applied for some periods and were well supported by the system.

Sewage is very strong in many countries of the MENA region due to the scarcity of water and to repetitive interruptions in drinking water supply. For example, Sana'a sewage has 1,500 mg/l of BOD. Even such high concentrations would not represent a handicap for the TSUAR as anaerobic reactors are reputed to tolerate highly concentrated sewage when compared to anaerobic ponds.

Regarding the chemical composition of the sewage, anaerobic bacteria need minimum concentrations of nitrogen and phosphorus to sustain their growth and to achieve optimal treatment performance. A minimum value of 100/1.25/0.17 is recommended for the ratio COD/N/P. Domestic sewage can easily provide these N and P concentrations and no need for external supply is requested. Higher concentrations than those recommended would not impair the performance of the reactor.

4.4.2. Sludge mass and sludge age

Both sludge mass and age (or solid retention time) are directly linked to the reactor performance. The idea behind the control of sludge mass and age is to optimise the contact of the anaerobic digestion bacteria or clusters of bacteria with the incoming influent waste. Sludge age in the reactor is determined following equation (8).

$$As = Xr / (Xd + Xe) \qquad (8)$$

Where, *As* is the sludge age (day); *Xr*, the amount of sludge hold in the reactor expressed as kg TSS; *Xd* and *Xe* are respectively daily amount taken out with the effluent and daily discarded (or removed) from the settler expressed as kg TSS/d.

Sludge specific production is given in kg of TSS or VSS per kg COD, BOD or TSS admitted (or removed) in the reactor. At the IAV, the TSUAR generated 0.22 g TSS g^{-1} COD admitted or 0.28 g TSS g^{-1} COD removed. The VSS/TSS ratio for produced sludge was 0.53 and the specific sludge production was found to be 4 kg per $capita^{-1}$ $year^{1}$.

4.4.3. Sludge profile in the reactor

The sludge profile in the reactor depends on the sludge age and on the upflow velocity. The profile must show the formation of a sludge blanket in the bottom followed by a decreasing sludge concentration gradient towards the top of the reactor. The average sludge profile obtained in the TSUAR at the IAV plant is seen in figure 20.

At the IAV, a sludge blanket was formed in the bottom of the two reactors. The sludge layer was 1 m in reactor R_1 and 2 m in R_2. However, the concentration of the sludge was higher in R_1.

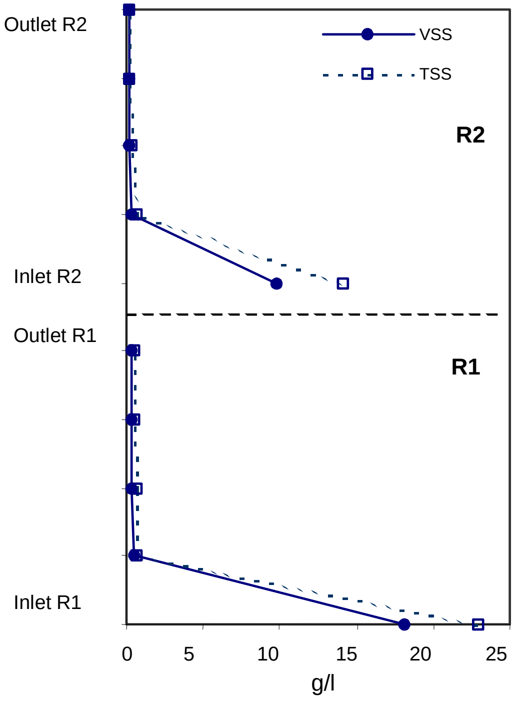

Figure 20. Sludge profile in the IAV reactors.

4.4.4. Sludge withdrawal from the reactors

The choice not to manually or mechanically withdraw excess sludge, from the reactors for operation simplicity, made it necessary to adopt the “maximum sludge hold up” mode (van Haandel & Lettinga, 1994). Following this mode, sludge accumulates in the reactors until a maximum concentration is reached in the reactor. This maximum is followed by an episode of sludge washout. The washout period is then followed by a period of sludge accumulation during which sludge washout is minimal.

The duration of a washout/accumulation cycle depends on the season and the campus activities, with an average solid retention time in the reactors of 32 days. Monitoring of the sludge thickness showed, however, that the beds never fall under 1 meter in the bottom of both reactors.

At the IAV plant, no sludge has been withdrawn from reactors R_1 and/or R_2 since 1997. Excess sludge leaving the reactors with the effluent was trapped in the settling tank. Sludge withdrawal from the settling tank consisted of a 20-minute, daily task. Daily, the worker in charge of the plant operation and maintenance opened the valve located in the bottom of the settling tank.

Hydrostatic pressure helps in conveying the thickened sludge to the drying beds. Sludge volume discarded daily varied from 60 to 100-litre d^{-1}. Dry matter content of the removed sludge amounted 2% in which 53% was organic.

Stability tests, applied to the sludge removed from the settling tank, showed that 7% of removed mass was still able to produce methane, meaning that the sludge expelled by the TSUAR was stabilised to 93%. For this sludge, the ratio VSS/TSS was approximately 0.53; a sludge with a ratio higher than 0.7 needs further stabilization treatment steps.

4.4.5. Sludge drying beds

Productivity, P_d of the sludge drying beds[2] is defined by equation (9)

$$P_d = F_s / T_t \qquad (9)$$

Where P_d is the bed productivity (kg of dry matter m^{-2} d^{-1}); Fs, the bed solid load (kg TSS m^{-2}) and Tt**,** the total time necessary for a drying cycle (d).

At the IAV plant, a bed productivity of 0.75 kg dry matter m^{-2} d^{-1} was obtained for product containing 90% dry solids. By comparison, at Pedregal, Brazil, a productivity of 1.4 kg dry matter m^{-2} d^{-1} was reported for a series of beds associated to a UASB (van Haandel and Lettinga, 1994). The difference is due to the more favourable tropical climate conditions in Brazil.

Under Rabat's conditions (Mediterranean climate) and using a solid load of 1.5 kg TSS m^{-2}, the relative moisture of the sludge decreased from 97 to 10% within 10 days in the cold and rainy season and within 3 days in the hot season. In rainy conditions, beds could be covered using polyethylene film used for greenhouse protection.

[2] Drying beds design is included in the excel based software developed for the ARHPCT (Annex 1).

5. Personnel and procedure for Operation and maintenance

5.1. Personnel qualification

The ARHPCT belongs to a new category of systems to be called "high concept-low cost" technologies. Anaerobic reactors are part of these technologies for example. The paradigm following which extensive treatment systems do not need qualified personnel and permanent attention for their operation and maintenance does not hold true for these types of technology.

It is of importance that a qualified team could be designated for the operation and the maintenance of the ARHPCT. In return, it does not need a high input in terms of labor for the regular maintenance of the plant. One permanent worker was coping with routine and basic daily operation at the IAV plant, which serves 1,000 inhabitants. There would be no need to hire additional labor if the size is to be increased to serve 5,000 inhabitants.

Personnel trained for WSP or Constructed Wetland operation and maintenance could easily take in charge a plant based on ARHPCT. No particular skill is requested for this ARHPCT.

5.2. Operation and maintenance (O&M) procedure

A model of logbook is presented in annex 2 in which main O & M operations must be written down on a daily basis. The logbook should also hold track equipment failure and replacement, component flooding or any particular event.

5.3. Troubleshooting

A step-by-step troubleshooting manual for malfunctions remediation is presented in annex 3. This procedure could help the operator in obtaining optimal and sustainable treatment performance.

6. Conditions of applicability of the ARHPCT

The development of the ARHPCT is to be understood based on the recent general trends aiming at the development of sustainable water management concepts. In this respect, decentralised sanitation constitutes one of the fundamental approaches. The ARHPCT is an environmentally friendly technology. It prevents methane emission into the atmosphere to reduce the greenhouse effect. Its low-cost, minimal land area requirements and its simplicity of operation designate it as a potential key-structure in implementing water recycling and reuse concepts for small communities.

6.1. Water scarcity and quality

Water scarcity is a central issue in the MENA region, mainly because most of the countries in this region are located in warm and dry climates and because these countries are all facing great water demands to satisfy their rapidly growing industrial and agricultural sectors.

More important is to provide the growing population with drinking water. However, provision for drinking water of good quality on a regular basis does have a high cost. In this respect, the quality of the raw water is a key factor. The final cost of drinking water production increases with decreasing water quality in the reservoirs and rivers. Low-cost and sustainable systems could be generalised in little settlements and small or mid-sized centres to protect water resources from pollution and to reduce the cost of drinking water production.

Construction of conventional sewerage in dispersed habitat is costly. Decentralized approaches combined with adapted collection systems, such as small-bore gravity sewer system (SGSS) could be combined with the ARHPCT in many instances to help implementing projects with reasonable costs.

The sector of tourism is important for many MENA countries (Egypt, Lebanon, Tunisia, Syria, Jordan and Morocco) where the historical, archaeological attractions or simply the sights are often found in remote sites. At the same time, hotels often need large and attractive green areas. The ARHPCT could be recommended for such situations. It would help in solving the problem of providing adequate sanitation and water for landscaping. The ARHPCT is well adapted for wastewater reuse projects particularly regarding its effluent quality that respects both environmental (N and P final concentration) and sanitary standards (absence of helmenth eggs and relatively low faecal coliform content).

6.2. Target groups for potential implementation of ARHPCT

Table 9. Target groups for potential implementation of the ARHPCT (Morocco as an example).

Target groups	Numbers /Geographical Location	Structure, organization Institutional statute	Interests and Expectations	ARHPCT will help in
Municipalities and elected local councils	Municipalities and rural communes (from 500 to 20,000) ,mainly located Inside the country Population: 10 millions (45% connection to sewerage and no treatment facilities.	In charge of drinking water and sanitation in their areas. Receive subventions from the government and can mobilize loans from the FEC (Communal equipment fund). The population contribution to the municipal services is very limited.	Expect more funds from the central government, to cope with the investment and O &M costs. Attracted by cheap and affordable sanitation projects. Have little access to innovative technologies.	Implementing small to mid-sized projects at affordable cost and within reasonable times.
National Drinking Water Agency (Office national d'eau potable, ONEP)	-	In charge of the drinking water production. Water distribution and sanitation for small and mid-sized centers.	Implementing sustainable projects. Expecting substantial population contribution to the investment and O&M costs of sanitation	The cost of sanitation projects for small communities could be reduced by adopting innovative

Target groups	**Numbers /Geographical Location**	**Structure, organization Institutional statute**	**Interests and Expectations**	**ARHPCT will help in**
				technologies.
Private sector, tourist sector and house building companies	-	Small and medium sized enterprises and some large companies	Expecting an increasing role in the sector to introduce competitiveness and higher service quality.	Introducing innovative technologies, recycling and low-cost sanitation. More people can have access to sanitation at affordable fees.
Ministry of Environment, Ministry of interior and Ministry of public works	Central Directorates, Watershed agencies, and their regional representative	Decision making, impose fees for sanitation services and planning, projects funding	Elaborate strategies for pollution control, conservation of natural resources and sustainable development,	Gain ability to adopt innovative technologies, development of non-conventional water use.

6.3. Climate conditions

The performance obtained with the ARHPCT at the IAV in Rabat, in a Mediterranean climate, could be improved in countries with mild winters.

6.4. Recommended infrastructure conditions

- Existence of a sewage collection network (conventional or small bore gravity sewer)
- Availability of an electrical or a Diesel power unit.

However, power failures even for long periods (a week for example) would not jeopardize the treatment efficiency in the AHRPCT except when the sewage has to be pumped due to topography.

6.5. Financial resources for operation & maintenance

The success of any maintenance program depends on the availability of funds dedicated to this purpose and on the availability of local qualified labour. Often, it is the lack of an annual O & M budget and the absence of qualified and well-motivated personnel that explain the collapse of many wastewater treatment plants. Failure of adopted technologies is rarely at the origin of unsuccessful stories.

7. Operation and maintenance costs

Average O & M costs for the ARHPCT were calculated from a five-year period of continuous operation and maintenance of the IAV plant constructed for 1000 p.e. Costs of pumping for conveying wastewater inside the campus are not considered here (table 10).

The cost falls within the range of those reported for WSP plants of similar sizes except for sludge withdrawal, which is very low for the ARHPCT. Items 1 and 4 would not significantly change within a range of 1000 up to 5,000 p.e.

8. Effluent reuse

ARHPCT Treated effluent has been used for landscaping inside the IAV campus since 1997. A small part is used for agronomical tests by some research workers. However, for the MENA region the emphasis should be put on reuse for crop production rather than on landscaping.

In this respect, the WHO's health guidelines for the use of wastewater in agriculture and aquaculture published in 1989 have been adopted in many countries of the MENA region. Following these guidelines, category A effluents can be used for unrestricted crop irrigation while B and C categories cannot be used to produce uncooked, edible crops (see table 11 ; WHO, 1989).

The ARHPCT could be adapted to both situations depicted above and could even offer reliable protection to sensitive water bodies. The technology provides a large flexibility allowing it to meet stringent quality standards or to satisfy limited environment protection goals dictated by low funding capabilities and/or by less stringent local disposal standards.

Annex 4 summarizes a case study related to a new approach for valorising treated effluent using flower production which discard the consumer-linked health concerns while still producing a high-value crop.

Table 10. Annual operation and maintenance costs for a 1000 p.e. plant.

Item	**(US $)**
1. Personnel	
01 worker	3,600
01 Gardner (½ time) to take care of the landscape	1,800
01 Night watchman	4,800
01 Technician (¼ time)	3,600
Sub-total	13,800
2. Electrical power	1,428
3. Maintenance	
Screening unit	600
Valves and other devices etc.	300
Pumps maintenance and repair	200
Paddle wheel motor & transmission mechanism	200
Works (walls, walkways etc.)	1,000
Polyester covers (painting every two to three years)	300
Plant painting	400
Sub-total	3,000
4. Water analysis	800
Total	**25,828**

Table 11. WHO guidelines for the use of treated wastewater in agriculture.[a]

Category	Reuse conditions	Exposed group	Intestinal nematode[b] (arithmetic mean no. eggs per litre)[c]	Faecal coliforms (geometric mean no. per 100ml)[c]	Wastewater treatment expected to achieve the required microbiological guideline
A	Irrigation of crops likely to be eaten uncooked, sports fields, public parks[d]	Workers, consumers, public	< or = 1	< 1000	A series of stabilization ponds designed to achieve the microbiological quality indicated, or equivalent treatment
B	Irrigation of cereal crops, industrial crops, fodder crops, pasture and trees[e]	Workers	< or =1	No standard recommended	Retention in stabilization ponds for 8-10 days or equivalent helminth and faecal coliform removal
C	Localized irrigation of crops in category B if exposure to workers and the public does not occur	None	Not applicable	Not applicable	Pretreatment as required by irrigation technology, but not less than primary sedimentation

[a] In specific cases, local epidemiological, sociocultural and environmental factors should be taken into account and the guidelines modified accordingly.
[b] *Ascaris* and *Trichuris* species and hookworms.
[c] During the irrigation period.
[d] A more stringent guideline (200 faecal coliforms per 100 ml) is appropriate for public lawns, such as hotel lawns, with which the public may cone into direct contact.
[e] In the case of fruit trees, irrigation should cease two weeks before fruit is picket, and no fruit should be picked off the ground. Sprinkler irrigation should be used.

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ANNEX

ANNEX 1

Design and drawings

Annex 1.1. Design example of ARHPCT

Design of the ARHPCT (example for a settlement of 6,000 inhabitants in Sana'a, Yemen)
(Salient characteristic: High COD concentration)

Excel based software available on the Website: http://www.iav.ac.ma
Institut Agronomique et Vétérinaire Hassan II
Department of rural engineering

Population and sewage

Population	person	6 000
Daily consumption per capita	m^3/person*day	0,050
Flow	m^3/day	300
Total COD of the sewage	g/m^3	1 600
Daily applied COD	kg/d	480
Suspended solids (SS)	g/m^3	600
Daily applied SS	kg/d	180
NTK	g/m^3	72
Daily applied NTK	kg/d	22
Total P	g/m^3	8
Daily applied P	kg/d	2
Faecal coliforms concentration	U/100ml	10 000 000

PRE-TREATMENT

1. Two-step upflow anaerobic reactor (TSUAR)

1.1. Reactor R1

Hydraulic retention time (HRT)	d	1,0
Volume	m^3	300
Depth	m	8
Diameter for total flow	m	7
Number of reactors in parallel	U	2
Area/Reactor	m²	19
Diameter per reactor	m	5
Area/inlet	m²	4,3
Number of inlets	U	4
Total area for R1	m²	38
Up flow velocity (max 0,7)	m/h	0,33
Volumetric loading rate of reactor R1	kg/m^3*d	1,6

1.2. Reactor R2

Hydraulic retention time (HRT)	d	1,0
Volume	m^3	300
Depth	m	8
Diameter for total flow	m	7
Number of reactor in parallel	U	2
Area/Reactor	m²	19
Diameter per reactor	m	5
Area/inlet	m²	4,3
Number of inlets	U	4
Total area for R2	m²	38
Up flow velocity	m/h	0,3
Volumetric loading rate of reactor R2	$kg/m^3/d$	1,1

1.2. Settling tank (to trap suspended material escaping reactor2)

Remaining TSS concentration	g/m3	300
Adopted flow 2 times normal flow	**m^3/d**	**600**
Overflow rate (settling velocity)	**mh^{-1}**	**1,5**
Area (A)	m²	17
Number of settler	U	1
Area per settler	m²	17
Depth average (2m-entry 1m end) slope 1/1,75	m	1,7
Volume per settler	m^3	28
Width per settler	m	2
Length per Settler	m	10
HRT	d	0,03
Residual SS in the effluent	mg/l	45
Removed TSS mass	kg/d	77
Volume of thickened sludge evacuated[1]	m^3/d	0,8
Concentration of removed sludge	kg/m^3	100

[1]Based on Rabat measurements

Expected performance of TSUAR

COD removal rate Empiric equation $(Ce/Ci)=100*(1-C_2*24*HRT\wedge^{-(C1)})$*

Constant C1		0,56
Constant C2		1,82
COD removal rate	%	79
Residual total COD	mg/l	333
Residual NTK	mg/l	72
Residual P	mg/l	8,2
Residual SS	mg/l	45
Total HRT	d	2,03
Biogas production (77% de methane)	m^3/d	36

* where Ce and Ci effluent and influent CODt concentration ; C1 and C2 design constants ; HRT : hydraulic retention time (h).

1.3. Sludge drying beds

Specific sludge production	**kgTSS/kg COD**	**0,22**
Daily sludge production	kgTSS/day	106
Bed productivity	**kg TSS/m^2*d**	**1,5**
Bed area	m²	70
Security coefficient	20%	1,2
Bed area final	m²	84,5
Bed loading rate	**kg TSS/ m^2**	**10**
Duration of one cycle	d	7
Divide the area among 7 beds of equal area	m²	12

Evacuate sludge from settling tank every day early morning and for drying on beds in which it stays 7 days for a complete cycle		
Number of beds	U	7
Sludge thickness in bed per operation	m	0,08
Area per bed	m²	12
Depth of bed	m	0,7
Width	m	2
Length	m	6
Add a sludge storage basin (2 m width*6 m length)	m²	12
Total drying bed area	m²	97
Construction depth	m	1,0
Gravel depth	m	0,3
Sand depth	m	0,4

1.4. Gravel filter

Hydraulic load * Rabat experience	**m/d**	**0,95**
Requested area	m²	315
Width	m	12,5
Length	m	25,1
Depth (0,8 m gravel 10 mm * 5 mm)	m	0,8
Void volume	m^3	101
HRT	d	0,34
Applied SS concentration	mg/l	45
SS removal rate	%	80
Residual SS concentration	mg/l	**9**
Residual total COD	mg/l	**33**
Residual Helminth eggs	egg/l	**Zero**

POST-TREATMENT

2. High rate algal pond (removal of Nitrogen & Phosphorus)

First order constant k tCOD removal @ 20°C ***	d^{-1}	**-0,245**
First order constant k total N removal @ 20°C	d^{-1}	**0,653**
First order constant k total P removal @ 20°C	d^{-1}	**0,249**
HRT	d	3
Depth	m	0,40
Area	m²	2 250
Water volume	m^3	900
width	m	27
Length	m	82
Number of ways (even number only 2 , 4 , 6 or 8)	Unit	4
Residual total COD****	mg/l	61
Residual soluble COD	mg/l	25
Residual soluble N (C43*2,3^-(C97*C102))	mg/l	14
Residual soluble P (D113*2,3^-(D115*C102))	mg/l	4

3. Maturation pond (Marais's assumption of a complete mixed reactor)

Daily applied COD	kg/d	7
Surface loading rate	**kg COD/ha*d**	**100**
Requested area	m²	737
Water depth	**m**	**1**
Water volume	m^3	590
HRT	d	2
Width of the basin	m	19
Length of the basin	m	38
Residual total COD***	mg/l	55
Residual soluble COD	mg/l	22
Residual soluble N (C43*2,3^-(C97*C102))	mg/l	13
Residual soluble P (D113*2,3^-(D115*C102))	mg/l	4
FC concentration in the effluent	U/100 ml	18 286

**** Negative value (see Document foe explanation)

**** Increases due to algae growth

4. Total requested area (water area)

Total HRT	d	7,3
Water requested area	m²	3 439
Ways and facilities (20%)	m²	688
Total requested area	m²	4 126

Faecal coliforms removal

Application of Marais's formula

$Ni/N_0 = 1/(1+k_T*t\ tsuar)(1+k_T*F)(1+k_T*t\ hrap)(1+k_T*t\ mp)$

K_T : first order decay constant ; t : HRT ; N et No : faecal coliforms concentration inlet and outlet respectively ; K_T : first order decay constant ; t : HRT

Feacal coliforms concentration in influent, Ni	UFC/100 ml	10 000 000
HRT in TSUAR	d	2,0
HRT in the gravel filter (GF)	d	0,3
HRT in HRAP	d	3,0
HRT in maturation pond	d	2,0
First order FC decay constant, K_T for TSUAR*	d^{-1}	0,75
First order FC decay constant, K_T for GF*	d^{-1}	0,37
First order FC decay constant, K_T for HRAP*	d^{-1}	3,70
First order FC decay constant, K_T for MP*	d^{-1}	7,60
FC concentration in the effluent	U/100 ml	18 286

*determined at IAV plant

Details of calculations

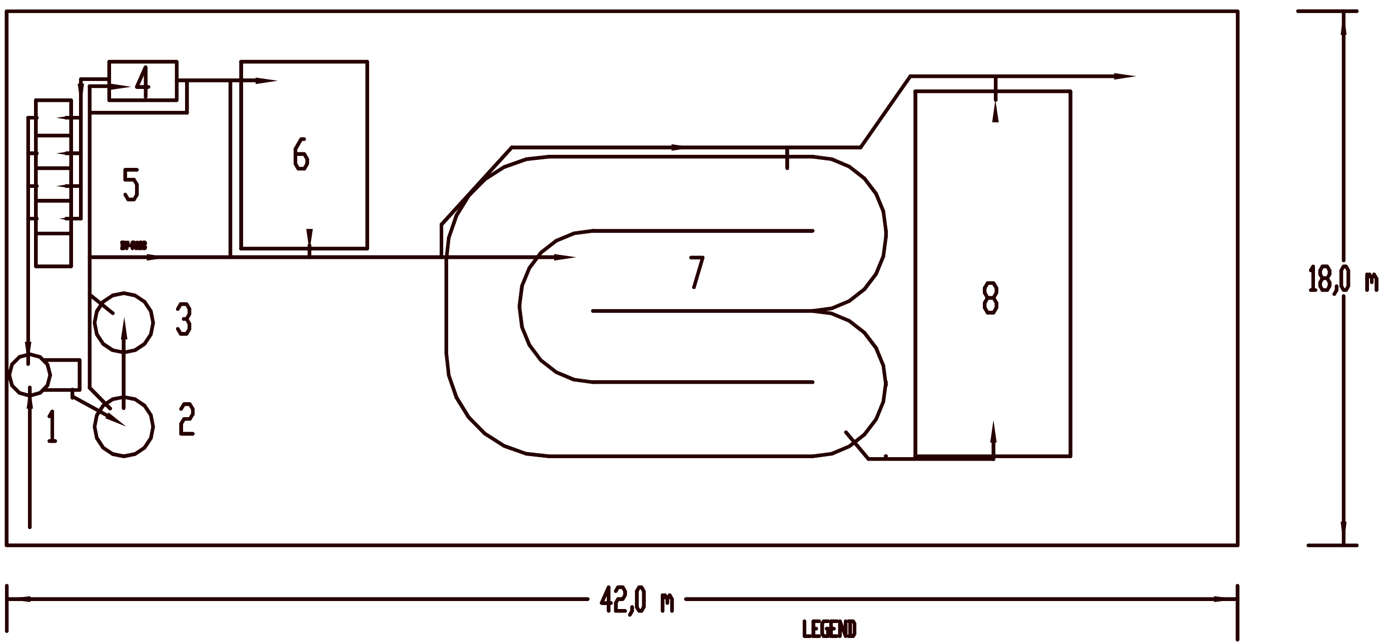
1+0,75*2		2,5
1+0,3*0,37		1,1
1+2,98*0,3		12,1
1+0,19*5,3		15,9
$N = N_0/[(1+0,75*2)*(1+0,3*0,37)(1+3,7*3)(1+7,6*2)]$		18 286
Category based on 1989 WHO guidelines		B

Category A =0 heminths eggs <1000 FC/100ml
Category B =0 heminthseggs >1000 FC/100ml

Assumed parameters are in bold

Annex 1 (continued)

Annex 1.2. Drawings example for 500-inhabitant facility

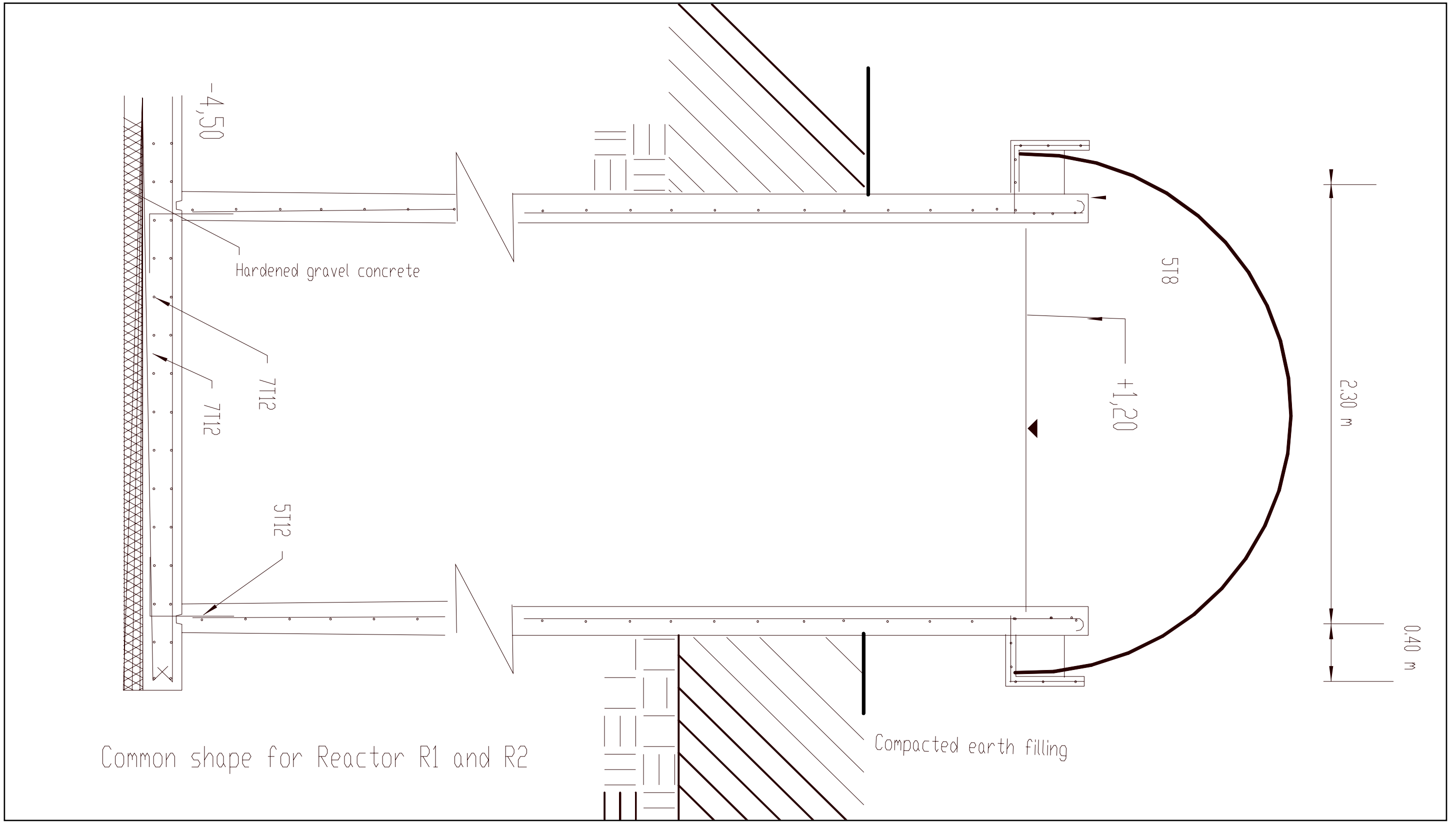


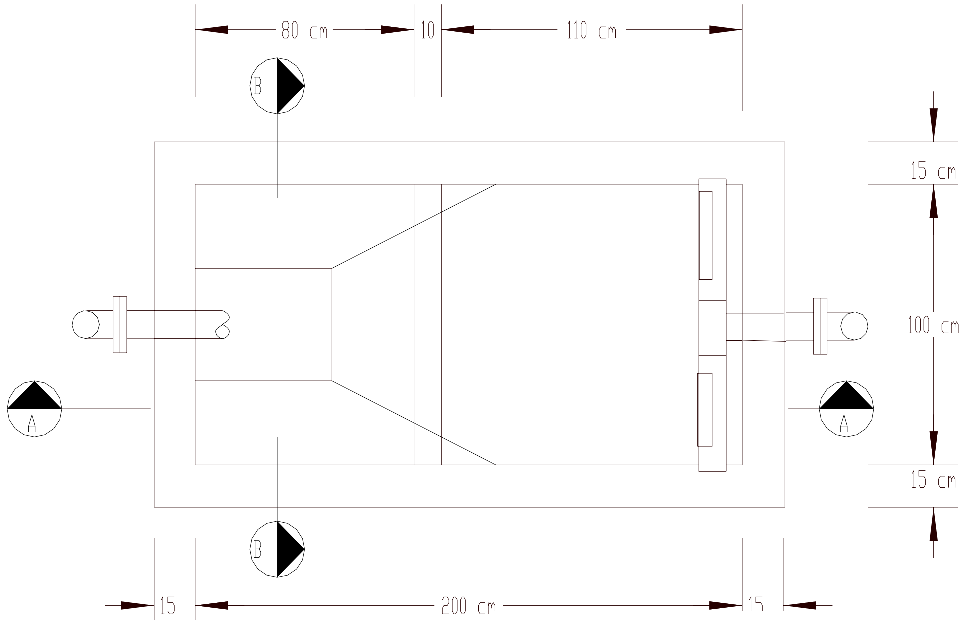
Site plan

Ait Faska wastewater treatment plant (500 equivalent-person)

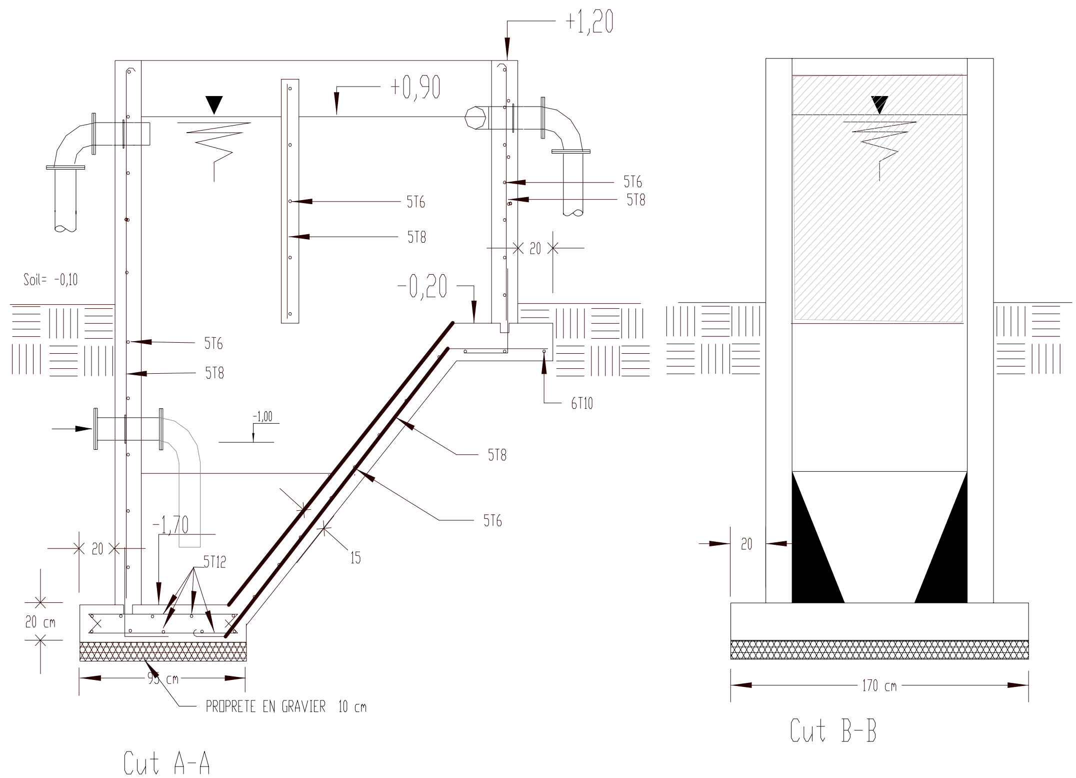
LEGEND

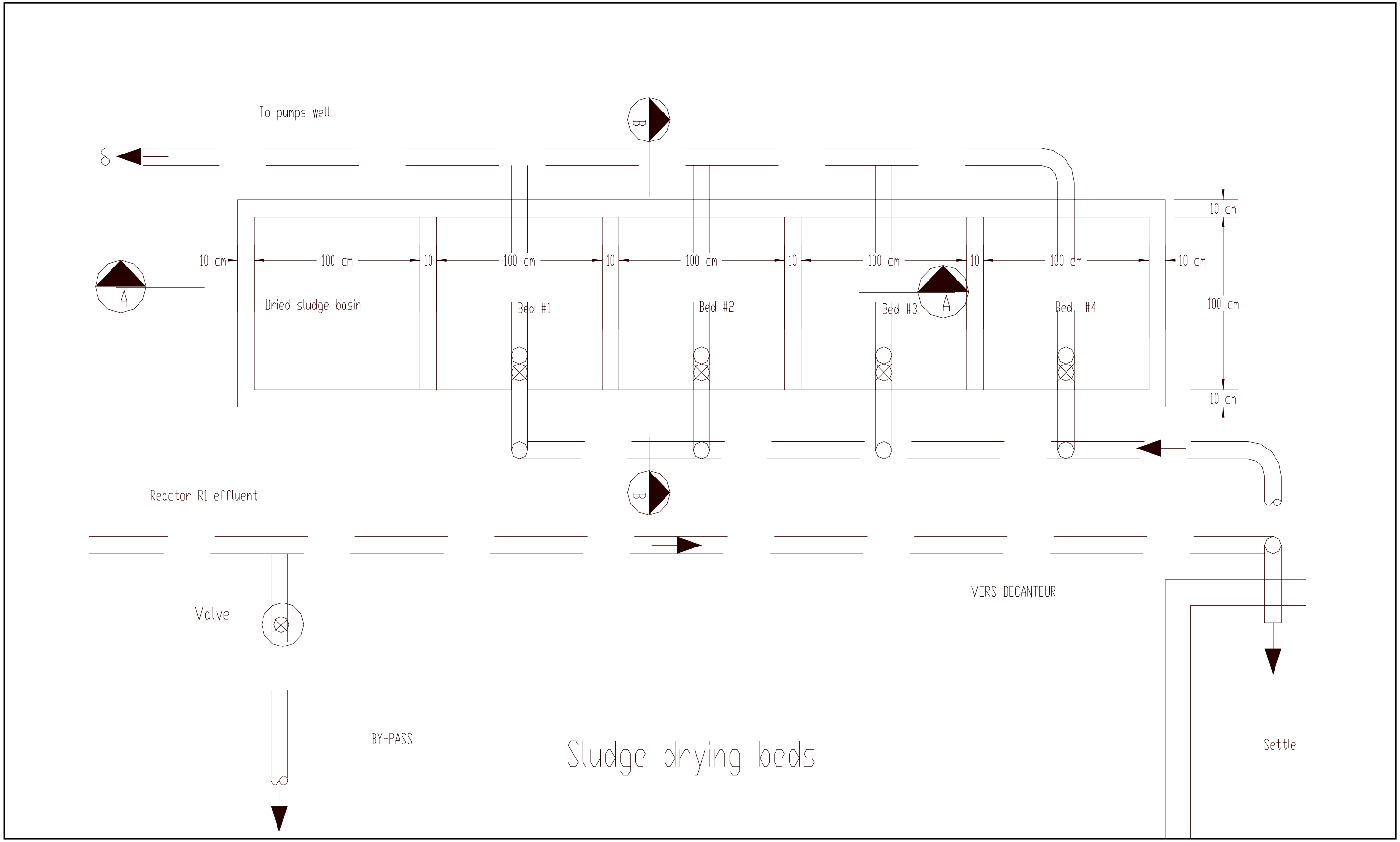
1. Pumps well
2. Reactor R1
3. Reactor R2
4. Settler
5. Sludge drying beds
6. Gravel filter
7. High-rate algae pond
8. Maturation pond


-4,50
Hardened gravel concrete
7T12
7T12
5T12
5T8
+1,20
2.30 m
0.40 m
Compacted earth filling
Common shape for Reactor R1 and R2

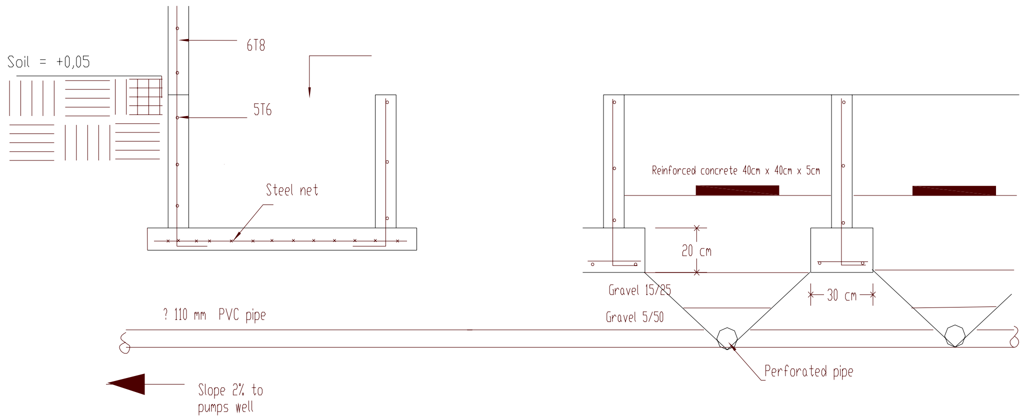


Settling tank

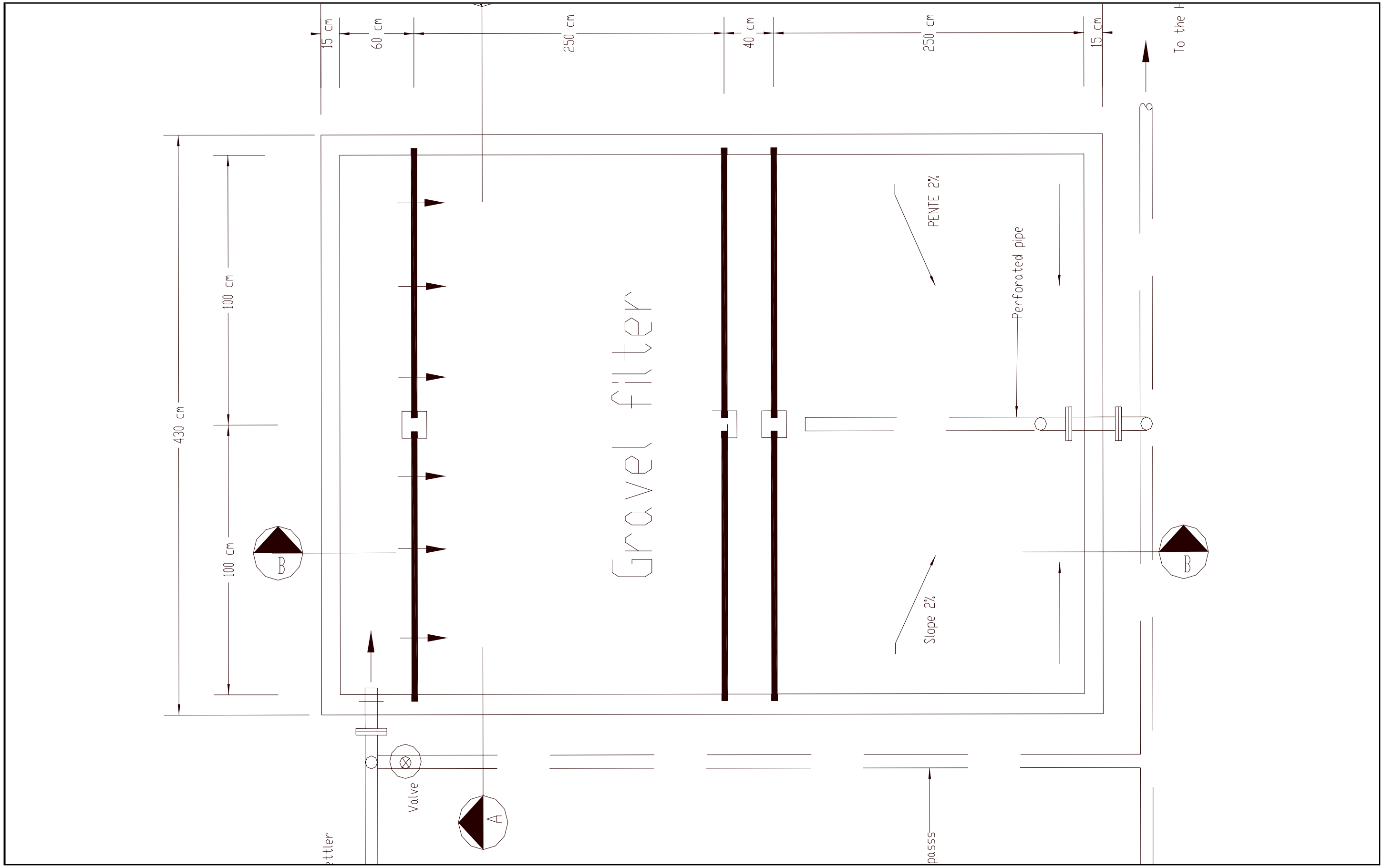

+1,20
+0,90
5T6
5T8
5T6
5T8
20
Soil= -0,10
-0,20
5T6
5T8
6T10
-1,00
5T8
5T6
-1,70
20
5T12
15
20 cm
95 cm
PROPRETE EN GRAVIER 10 cm
Cut A-A
20
170 cm
Cut B-B

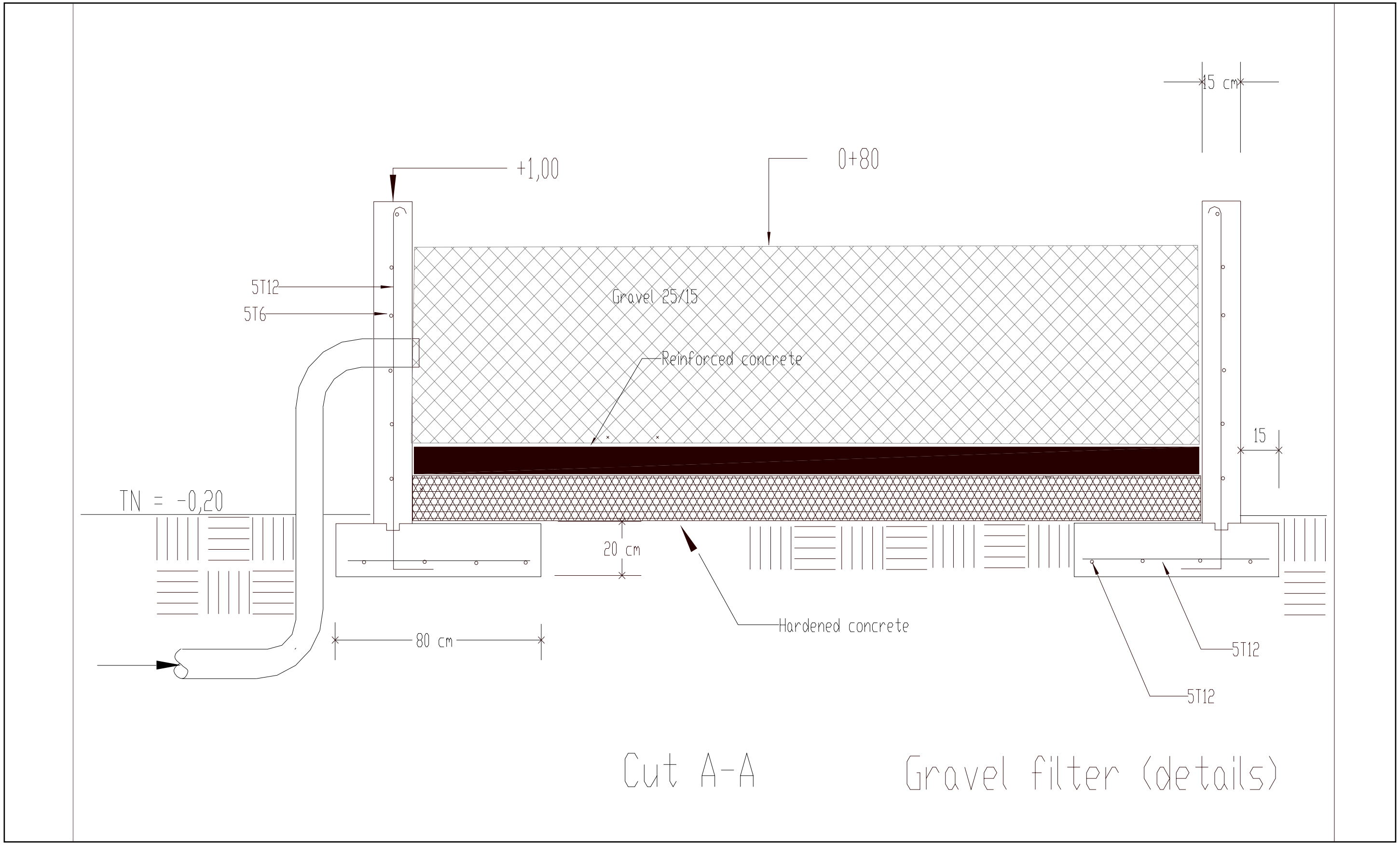


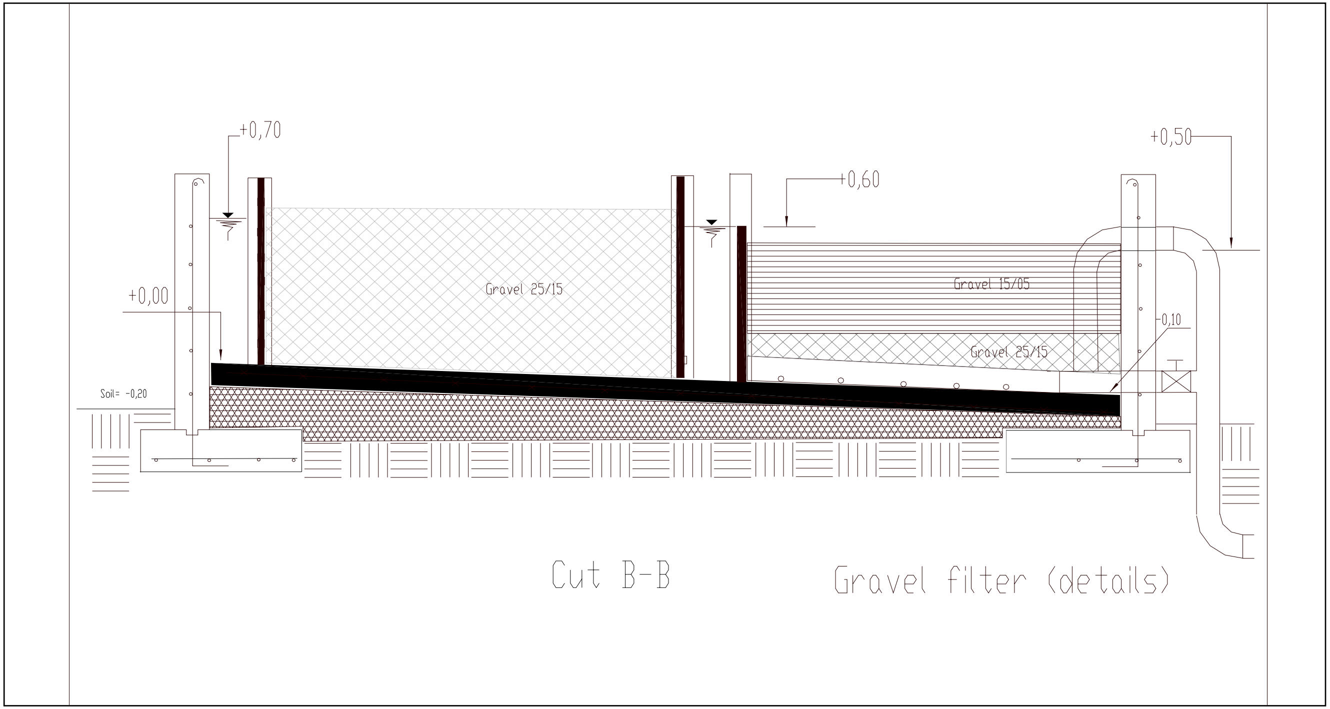
To pumps well
B
10 cm
100 cm
10
100 cm
10
100 cm
10
100 cm
10
100 cm
10 cm
10 cm
100 cm
10 cm
A
Dried sludge basin
Bed #1
Bed #2
Bed #3
A
Bed. #4
B
Reactor R1 effluent
VERS DECANTEUR
Valve
BY-PASS
Sludge drying beds
Settle

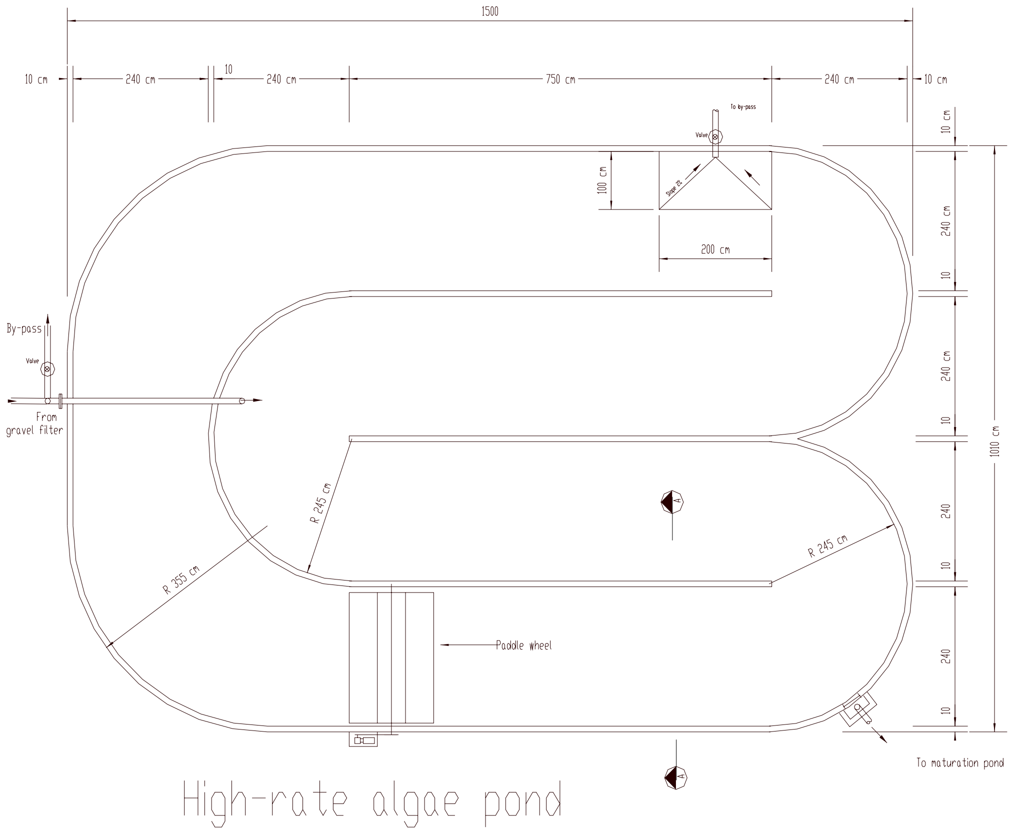

6T8
Soil = +0,05
5T6
Steel net
Reinforced concrete 40cm x 40cm x 5cm
20 cm
Gravel 15/25
30 cm
? 110 mm PVC pipe
Gravel 5/50
Perforated pipe
Slope 2% to
pumps well

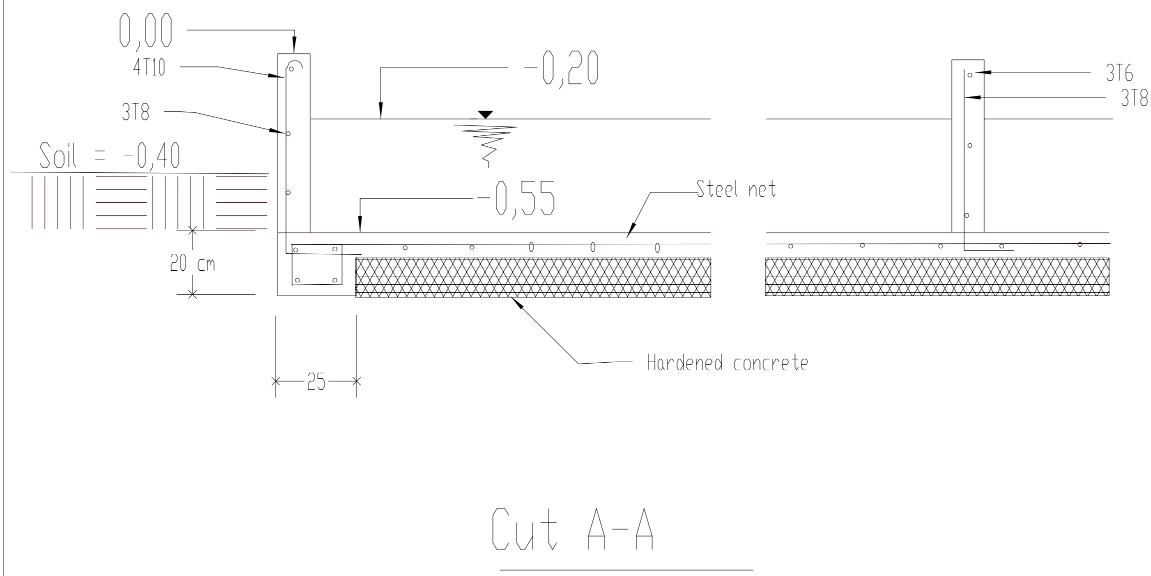
Cut A-A


15 cm
60 cm
250 cm
40 cm
250 cm
15 cm
To the
430 cm
100 cm
100 cm
Gravel filter
PENTE 2%
Perforated pipe
Slope 2%
B
B
A
Valve
passs

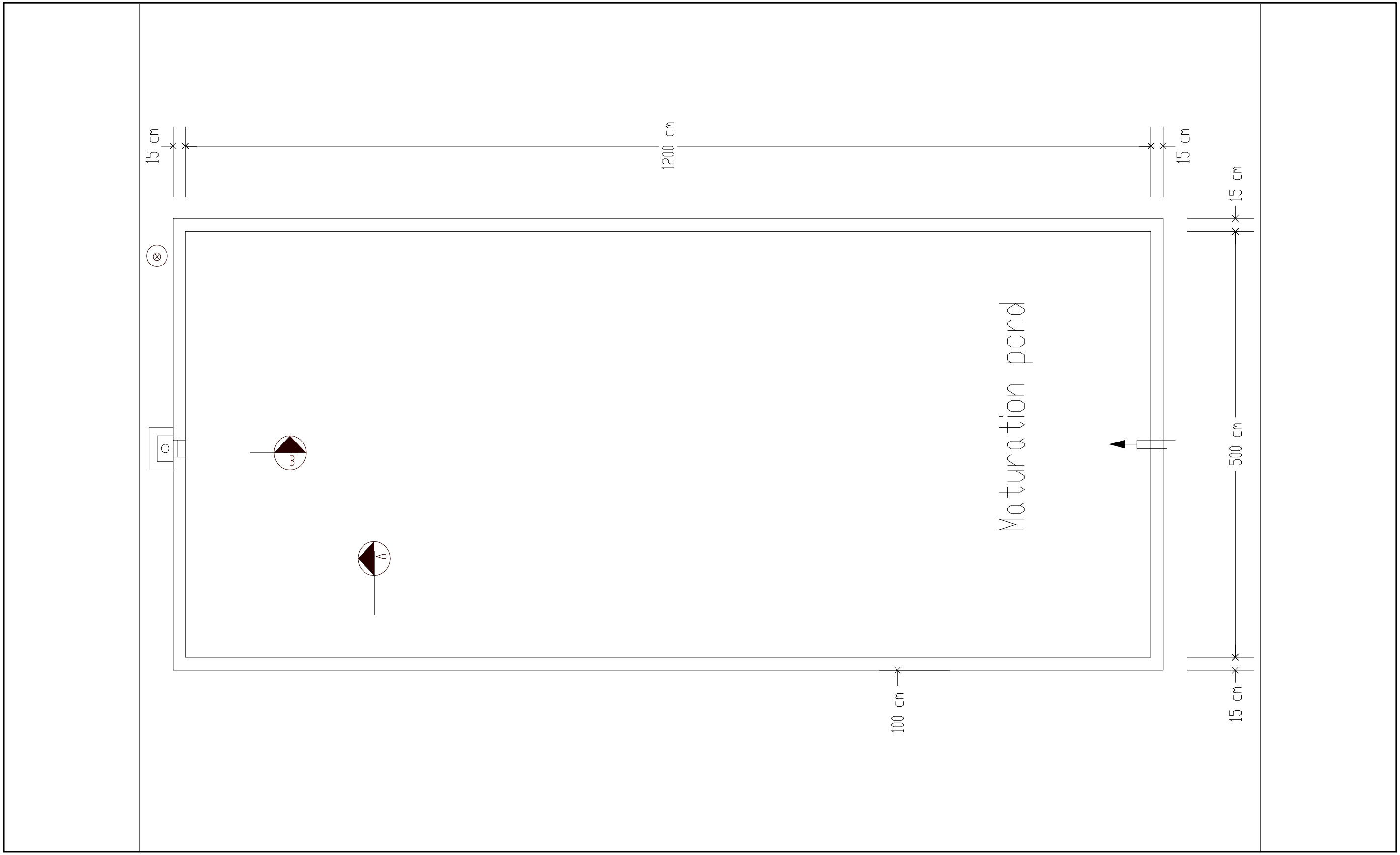

15 cm
0+80
+1,00
5T12
5T6
Gravel 25/15
Reinforced concrete
15
TN = -0,20
20 cm
80 cm
Hardened concrete
5T12
5T12
Cut A-A
Gravel filter (details)

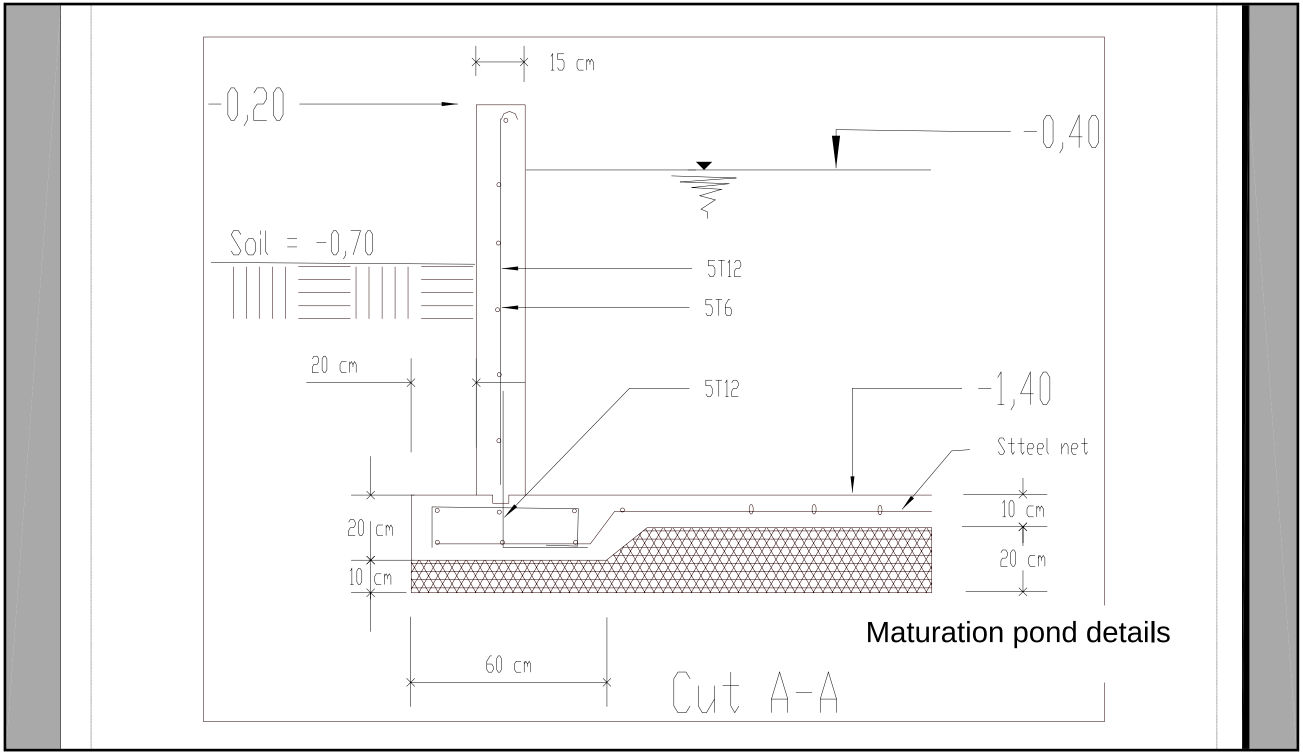

+0,70
+0,60
+0,50
+0,00
Gravel 25/15
Gravel 15/05
Gravel 25/15
0,10
Soil= -0,20
Cut B-B
Gravel filter (details)


1500
10 cm
240 cm
10
240 cm
750 cm
240 cm
10 cm
To by-pass
Valve
Slope 2%
100 cm
200 cm
By-pass
Valve
From
gravel filter
R 245 cm
R 355 cm
R 245 cm
Paddle wheel
To maturation pond
1010 cm
High-rate algae pond

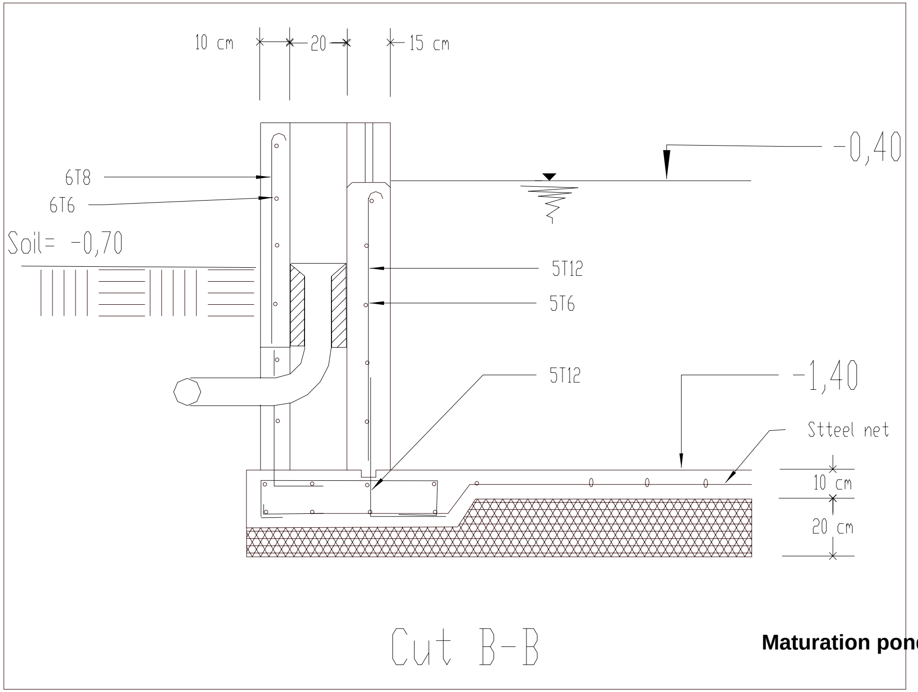


High rate algal pond (details)


15 cm
1200 cm
15 cm
15 cm
Maturation pond
B
A
500 cm
15 cm
100 cm



Maturation pond details



Maturation pond details

ANNEX 2

Operation and Maintenance procedure

Logbook for daily operation and maintenance

To be filled in by the technician in charge of the plant

Date	Name	Function	Signature
Technician			
Supervisor			

Site observations

General aspects	Odor	Birds and other	Works (walls)	Way of	Fence	Observations

Weather	Average minimum temperature		Average maximum temperature		Direction of wind	Observations
Cloudy, sunny, rainy						

Pumping Station

	Control panel	Electronic switch	Pumps	Others	Observations

Logbook for daily operation and maintenance

Pressure pipe

	Leaks	Pipe failure	Clogging	Others	Observations

Two-step up flow Anaerobic Reactor (TSUAR)

Reactor 1	Cover	Pipe & valves	Out flowing	Color and turbidity of the effluent	Sludge bed thickness	Observations
Reactor 2	Cover	Pipe & valves	Out flowing	Color and turbidity of the effluent	Sludge bed thickness	Observations
Settling tank	Cover	Pipe & valves	Out flowing	Color and turbidity of the effluent	Sludge bed thickness	Observations
Gravel Filter	General aspect	Pipe & valves	Out flowing	Color and turbidity of the effluent	Observations	

Logbook for daily operation and maintenance

Biogas unit	Gasholder	Pipe & valves	Generator (maintenance)	Biogasmeter	Flammability test	Observations
Sludge	Extraction frequency	Amount removed	Consistency	Odor emanation	Observations	
Sludge drying beds	Beds outlook	Pipe & valves	Drain function	Drying duration (day)	Amount dried/day	Observations

High-rate Algal Pond

Plant outlook	Water speed (surface)	Water color	Ciliates and rotifers	Sediment thickness	Odor emanation	Observations
Algae domination species						

Logbook for daily operation and maintenance

Paddle wheel	Outlook	Motor	Transmission unit	Speed of rotation	Observations

Maturation pond

General aspect	Water color	Flocculent algae	Ciliates and rotifers	Short-circuiting	Odor emanation	Observations
Dominating algae species						

Recommendations:

ANNEX 3

Assessment of the construction cost

Works and supplies forecasting for building a 500 person equivalent treatment plant

Item	Unit	Quantity
Digging	m^3	500
Gravel bed	m^3	24
Concrete	m^3	80
Cement rendering	m^2	390
Steel	kg	2750
Gravel for filter	m^3	24
PVC Pipes		
40 mm	m	10
50 mm	m	5
75 mm	m	20
110 mm	m	120
Valves		
40 mm	Unit	6
75 mm	U	4
110 mm	U	7
Bend 90°		
40 mm	U	4
50 mm	U	2
75 mm	U	10
110 mm	U	25
Bend 120		
75 mm	U	2
110 mm	U	2
T-shaped		
40 mm	U	4
50 mm	U	1
75 mm	U	2

Works and supplies forecasting for building a 500 person equivalent treatment plant (Continued)

Item	Unit	Quantity
110 mm	U	8
Connector		
50 mm	U	2
75 mm	U	2
Coupling		
75 mm	U	1
Anti-flow back 75 mm	U	1
Submersible pump*	U	2
Screen**	U	1
Reactor covers made of Polyester	U	1
Parshall	U	1
Paddle wheel	U	1
Electrical motor and speed reduction box	U	1

* 380/400 V; 1.5 kW maximum head 12 m

** The screen and paddle wheel can be constructed by a local iron workshop.

IAV wheel cost 300 US $.

The motor and adapted by machining shop. The cost may varied from 400 to 600 US $

ANNEX 4

Troubleshooting

Troubleshooting

Step	**Parameter to check**	**Action to be undertaken**	**Result**	**Next step**
1. Screening unit				
1.1	Removal of coarse solids		OK	Go to 2
			No	Go to 1.2
1.2		√ Increase cleaning frequency √ Check worker's seriousness	Issue resolved	Go to 2
2. Grit removal unit				**Go to 2.1**
2.1	Grits concentration		OK	Go to 3
			No	Go to 2.2
2.2		√ Increase cleaning frequency of the grit channel √ Check water speed in the channel √ Check worker's seriousness	OK	Go to 3
3. Two-Step Upflow Anaerobic Reactor (TSUAR)				**Go to 3.1**
3.1 Reactor R1				Go to 3.1.1
3.1.1	Water flow and water level in the reactor normal		OK	Go to 3.2
			No	Go to 3.1.2
3.1.2		√ Check inlet and outlet valves	OK	Go to 3.2
		√ Check R1/R2 connecting pipes	No	Go to 3.1.3
3.1.3	By-pass R1 momentarily and use R2 alone.	Proceed with the necessary intervention to solve the issue	OK	Go to 3.2

3.2 Reactor R2				**Go to 3.2.1**
3.2.1	Water flow and water level in the reactor normal		OK	Go to 3.2.4
			No	Go to 3.2.2
3.2.2		√ Check inlet and outlet valves	OK	Go to 3.2.4
		√ Check R1/R2 connecting pipes	No	Go to 3.2.3
3.2.3		By-pass R2 momentarily and use R1 alone and Proceed with the necessary interventions to solve the issue	OK	Go to 3.2.4
3.2.4	Removal rate of DBO_5		70 to 80%	Go to 3.3
	Or CODs		RR* < 60%	Go to 3.2.5
3.2.5		Check : √ Admitted flow and HRT √ pH of the effluent √ biogas production and smell √ Ascension speed < 0.7 mh^{-1} √ Check Influent COD √ Sludge concentration in the reactors. √ Check the influent for toxic substances	OK	Go to 3.2.6
			No	Go to 3.1.1
3.2.6	Offensive odors		No	Go to 3.3
			Yes	Go to 3.2.7

Troubleshooting (continued)

Step	Parameter to check	Action to be undertaken	Result	Next step
3.2.7		√ Check reactor covers √ Check water level in the channel around the reactor √ Check biogas withdrawal circuit	OK	Go to 3.3
3.3 Settling tank (ST)				**Go to 3.3.1**
3.3.1	Water flow and water level in the reactor normal		OK	Go to 3.3.3
			No	Go to 3.3.2
3.3.2		√ Check inlet and outlet valves √ Check connecting pipes ST/gravel filter (GF)	OK	Go to 3.3.3
3.3.3	TSS removal rate		> 70%	Go to 3.4
3.3.4		√ Check the regularity of removing sludge from the settler √ Check sludge drying beds are regularly receiving fresh sludge √ Increase sludge withdrawal frequency √ Check overflow rate (<1.5 mh^{-1}). √ Check influent distribution inside the ST	**OK**	**Go to.4**

* Removal rate ; calculated as follows : (CODt influent –CODs effluent)/COD t influent ; CODt COD total ; CODst COD settled

Troubleshooting (continued)

Step	Parameter to check	Action to be undertaken	Result	Next step
3.4 Sludge drying beds (SDB)				**Go to 3.4.1**
3.4.1	Water flow normal		OK	Go to 3.4.3
			No	Go to 3.4.2
3.4.2.		√ Check sludge evacuation pipe in ST	OK	Go to 3.4.3
		√ Check for air trapping in evacuation pipe (high point)	No	Go to 3.3.
		√ Check for sludge accumulation in ST		
3.4.3	Water infiltration through the bed	√ Check removal of dried sludge	OK	Go to 3.4.5
		√ Check if the first sand layer is regularly scarified	No	Go to 3.4.4
		√ Check drainage pipe and reception basin		
3.4.4.		√ Replace the sand and the gravel of the bed	OK	Go to 3.4.5.
		√ Replace the drainage pipe		
3.4.5	Low sludge production	√ Check if the reactors are in a washout period	OK	Go to 3.5
			No	Go to 3.
3.4.6	Sludge fermenting and smell emanation		OK	Go to 3.
			No	Go to 3.5.
3.5 Gravel Filter (GF)				**Go to 3.5.1**
3.5.1	Water flow from feeding pipe		OK	Go to 3.5.2
			No	Go to 3.4.
3.5.2	Surfacing of effluent		OK	Go to 3.5.3
			No	Go to 3.5.5

Troubleshooting (continued)

Step	Parameter to check	Action to be undertaken	Result	Next step
3.5.3		Check	OK	Go to 4..
		√ drainage pipe and outlet valve √ ST effluent for TSS content √ Conductivity of the bed √ ST efficiency	No	Go to 3.5.4.
3.5.4.		√ Replace the gravel of the bed Unblock or change draining pipe	OK	Go to 4.
4. High Rate Algal Pond (HRAP)				**Go to 4.1**
4.1	Water flow and water level normal		OK	Go to 4.3
			No	Go to 4.2
4.2		Check	OK	Go to 4.3
		√ For Infiltration √ Inlet and outlet pipes or channels	No	Go to 4.3
4.3		By-pass and proceed with urgent maintenance actions	OK	Go to 4.4
4.4	Water speed at the surface		0.5 to 0.2 m/s	Go to 4.6
			> 0.2 or 1.8	Go to 4.5

Troubleshooting (continued)

4.5		Check √ Rotation speed of the paddle wheel √ Power transmission system √ Maintenance of mechanical components	OK No	Go to 4.7 Go to 4.6
4.6		Replace deficient parts	OK	Go to 4.7
4.7	Type of algal cells Non motile green cells from the genus *Scenedesmus, Micractinium, Actinastrum, Chlorella, or Pediastrum*		OK	Go to 4.9
4.8	Motile cells, genus *Euglena or Chlamydomonas*		Yes	Go to 3.
4.9	Concentration of algae < 3 millions cells/ml		OK No	Go to 4.11 Go to 4.10
4.10	<0.6 or > 5 millions/ml			Go to 3.
4.11	Chlorophylle a 1 to 2 mg/l;At 2 PM : pH>8, Dissolved oxygen > 15 mg/l		Yes No	Go to 4.12 Go to 3.
4.12	Removal rate of DCOs or de DBO_5 15 to 20%		Yes No	Go to 4.13 Go to 3.

Troubleshooting (continued)

4.13	Removal rate of-ammonia (N-NH_4^+) and orthophosphate (P-PO_4^{3-}) 80% for N ; 50% for P		No	Go to 3.
			Yes	Go to 4.14
4.14	Fecal coliforms 10^4 to 10^5 FC/100 ml		OK	Go to 4.15
			No >10^6	Go to 3.
4.15	Helminth eggs : Zero egg/l		OK	Go to 5
			> 1egg/l	Go to 3.
5. Maturation pond				**Go to 5.1**
5.1	Water flow and water level normal		OK	Go to 5.3
			No	Go to 5.2
5.2		Check √ For Infiltration √ Inlet and outlet pipes or channels	OK	Go to 5.4
			No	Go to 5.3
5.3		By-pass and proceed with urgent maintenance actions		Go to 5.4
5.4	Non motile green cells of the genus *Scenedesmus, Micractinium, Chlorella, Actinastrum, or Pediastrum*		Yes	Go to 5.6
			No	Go to 5.5

Troubleshooting (continued)

Step	Parameter to check	Action to be undertaken	Result	Next step
5.5	Motile cells From genus *Euglena or Chlamydomonas*		Yes	Go to 4.
5.6	Concentration of algae 1 millions cells/ml		OK	Go to 5.8
			No	Go to 5.7
5.7	<0.3 or >2.5 millions/ml			Go to 4.
5.8	Chlorophylle a 0.5 to 0.8 mg/l At 2 PM : pH >8 Dissolved oxygen> 15		OK	Go to 5.9
			No	Go to 4.
5.9	Removal rate of DCOs or de DBO_5 10 %		OK	Go to 5.10
			No	Go to 4.
5.10	Removal rate of-ammonia ($N\text{-}NH_4^+$) and orthophosphate ($P\text{-}PO_4^{3-}$) 10% for N and 5% for P		OK	Go to 5.11
			No	Go to 4.
5.11	Fecal coliforms 10^3 to 10^4 FC/100 ml		OK	
			No	Go to 4.

ANNEX 5

ARHPCT treated effluent reuse

Summary of a case study

Reuse for flower production

Contributed to this work
Prof. Driss Alami, department of Horticulture, IAVH.
Eng. Abderrahim Handoufe, Administration du Génie Rural, Ministry of Agriculture, Morocco

Introduction

When the climate allows it, flower production represents an attractive alternative for valorizing treated wastewater in agriculture. Flower crop can not stand the high temperatures nor the low winter temperatures necessitating the use of greenhouses. The choice of flower crops for valorizing treated wastewater helps solving the dilemma coming from the fact that high value crops, which are generally eaten raw, represent, in the same time, a great health hazard to the consumers. Since the flowers are not consumed, the problem of valorizing treated wastewater in irrigation become simpler. Protecting the field workers and optimizing the use of treated wastewater and the nutrients it conveys become the main objectives.

The case study related here deals with a flower production experiment done at the IAVH campus in Rabat using the ARHPCT effluent and conventional water (control) for irrigation. The experimenters adopted a localized Irrigation system and did not supply fertilizers during the period of production to determine the fertilizing potential of the effluent by comparison with the control (groundwater and no fertilizers).

The work also addressed the issue of determining whether the relatively high concentrations of algae cells found in ponds effluents constitute an obstacle to adopt localized irrigation for effluent reuse.

Carnation flower

Four-week-disease-free-rooted cuttings were purchased from a flower nursery specialized in carnation production. Density of plantation was 32 plants/m^2. Cuttings were planted in Mai, 1999 and flower harvest started in September of the same year. The experiment ended in July 2000.

Greenhouse facility

A 240 m^2.-tunnel shaped greenhouse was used (30 x 8 m). The tunnel was oriented Northwest-Southeast with four openings on the length sides. During the summer time, the polyethylene sheet was whitewashed to reduce light (and heat) penetration.

Irrigation system

Final effluent of the ARHPCT was pumped to the greenhouse. Pumping unit was equipped with a backwash sand filter to stop algae cells followed by a screen to prevent escaping sand particles from reaching the distribution network and clogging the emitters. The irrigation system consisted of a drip irrigation type “Arab drip” with emitters delivering 2 l/h at 1 bar pressure. The distance between emitters was set at 0.25 m.

Emitter clogging

The rate of emitter clogging was 17% for ARHPCT effluent and 4% for conventional water. This rate was similar to those already reported for waste stabilization ponds effluent reuse in Ouarzazate (El Hamouri *et al.*, 1996). The impact of such a clogging rate could be minimized with an intensive surveillance and maintenance program that are affordable in the MENA region where the cost of labor is low.

Other factors found to have an impact on the clogging rate were the frequency of water applications and the pressure. Lower pressures increased the rate of clogging while higher pressures reduced it.

Diameter of the flowers stem

The stem diameter has a particular importance for flower marketing. The flower must stand alone on the vase and keep a vertical position. Leaning flowers have lower value on the market. The mean stem diameter of the flowers irrigated with ARHPCT effluent (11mm) was almost two times higher than that of to control (conventional water).

Production yield

The yield recorded after one year of production was 7 and 5 flowers/plant/year respectively for ARHPCT effluent and the control. But, flowers from the control were of low market value.The yield obtained was even higher than the average production yield of specialized production units in Morocco, which is 5.75 flowers/plant/year. (Maach, 2001).

Life duration in flower vase

The life duration in flower vase is an important factor for marketing. The longer the life duration the higher is the price. Flower produced on ARHPCT effluent showed a longer period than the control. The difference was 8 days. The flower irrigated with ARHPCT effluent could stay 20 days in the vase while the controls could not exceed 12 days.

Conclusion

Localized irrigation systems could be successful with ARHPCT effluent provided simple measures are taken. Placing a backwash-sand filter and a screen behind the pumping unit did succeed avoiding major clogging problems. Together with an intense surveillance and maintenance program, this approach helped overcoming emitters clogging problems and brought it to acceptable levels for areas where the cost of labor is low.

Carnation production yield using the ARHPCT effluent for irrigation was satisfactory keeping in mind the fact that this yield was obtained without any fertilizers supply. Also, flower vase life tests showed that flowers irrigated with ARHPCT effluent have higher stem diameter and stand longer in the vase compared to the control.

Fertilizers were not supplied during the time of experimentation. Soluble nitrogen, phosphorus and trace elements contained in the effluent did cover the crop needs in these elements as demonstrated by the obtained yield.

Adequate measures must be taken to protect technician and workers in charge of the production even thought the contact with irrigating water was minimal.

References

El Hamouri B., Handoufe A., Mekrane M. Touzani M., Khana A. Khallayoune K. & Benchokroun T. (1996). Use of wastewater for crop production under arid and saline conditions: yield and hygienic quality of the crop and soil contamination's. Water Science & Technology 33(10-11) 327-334.

Maach Yassine (2001). Valorisation des eaux usées épurées en floriculture : cas de l'œillet. Mémoire de fin d'études. Institut Agronomique et Vétérinaire Hassan II, Rabat

ANNEX 6

Photo album

IAV treatment plant

Maturation and high rate algal pond

Anaerobic reactors and maturation ponds

ARHPCT dried sludge

Sludge drying beds

Biogas storage and generator

Biogas storing unit

Diesel-biogas combined generator

Outlet structure

Outlet weir for the high Rate algal pond

Maturation pond effluent

Raw wastewater (right), TSUAR second, HRAP third and final effluent (left)

Paddle wheel to homogenize the high rate pond

Paddle wheel details of material and construction

Paddle wheel driving mechanism (electric motor and gear box control)

Greenhouse whitewashed in summer time

Carnation production unit

Flowers ready for delivery

***In-situ* effluent reuse for landscaping**

Reactors, Settler and sludge drying beds

Reactors and lift station

Connection pipes and valves

Two-unit -horizontal-vertical gravel filter