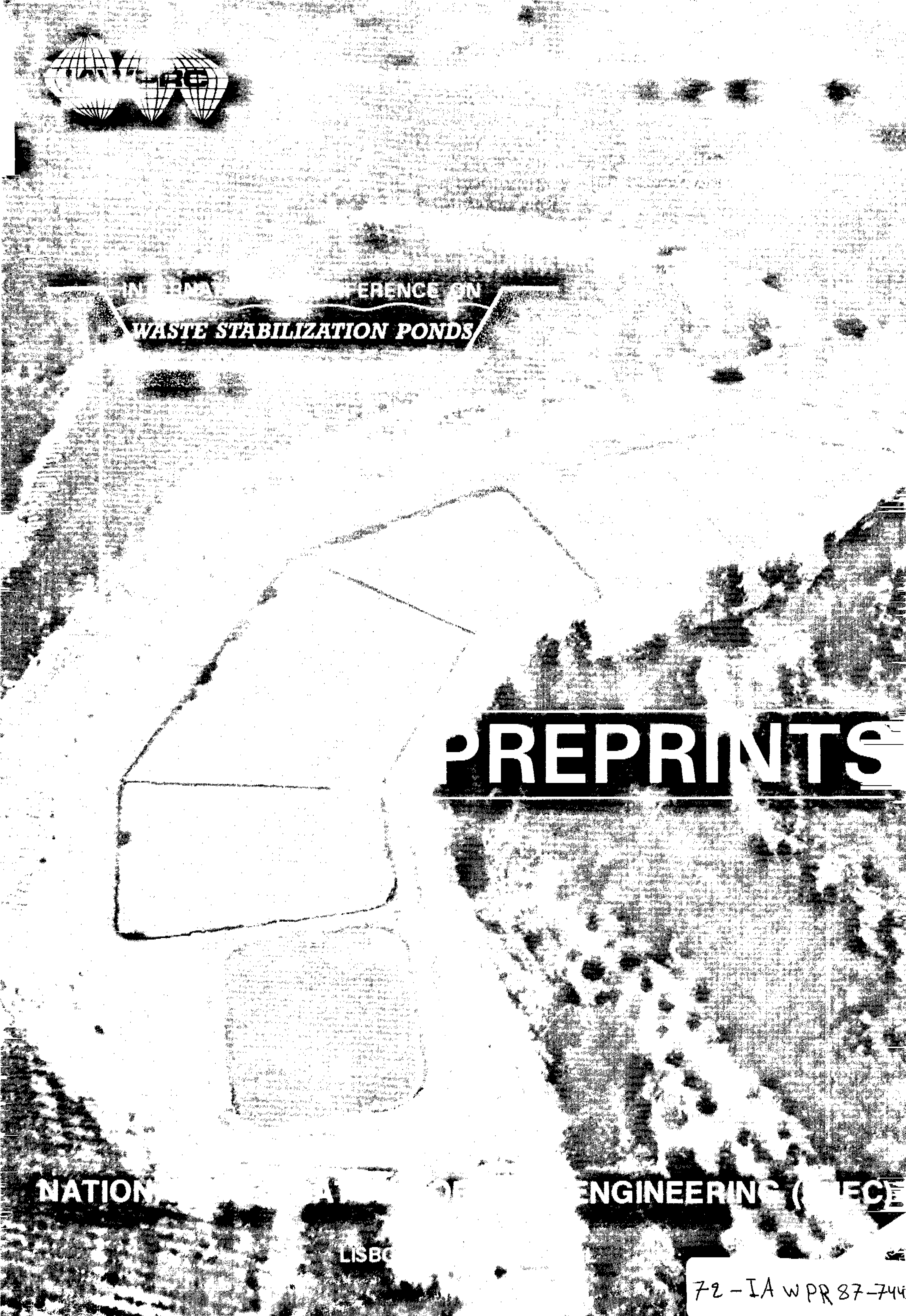

IN RNA FERENCE ON

WASTE STABILIZATION PONDS

PREPRINTS

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PREPRINTS

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THEME 1

POND EXPERIENCE

WASTE STABILIZATION PONDS IN EUROPE :
A STATE OF THE ART REVIEW

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ABSTRACT

Waste stabilization ponds can be found in 16 countries in Europe. Information given at a consultation of WHO specialists to review the state of the art in lagooning, highlighted the varying development stages which have been achieved in this field in 10 different countries. Design criteria vary from one country to another, to suit local conditions and conform with the quality standards which effluents are expected to meet. Lagooning has found many different fields of application in Europe : small communities, tourist areas, tertiary treatment, where it is likely to be further developed.

KEYWORDS

Waste stabilization ponds, lagooning, wastewater treatment, Europe, review report.

INTRODUCTION

In many countries, waste stabilization ponds -WSP- are one of the techniques available for the treatment of wastewaters before they are discharged into the environment. However, the design and construction, as well as the fields of application of this process vary from one country to another, and even within the same country, depending on local conditions. A first comparison of the various lagooning techniques used in Europe was made at a specialists' consultation organized jointly by WHO Regional Office for Europe and CEMAGREF in Lyon (France) from the 20th to 23rd october 1986. 32 experts, representing the technical experience of 10 European countries and 3 countries outside Europe, attended the meeting.

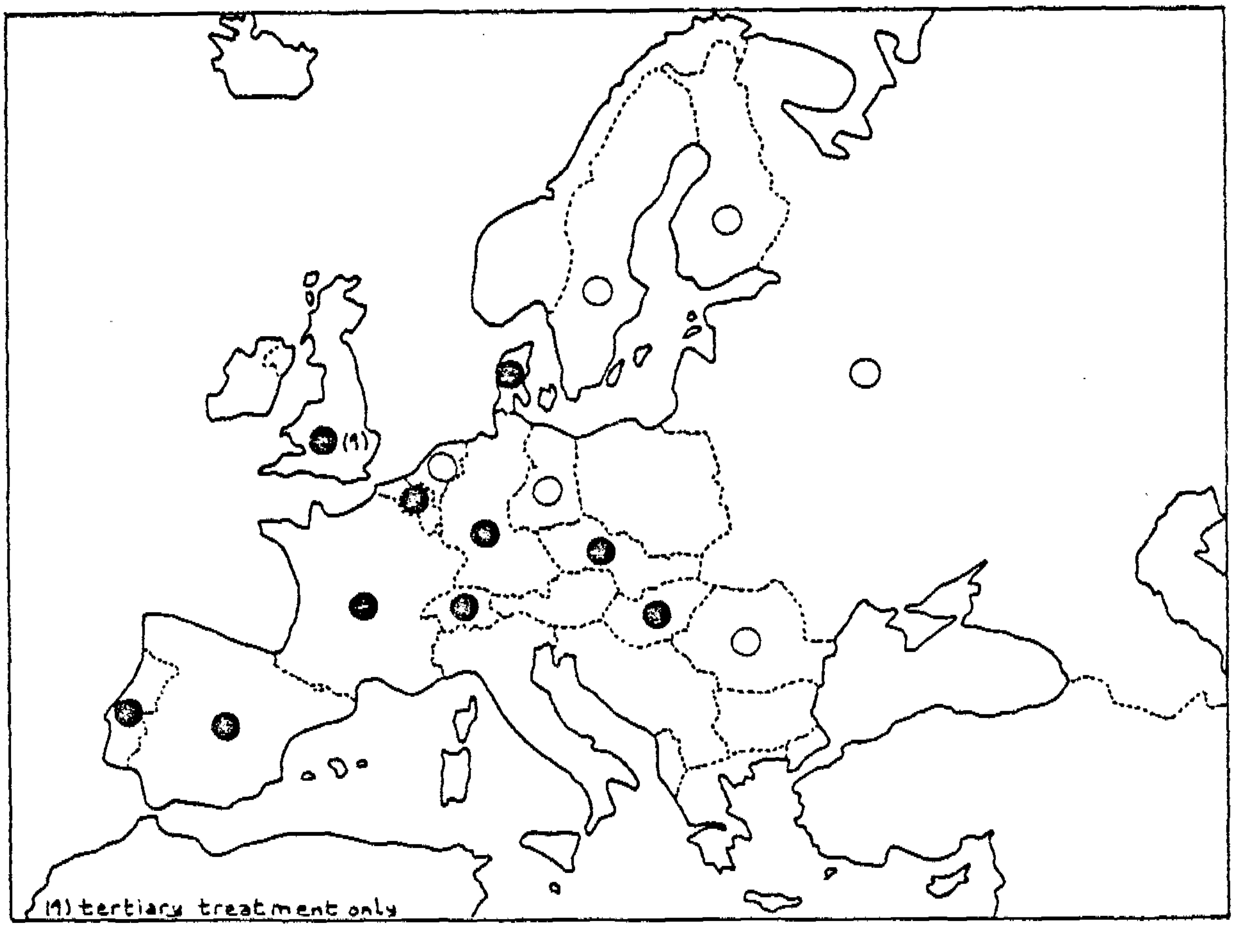
This paper is largely based on information received during the consultation preliminaries and from working documents prepared by the invited participants. It also reviews some of the findings of the working groups. This report exclusively refers to European countries, and to non aerated ponds:mechanically aerated lagoons although they are sometimes included in WSP statistics for individual countries, are not considered, as they differ, in our opinion, as far as both their functioning and their applications are concerned.

THE DEVELOPMENT OF LAGOONING IN EUROPE

The use of ponds to accumulate and treat wastes with a high content of organic matters of animal and domectic origin is an ancient method. In Europe, one could say that farm ponds, which are well known in certain regions, or fishponds enriched with organic wastes, which have been developed in Central Europe since the Middle Ages, are the earliest examples of the use of the self-purification process which takes place in stagnant ponds and of their ability to transform and recycle organic matters. The oldest WSP in Europe, still operating today, are probably the "Fischteiche" built around 1930 as a tertiary wastewater treatment for the City of München (Bavaria, FRG). The plant covers a total area of 233ha (576 acres) divided into ponds of 7ha (17.3 acres). It receives part of the effluents of the biological treatment plant of the city, (approximately 50 %, i.e. $3m^3/S$, diluted in river water) in addition to stormwater after primary settlement. Carps breed and are regularly harvested in those ponds (CEMAGREF 1986). However, this is a totally unique case. As from the 1940's, WSP were developed in Denmark for the secondary treatment of domestic or industrial wastewaters, according to specific purification criteria (WHO, 1986a).

A first WHO survey conducted in 1964 and mentioned by Gloyna (1972), showed that WSP were used in 39 countries, including 7 European countries : Findland, FRG, GDR, the Netherlands, Rumania, Sweden, USSR.

Twenty two years later, in 1986, WSP are found in at least 16 European countries : the map below gives a summary of their distribution, according to the information available.



●	Existing operational WSP. Technical documentation available
○	Operational WSP reported. No sufficient technical documentation
	No operational WSP reported. No information

Fig. 1 : Map showing WSP distribution in Europe

More or less complete technical data is available for 10 out of the 16 countries where WSP are used. There are however a dozen countries on which no reliable information is available as to the existence or not of operationel WSP. If one refers to the 10 countries from which information is available, it appears that WSP have reached varying development stages from one country to another. One can distinguish between the following cases :

- Countries where the technique is largely developed and commonly implemented by engineers, as in the case of FRG (more than 2000 operational plants) and France (almost 1500 plants)
- Countries which have begun to collect local data on WSP from operating pilot plants and which favourably consider the use of this process. These include Portugal (more than 30 operational WSP), Spain (more than 10 operational WSP) and to a certain extent, Belgium (variable situation, depending on the regions) and Switzerland (fairly limited applications).
- Countries where WSP have been used, but where future development is unlikely, for various reasons (geographical situation, regulations,...) as in Denmark (66 WSP presently operated, generally old ones progressively being replaced by conventional wastewater treatment plants), in Hungary (unfavourable geological conditions), and the United Kingdom (were WSP seem to be considered only for the tertiary treatment of conventional wastewater treatment plant effluents).

A SURVEY OF TECHNICAL PRACTICE

Table 1 below gives a summary of some of the technical data on WSP in 8 European countries from which detailed information was available. This data is further developed in the following paragraphs.

Plant design

Two treatment lines are to be considered from a design point of view, depending on whether the plant includes a primary aneorobic pond, or settlement pond as it is called in FRG. Anaerobic ponds, which are used in many countries, have so far practically never been used in Belgium, France, Hungary because of the potential odour nuisance.

Aneorobic ponds are usually designed according to the retention time of effluents: 5 to 6 days in Denmark, 1 to 2 days in FRG (taking into account the sludge volume which may reach 50 % of the total volume before desludging). Experiments Conducted at FRIELAS (Portugal) have shown a 44 % reduction in BOD_5, for a theoretical retention period of 1.6 days (i.e. : $VA = 0.2\ m^3$/inhab. eq.)(DO NASCIMENTO et al, 1985).

In all countries, anaerobic ponds are followed by one or several facultative or maturation ponds. The total surface area of these ponds varies, depending on local design practice, from 5 to 10 m^2/inhab. eq. over 1, 2 or 3 ponds. Work carried out in Bavaria (WHO 1986,c) shows that up to 10 m^2 (covering 2 ponds) the increase in specific surface area coincides with an improvement in the discharge quality (BOD5, COD), whereas beyond that surface, no further improvement is detected (except for NH_4-N). The Frielas experiment (A_1-F_1-M_1 treatment line) shows a 94 % reduction on filtered BOD_5, with a size corresponding to a specific surface area (F + M) of 4.5 m^2/inhab. eq (according to DO NASCIMENTO et al, 1985).

The treatment lines without anaerobic ponds ("facultative treatment lines") can be found in all the countries refered to except Denmark. The total specific surface area of the plants varies from one country to another, from 5 to 18 m^2/inhab. eq.. Some plants may consist of only one pond but as a rule, there are 2 or 3 ponds in series. The primary facultative pond then covers between 30 % (FRG, Switzerland) and 50 % (Belgium, France) of the total surface area of the plant. Available data in France shows that the size generally applied (10 m^2/inhab. eq., surface ratio 2/1/1) provides a treatment quality which conforms with French standards. These are : filtered $BOD_5 < 40$mg/l; TSS $<$ 120 mg/l. The quality of the discharge is improved when the plant is under-loaded, which is often the case (P. BOUTIN, 1987). On the other hand, it can be temporarily inferior, particularly in Summer in the Southern part of the country, due to a high concentration of algae. The Frielas experiments on a similar treatment line (8 m^2/inhab. eq. surface ratio 2/1/1) shows a 95 % reduction in filtered BOD_5, with very little variation over time (standard deviation less than 2).

TABLE 1 : Review of WSP current practice in 8 european countries

Country (ref)	Nb of WSP (1)	Size (inhab. eq.) (2)	Type of WSP (3)	Type of influent (4)	Remark - design criteria (5)
BELGIUM (WHO, 1986b)	1 - [4]	150 7700	F	D	Data refer to Walloon region SF = 5 to 11
DANEMARK (WHO, 1986a)	66	50 [500-1000] 3000	A	D, I	VA 0.9+SF = 8 to 10
FRG (WHO, 1986c)	2000	100 to 1000 and +	F - - - - - - - A	D, I	northern Germany SF = 10 to 15 (3 ponds-surf ratio= 3/4/3) - - - - - - - Southern Germany VA=0.5+SF= 5 to 10
FRANCE (P. BOUTIN et al, 1987)	1500	100 [600] 6000	F (95%) - - - - - - - TT (5%)	D, DS few I	SF=10 (2 or 3 ponds surf ratio 1/1 or 2/1/1) - - - - - - - SF = 5
PORTUGAL (SANTOS OLIVEIRA et) all, 1986)	30 (20 for domestic ww)	300 [4300] 15000	A - - - - - - - A (8 plants) F (10 plants) TT (1 plant)	I - - - - - - - D, DS	various design See reports on experimentations in Frielas 1 high rate algal pond
SPAIN (WHO, 1986 g and h)	10 [15]	400 12000	A (5 plants) F (5 plants)	D, DS	various design Existing experimentations in Ranilla (Sevilla)
SWITZERLAND (WHO, 1986 d)	6	160 [340] 700	A (1 plant) - - - - - - - F (4 plants) TT (1 plant)	D	VA=6.7 (includes storm water - - - - - - - SF=6 to 18 3 plants with primary settlement
CZECHOSLOVAKIA (STASTNY et al, 1984)	?	?	F - - - - - - - TT	?	SF = 7 - - - - - - - SF=2.5 (assume influent BOD=40mg/l)

(1) In operation and [under construction or projected]
(2) mini - [average] - maxi
(3) A = Anaerobic lines (at least 1 anaerobic pond)
 F = Facultative line (no anaerobic ponds)
 TT= Tertiary treatment
(4) D = Domestic
 I = Industrial or agricultural
 DS= Domestic with seasonal variations in population
(5) VA= Volume of anaerobic ponds in m^3/inhab. eq.
 SF= Surface of facultative ponds, in m^2/inhab. eq.

Fields of application

Presently, WSP are mainly used in Europe for the treatment of small communities domestic wastewaters (< 2000 inhabitants). This applies to most of the 3500 plants operated in FRG and France.

Most large size plants (> 10000 inhabitants) were built for the treatment of wastewaters in communities presenting a large increase in population during the Summer season (Tourist areas). This is the case of the largest ponds operated in Portugal, Spain and France. Provided a number of measures are observed in their design and operation (WHO 1986,e), this process has proved to be both worthwhile and reliable for this particular application.

"Anaerobic treatment lines" are used in certain countries for the treatment of mainly agricultural of industrial wastewaters (Portugal, FRG). These treatment lines seem to be well-suited to this type of wasterwater. On the other hand "facultative treatment lines" have been less extensively used in this field. According to French experience, they seem to be less suitable for the treatment of concentrated or highly fermentable agro-industrial watewaters.

In certain countries, WSP are also used for tertiary treatment (5 % of Plants in France, positive results in the United Kingdom (WHO 1986, f), one plant recorded in Portugal and in Switzeland). This process is applied when a microbial desinfection of the wastewater or a reduction in nutrient content is required.

Treatment processes derived from WSP

Judging from the consulted documents, it appears that several countries have developed new wastewater treatment concepts based on lagooning. The main ones are listed below.

- Lagoons planted with rooted macrophytes. The use of maturation ponds partly planted with rooted macrophytes has mainly been developed in France, particularly in the North, (more than 70 plants in operation). This concept allows a reduction in the pond depth (0.4 m instead of 1.2 m) whilst maintaining as reliable a purification process as in the case of algal maturation ponds with the same surface area and even improving the SS concentration in the final effluent.
- Use of floating macrophytes (water jacinth, duckweeds). This type of lagoon seems to have been envisaged in South European Countries. Experiments have been conducted, particularly in Italy, Portugal and France. The practical advantage of floating macrophytes has not yet been totally ascertained. In France duckweeds grow naturally on a large number of wastewater lagoons. Their proliferation results in substantial maintenance constraints.
- High Rate Algal Ponds (HRAP). These are shallow ponds combined with a harvesting system for the algae produced. HRAP are tested in FRG and Portugal. The field of application for these processes seems relatively limited, due to the rather complex harvesting technique and difficulties in finding outlets for the algal biomass.
- Ponds combined with a Trickling filter or rotating biological contactors. These processes have been particularly developed in FRG and France. They have produced good results. The use of ponds to replace primary and secondary settlement tanks makes the classical design simpler. These processes constitute a valuable alternative when the WSP cannot be constructed, due to a shortage of space. Their surface area covers 1.5 to 3m²/inhab. eq. (WHO 1986,c).

AGRICULTURAL REUSE OF POND EFFLUENTS

The agricultural reuse of treated watewaters has been examined or considered in several European countries (FRG, France, Portugal, Spain). From a bacteriological viewpoint, the available data confirm that normally designed WSP provide an average reduction of at least 3 log units in faecal coliforms, whilst additional maturation ponds further increase this result.

The conclusions of the "Engelberg report" (IRCWD, 1985) follow the same line : "...WSP are well able to produce an effluent which meets the recommended microbiological quality guidelines for unrestricted irrigation...(i.e : intestinal nematodes < 1 viable egg/l and faecal coliform < 1000/100ml ; both geometric means)... at low cost and with minimal operational and maintenance requirements".

The main limiting factors for the use of WSP are the substantial ground space taken up by these plants and, as a direct result, the large quantity of water lost through evaporation in hot climatic conditions.

CONCLUSIONS

This being a first analysis of WSP in Europe, and considering the lack of available data from several countries, no final conclusion can be drawn at this stage.

WSP have undergone a substantial expansion in Europe : the number of countries concerned has more than doubled over the last 20 years. This process was rapidly developed in countries where it is now fully operative. There are still very important potential WSP applications in Europe : they are applicable to all countries where there is a problem of wastewater treatment in small communities, or at least to those countries where favourable geological and pedological conditions make the construction of WSP feasable at a reasonable cost. WSP can also be recommended in areas with a high summer tourist population, particularly in mediterranean Europe. The plants operating in Portugal, France and Spain could lead the way for other European countries. Finally, the use of WSP as tertiary treatment for pathogens and nutrients removal is also of interest for industrialized countries, already well equiped in wastewater purification plants.

The design criteria vary considerably from one country to another and it is not possible to indicate any "average value". It appears that the design standards need to be adapted each time to the local conditions prevailing in the country or region concerned. Even while taking advantage of other countries' previous experience, it appears that in almost every country, WSP have had to go through an experimental stage (pilot plants, or analysis of first operational plants) so as to determine locally, not only the design criteria, but also the construction and operation conditions. In this respect, an analysis of the French experience shows that failures are more generally due to defective physical design rather than a lack of precision in the process design criteria.

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SEWAGE TREATMENT IN PONDS - GERMAN EXPERIENCES

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ABSTRACT

In Germany there are both highly industrialized regions with large towns as well as extended rural areas with small communities. Most of these communities also need regular sewage disposal systems. Sewage treatment plants have to be well adapted to the special situation in the rural villages. Ponds fulfill the requirements in an excellent way. Many hundred of ponds are in operation; three kinds of pond systems are in use:

- facultative ponds (more than 1000 plants) for serving less than about 1000 i+ie;
- artificially aerated ponds (more than 300 plants) for treating sewage of about 1000 to 10000 i+ie especially when also food industries are connected or as an individual solution for those industries;
- ponds combined with trickling filters or rotating bio-filters (more than 100 plants) as a solution for treating preferably domestic sewage of about 1000 to 4000 i+ie.

Lay out, design figures, recommendations for construction and operation, effluent results and costs are presented on the basis of experiences with numerous plants. Actual effluent results are considerably below German Effluent Quality Standards. Facultative ponds with 10 m^2/ie specific surface area meet the following figures: COD ≤ 90 mg/l, BOD_5 ≤ 25 mg/l, NH_4-N ≤ 15 mg/l, PO_4-P ≤ 6 mg/l. Algae in pond effluents increase the residual organic load: 100 µg Chlorophyll-α represent around 15 mg COD resp. 5 mg BOD_5 in the average.

KEYWORDS

Rural villages; pond systems; advantages; disadvantages; lay out; design figures; construction; operation; effluent results; costs.

INTRODUCTION

In Germany there are both highly industrialized regions with large towns as well as extended rural areas with small communities. Most of these communities built central water supply systems after 1950. Villages are growing, since people like to live in rural areas permanently or seasonally, nowadays. In many villages industries based on agricultural products are existing, especially dairies, breweries, slaughter houses or canneries. Therefore, controlled sewage disposal is very necessary.

Because of large distances between rural villages it is too expensive in most cases to connect them by long sewer lines to one large sewage treatment plant. Individual small treatment units are necessary quite often located at receiving streams with very poor flow. These treatment plants have to be well adapted to the special situation in rural areas.

Ponds fulfill this requirement (Bucksteeg 1982, 1983); their advantages are:

- high treatment efficiency with great process stability
- extremely high equalization capacity to hydraulic and/or organic shock loads
- no problems with treatment of stormwater when combined sewer systems are applied
- simple plant construction
- no or only little machinery
- easy plant operation and little maintenance
- low construction and operation costs.

On the other hand the disadvantages of facultative ponds also should be mentioned:

- great land consumption
- odours may occur sometimes
- heavy algae growth may appear
- large surface area may cause high water losses in hot seasons and climates because of evaporation.

The advantages overbalance the disadvantages. No technical sewage treatment plant would combine so many advantages mentioned above as ponds do. Ponds should not be considered as old fashioned; they are a very good solution for treating sewage of small, especially rural communities (and for storing it for agricultural reuse; Bucksteeg 1982, 1983).

FACULTATIVE PONDS

Two different facultative pond systems are in use in Germany (Bucksteeg 1982):

- Pond system "A" (applied in the northern part of Germany)	
specific surface area totally required	$10 - 15\ m^2/i$
total area distributed among 3 ponds in series with a ratio of area	about 3 : 4 : 3
depth of ponds	about 1.2 m
- Pond system "B" (applied in the southern part of Germany)	
sedimentation pond as the first unit, specific volume required	$0.5 - 1.0\ m^3/i$
depth	1.5 m
two ponds as the second and third unit, specific surface area totally required	$(5 -)\ 10\ m^2/i$
depth	about 1.0 m

Fig. 1a. Facultative pond system "A"

Fig. 1b. Facultative pond system "B"

The following discussions refer to pond system "B". Figures 3 and 4 show effluent results for COD and BOD_5 depending on specific pond surface area (Schleypen 1982, Bucksteeg and Schleypen 1983, Schleypen and Wolf 1983). The curves in the diagrams represent the 90 %-probability. Similar results are known from ponds of system "A". (Neumann 1983, Voß 1985, Kayser and Fröse 1986) COD- and BOD_5-effluent results of summer resp. winter performance do not differ very much; fig. 2 shows a typical monthly temperature distribution of sewage from rural villages in Germany.

CONCLUSIONS

- Below a specific pond area of $1.5\ m^2/i$ the COD- and BOD_5-effluent results differ considerably; in that case sedimentation as well as facultative processes of biodegradation go on. Between 1.5 and $5.0\ m^2/i$ a high reduction of the organic load occurs. From 5 to $10\ m^2/i$ there is still little improvement of effluent values. Providing more than $10\ m^2/i$ COD- and BOD_5-effluent values do no more decrease.

Fig. 2. Typical monthly temperature distribution of pond influents in rural villages in Germany

Fig. 3. COD-effluent results (averages from non-filtrated samples) of facultative ponds depending on specific pond area (Bucksteeg and Schleypen 1983)

Fig. 4. BOD_5-effluent results (averages from non-filtrated samples) of facultative ponds depending on specific pond area (Bucksteeg and Schleypen 1983)

Fig. 5. Increase of COD in pond effluents caused by algae (Schleypen 1985, 1986)

Fig. 6. Increase of BOD_5 in pond effluents caused by algae (Schleypen 1985, 1986)

- From fig. 3 and 4 it can be seen that even the results of non-filtrated samples fulfill German Effluent Quality Standards with a great margin of safety, when facultative pond systems are designed and operated with 10 m^2/i.
  Fig. 5 and 6 show the portion of COD and BOD_5 in pond effluents caused by algae which are removed by filtration; 100 µg Chlorophyll-α represent around 15 mg COD resp. 5 mg BOD_5 in the average.

- In figure 3 effluent results of shock loads of liquid manure and silage wastes are indicated. Influent figures for COD of 2000 mg/l are reduced to less than 200 mg/l in facultative ponds providing more than 10 m^2/i surface area. In cases where manure or silage wastes may enter occasionally the sewer pipes, ponds should be designed for 10 m^2/i. (This figure is suggested for facultative ponds in moderate climates by many authors in the international literature.)

- A special evaluation of pond effluent data for storm flow under combined sewerage conditions shows that hydraulic as well as organic shock loads are equalized very efficiently by the large area and volume of facultative pond systems. Run off from roads of rural villages may have COD-concentrations of more than 1000 mg/l and BOD_5-concentrations of more than 500 mg/l short time after beginning of rainstorms. It was found that pond systems with a specific surface area $> 5\ m^2/i$ and with some storage capacity and with suitable inlet and outlet constructions can be loaded with 40xDWF (dry weather flow) during rainy periods. Under these conditions COD- and BOD_5-effluent results are of the same order as they are at dry weather. (Schleypen 1982) Pond Systems "A" can be loaded even with higher flow rates. Treating storm flow rates of that magnitude without any problem is a foundamental advantage of pond systems compared to technical sewage treatment plants, which are able to treat only 2-4xDWF under suitable conditions. (Bucksteeg 1982)

- Fig. 7 shows the reduction of ammonia in facultative ponds. Specific surface areas up to $15\ m^2/i$ give considerable NH_4-reductions. The efficiency for nitrification averages to about 0.5 g NH_4-N/(m^2.d). Nitrification and denitrification take place simultanously and successively. Nitrate is reduced partially to nitrogen gas which escapes from the system into the atmosphere.
  Fig. 8 gives survey on the extent of nitrification and denitrification during hot and cold saisons. The increase of ammonia as well as of nitrate effluent concentrations is evident in the cold saison of Central Europe.

Fig. 7. NH_4-N-effluent results (averages) of facultative ponds depending on specific pond area (Bucksteeg and Schleypen 1983)

Fig. 8. Ammonia and nitrate effluent concentrations (averages) of facultative ponds during summer and winter saisons (Schleypen 1987)

Fig. 9. PO_4-P-effluent results (averages) of facultative ponds depending on specific pond area (Schleypen 1982)

Fig. 10. Phosphate and total phosphorus concentrations in the effluent of a maturation pond depending on raising pH caused by algae growth (Schleypen 1985)

- Fig. 9 shows phosphate elimination figures. Formation of calciumphosphate (hydroxylapatit) and calciumcarbonate due to the fact of raising pH caused by algae growth as well as incorporation of phosonate into algae cells eliminate dissolved phosphate considerably. Calciumphosphate and calciumcarbonate precipitate and act as nuclei for copreciptitation with algae. It can be seen that specific pond areas of more than 10 m^2/i leed to effluent values of less than 6 mg PO_4-P/l.
- Fig. 10 shows the phosphorus alimination in a maturation pond depending on raising pH.
- Some idea on the bacteriological efficiency of ponds may give fig. 11. Two maturation ponds after technical plants were sampled. It can be seen that colicounts show a decrease of two ten-powers when the detention time amounts to 9 days. One day detention time does not leed to considerable improvements.

Fig. 11. Fecal coli counts of influents and effluents of two maturation ponds (Schleypen 1985)

Summarizing it can be said, that German Effluent Quality Standards can be fulfilled by facultative ponds with a specific surface area >5 m^2/i under dry weather flow- as well as under storm flow-conditions (and under winter-conditions, too). With a specific surface area of about 10 m^2/i even unfavourable situations, for example short time shock loads caused by liquid manure or silage wastes can be solved without great difficulties. The following effluent results can be met in case of ordinary domestic sewage to be treated:

		(concentrations of raw domestic sewage)
COD	< 90 mg/l	(300 - 700)
BOD_5	< 25 mg/l	(200 - 400)
NH_4-N	< 15 mg/l	(30 - 60)
PO_4-P	< 6 mg/l	(8 - 15)

Odour nuisance, which may araise from facultative ponds, is reduced considerably by removing the sludge settled before its surface has grown up near to or above the minimum water level. The sludge surface should be covered by a water layer of at least 30 cm. (see fig. 12a, b)

Fig. 12a. Cross section through a not well operated pond (Bucksteeg 1982)

Fig. 12b. Sludge should be removed from the pond (Bucksteeg 1982)

The greatest disadvantage of facultative ponds is the large area needed. For that reason we were looking for solutions which include the advantages of facultative pond systems and prevent the disadvantages. Alternatives are: artificially aerated ponds or ponds combined with trickling filters or ratating bio-filters. (Bucksteeg 1982, 1983)

SUMMARY

Facultative pond systems as well as artificially aerated ponds and ponds combined with trickling filters or rotating bio-filters are able to fulfill very high effluent quality standards. Pond effluents can be reused for agricultural purposes under certain precautions and restrictions. (Bucksteeg 1982, 1983)

In the table shown below all pond systems in use in Germany are compared with each other.

TABLE: Comparison of Pond Systems

		Facultative pond systems	Artificially aerated ponds	Ponds combined with trickling filters or rotating bio-filters
Consumption of area	(m^3/ie)	15 - 20	2 - 4	1.5 - 3
Energy consumption	(kWh/ie.y)	0	25 - 50	7 - 25
Time required for maintenance	(h/week)	2 - 3	3 - 6	4 - 8
Construction costs	(DM/ie)			
for plant seize	500 ie	400 - 600	-	-
	1000 ie	300 - 500	200 - 600	400 - 600
	5000 ie	-	200 - 300	200 - 300
Annual costs	(DM/ie.y)			
for plant seize	500 ie	25 - 35	-	-
	1000 ie	20 - 30	30 - 40	30 - 40
	5000 ie	-	15 - 20	15 - 20

Cost advantages of ponds compared with technical plants shows fig. 13..

Fig. 13. Comparison in construction- and operation costs between ponds and technical treatment plants

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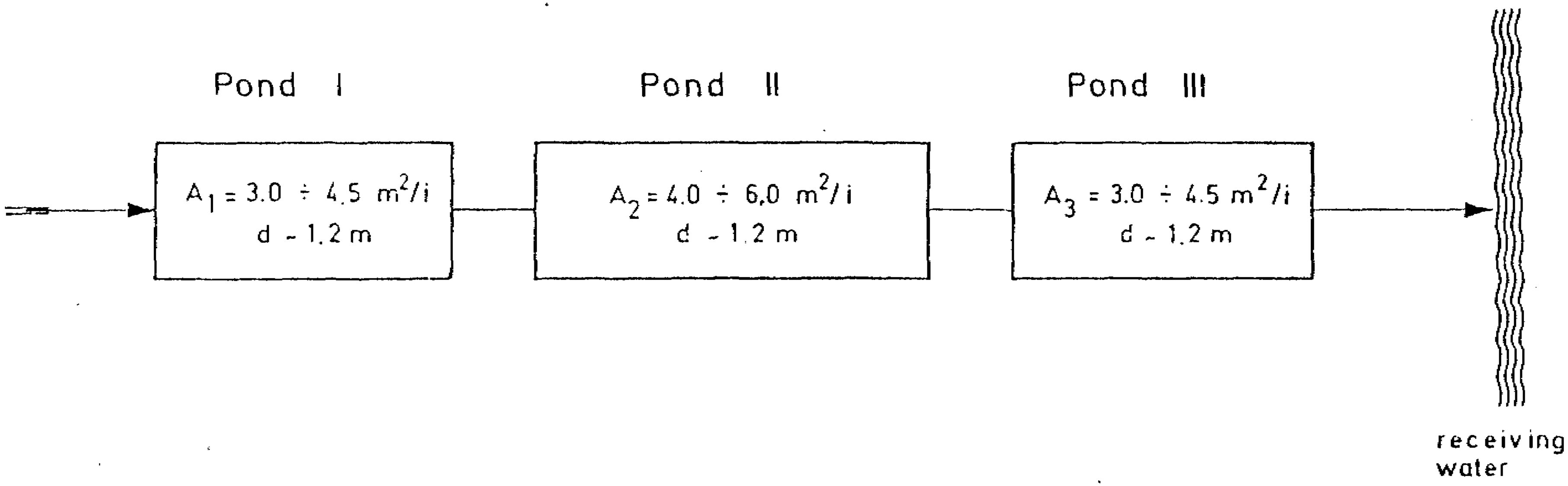
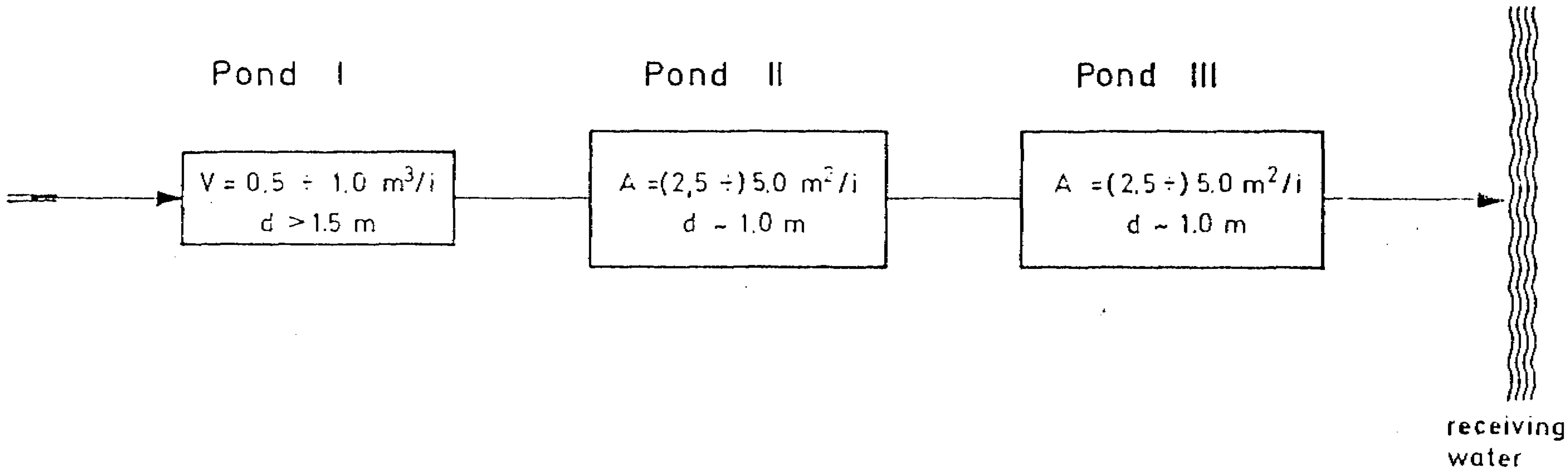
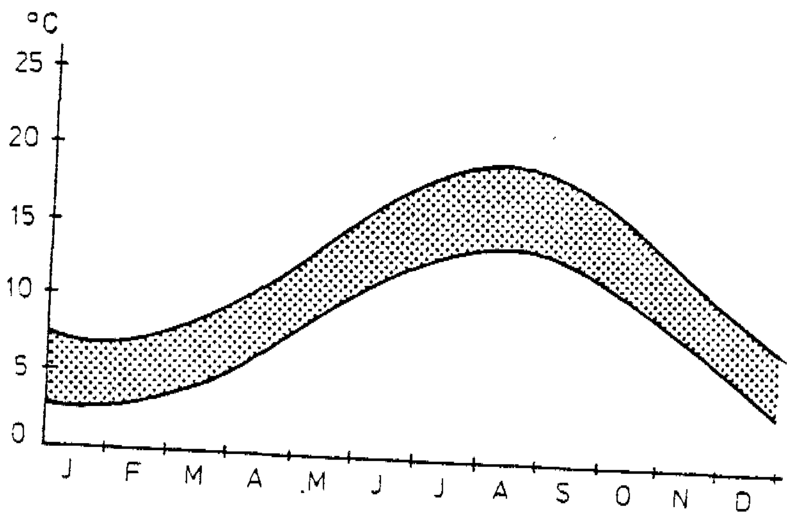


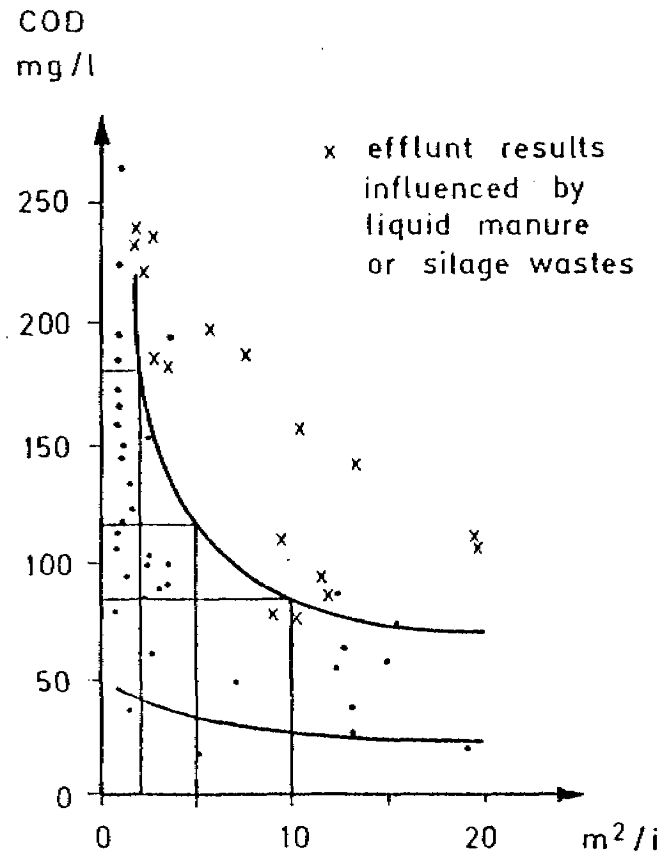
Facultative pond system „A„



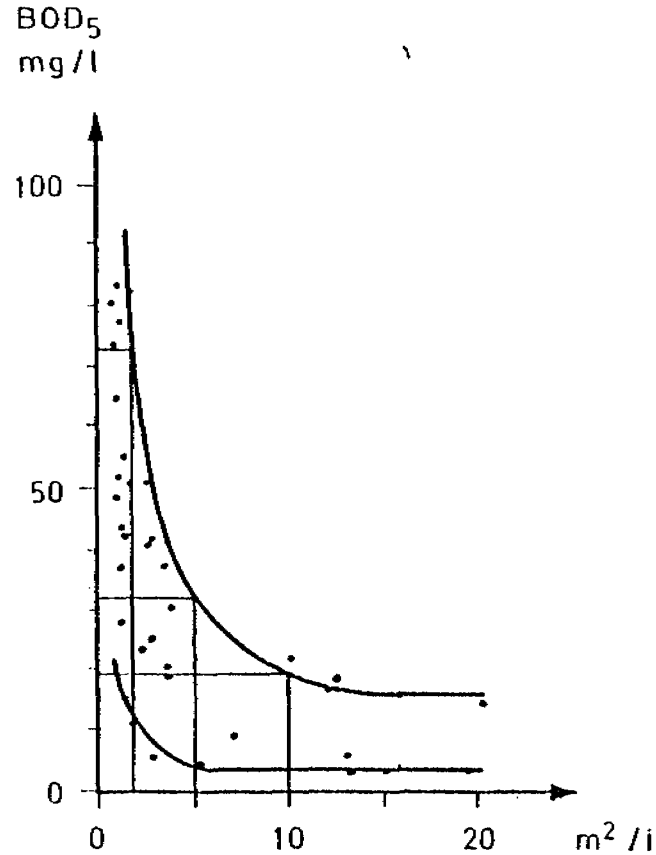
Facultative pond system „B„



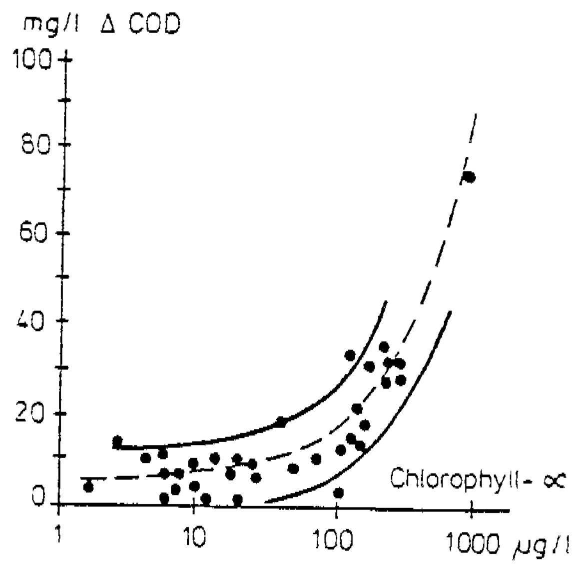
Typical monthly temperature distribution
of pond influents in rural villages in Germany



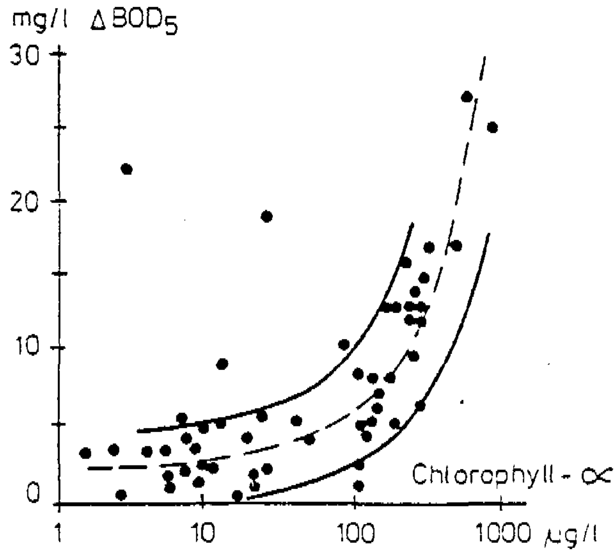
COD-effluent results of facultative ponds depending on the specific pond area (averages from <u>non-filtrated</u> samples)



BOD_5-effluent results of facultative ponds depending on the specific pond area (averages from <u>non-filtrated</u> samples)



Increase of COD in pond-effluents caused by algae



Increase of BOD_5 in pond-effluents caused by algae

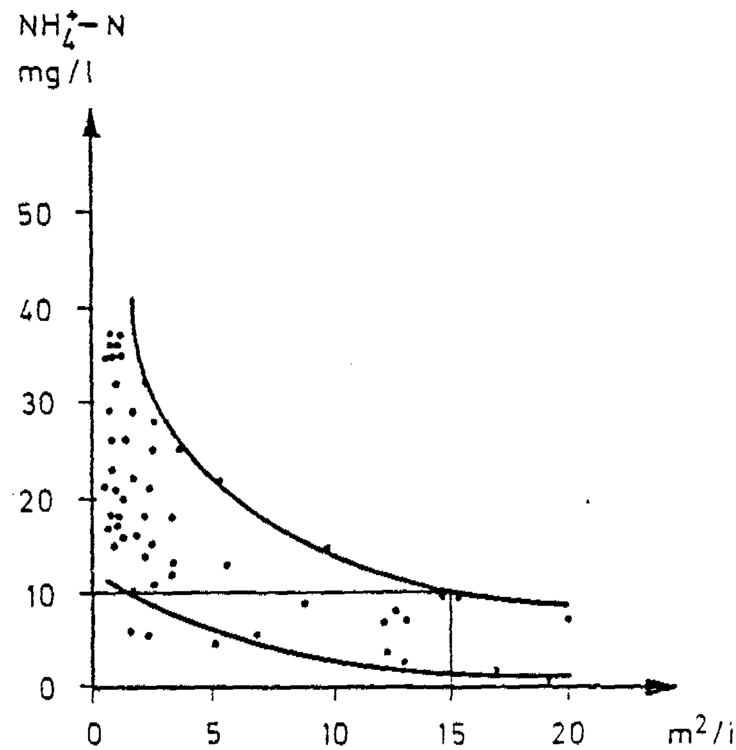
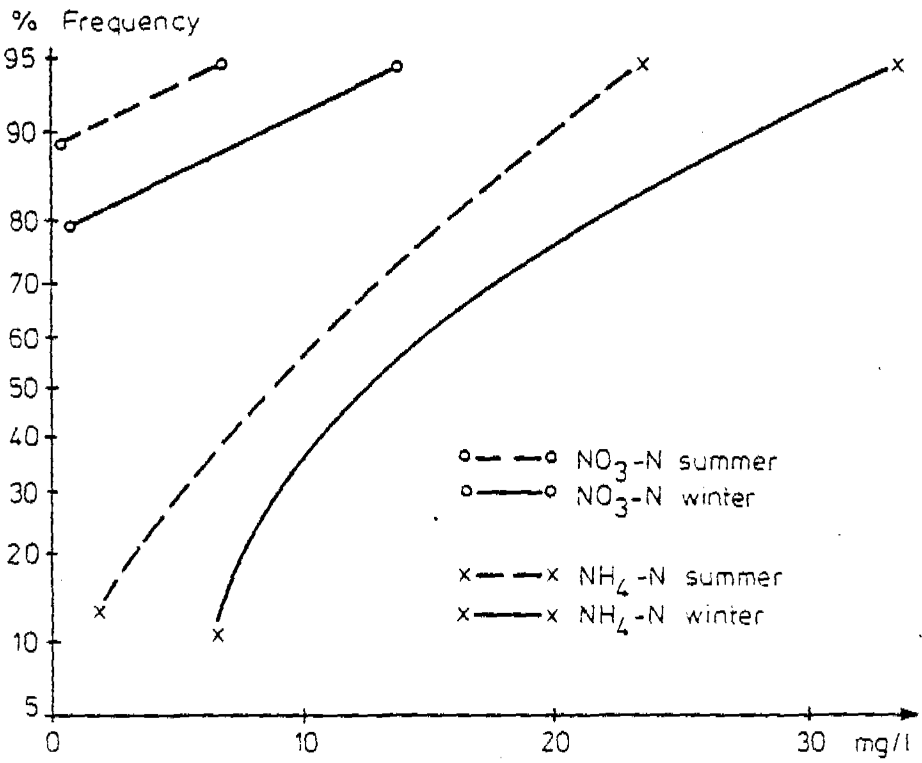
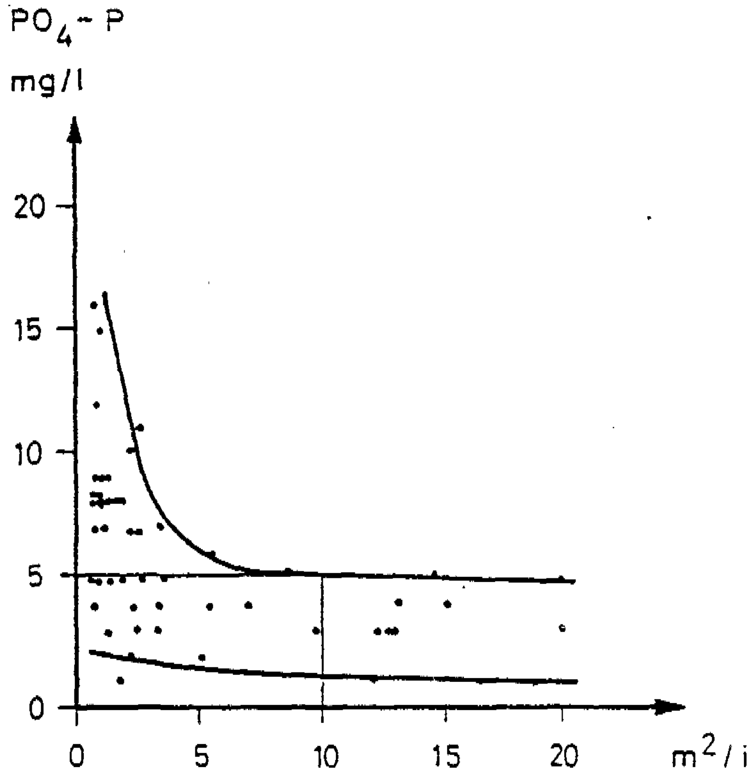


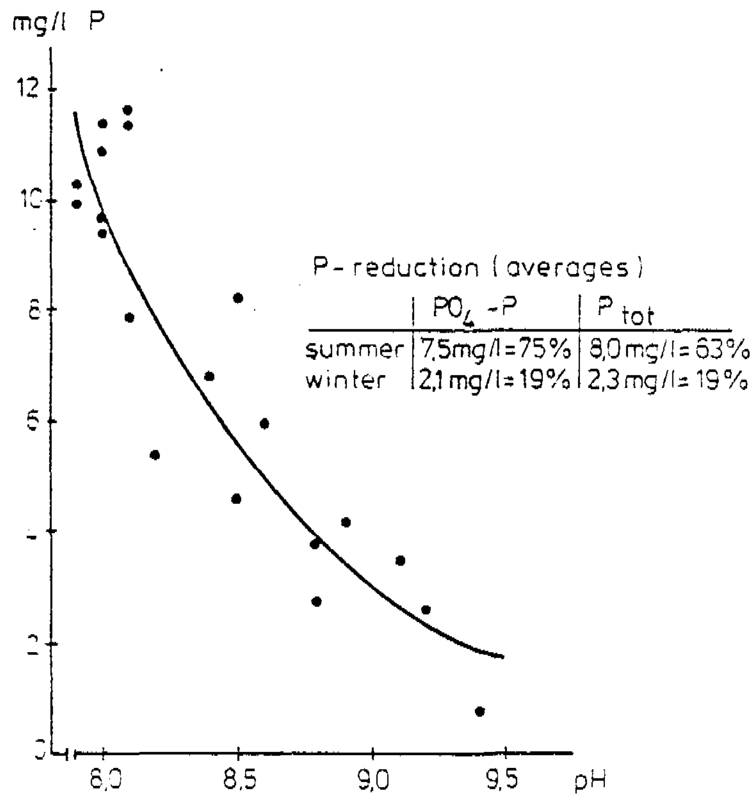
Fig. 6
NH_4^+-N-effluent results (averages) of facultative ponds depending on the specific pond area



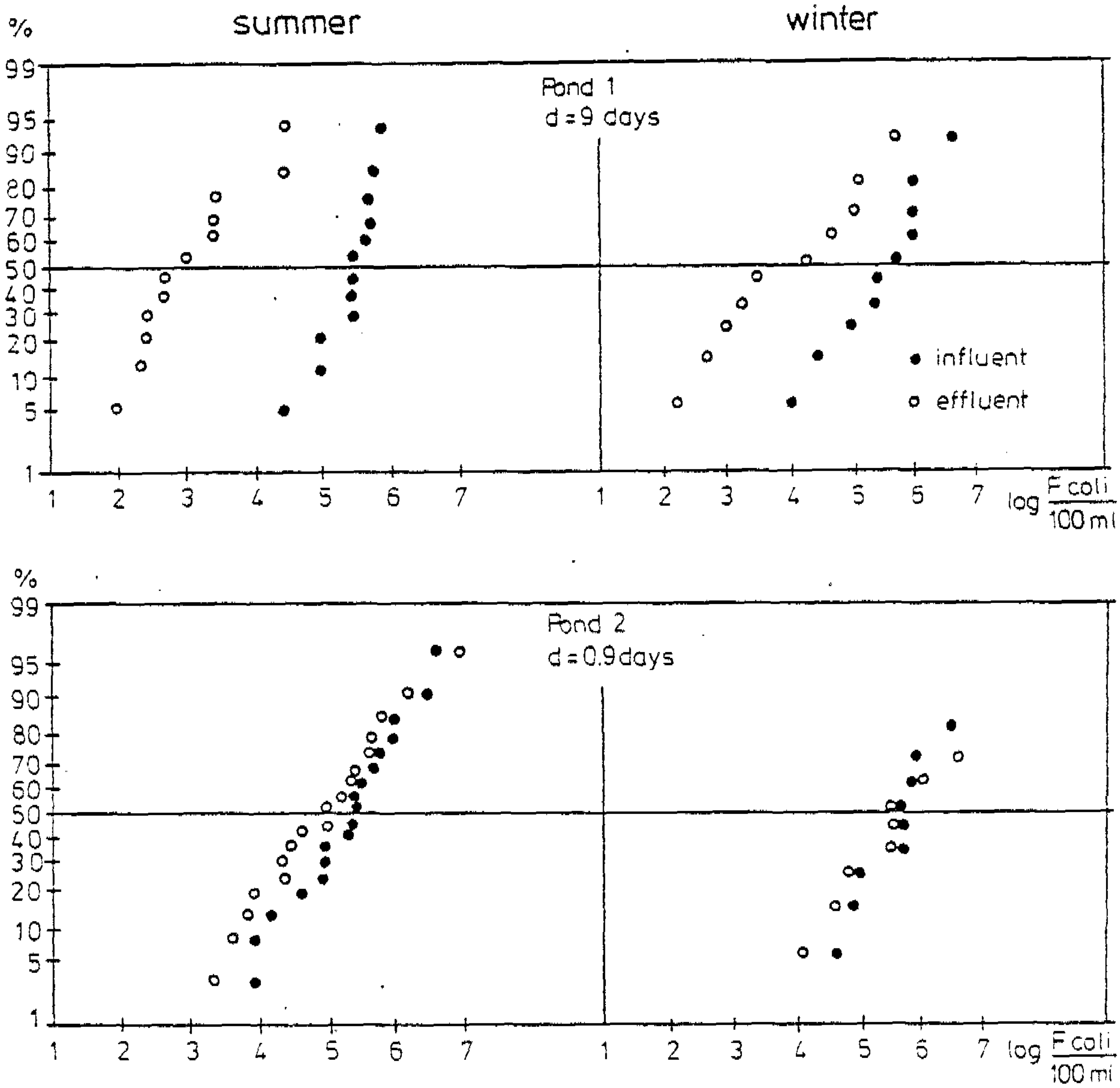
Ammonia and nitrate effluent concentrations (averages) of facultative ponds during summer and winter saisons



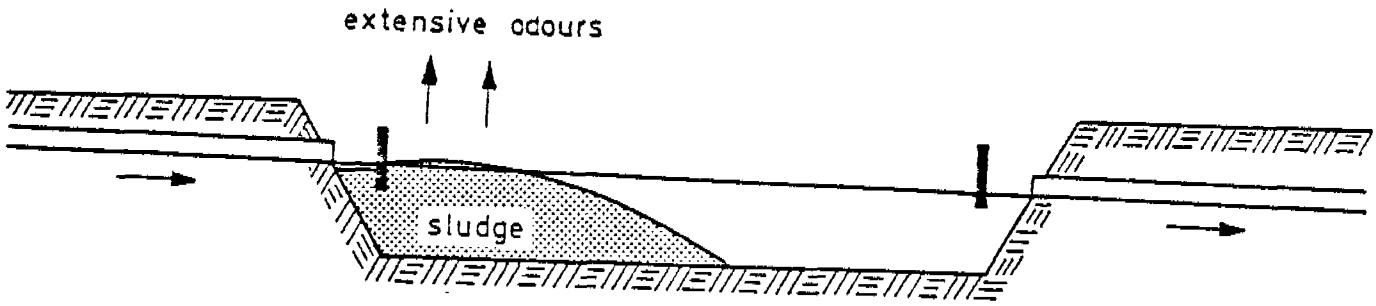
PO_4-P-effluent results (averages) of facultative ponds depending on specific pond area



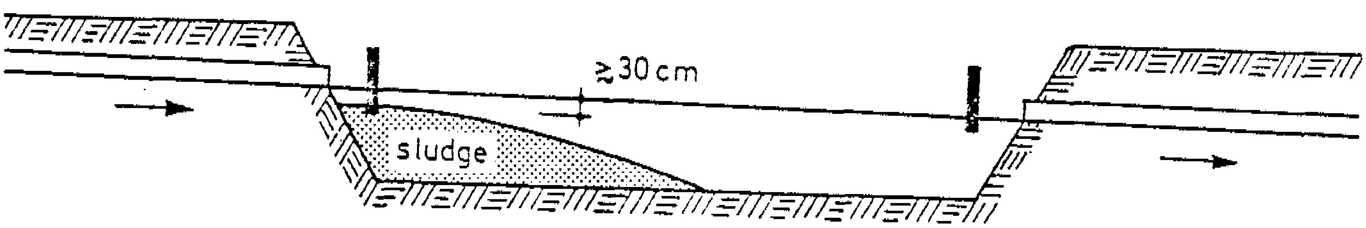
Phosphate and total phosphorus concentrations in the effluent of a maturation pond depending on raising pH caused by algae growth



Fecal coli counts of influents and effluents of two maturation ponds

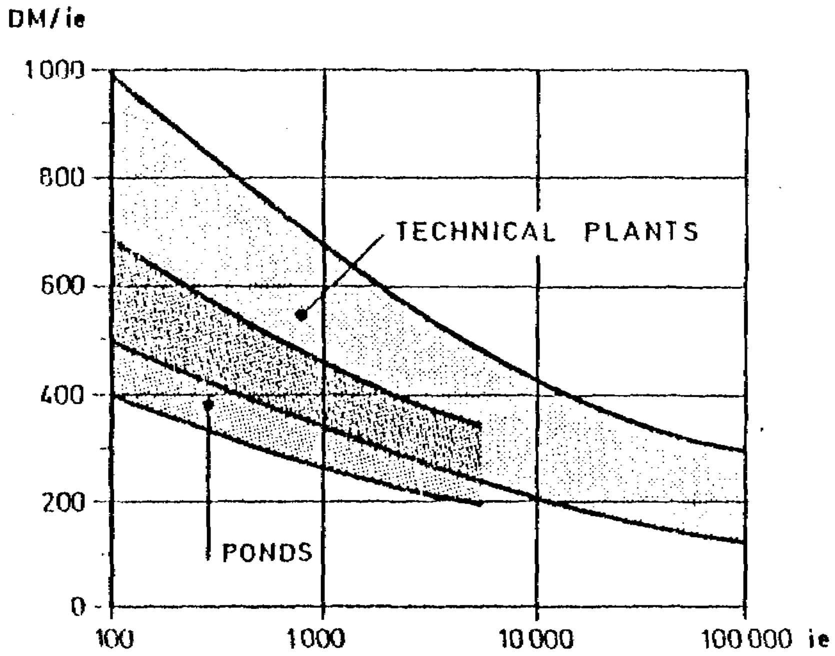


Cross section through a not well operated pond

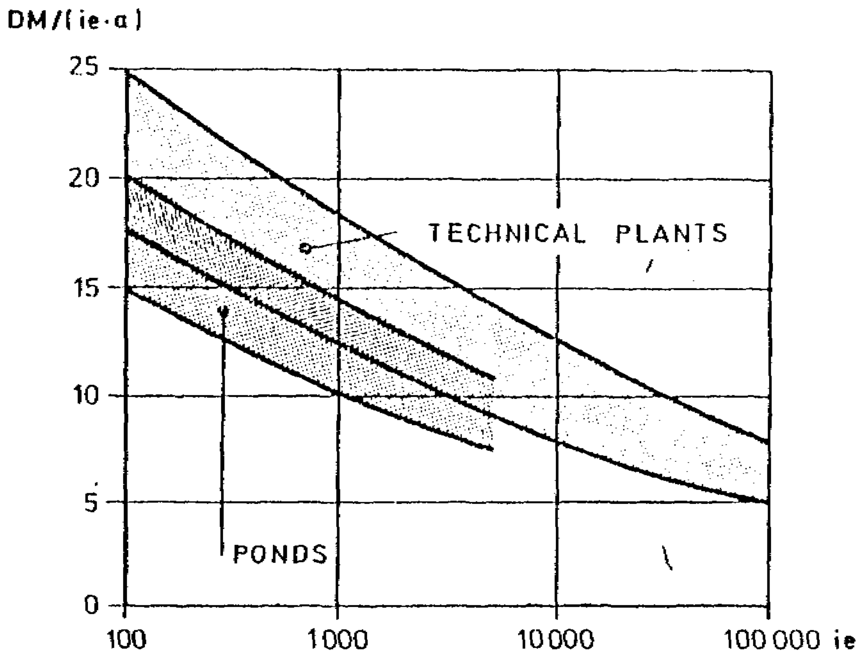


Sludge should be removed from the pond

CONSTRUCTION COSTS



OPERATION COSTS



WASTE STABILIZATION PONDS IN FRANCE: AN OVERALL REVIEW

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ABSTRACT

In 1983 and 1986, surveys were conducted all over France on wastewater stabilization by "natural" ponds (lagoons), including an almost exhaustive inventory of the 1400 plants then operating. The statistical interpretation of collected data made it possible to analyse the spreading of this technique, which became truly popular only ten years ago, and to bring out usual design basis and regional features. With a mean plant area of 5500 m2, lagoons are usually restricted to the treatment of effluents from rural communities, with the exception of large plants in coastal touristic areas. Partially planted ponds appear a competing alternative to conventional algal lagooning. Current conditions of pond management are reviewed, with results on effluent quality.

KEYWORDS

Wastewater, biological treatment; wastewater stabilization ponds, lagooning; survey; France.

INTRODUCTION

Two surveys (1983; 1986) were performed for a more accurate knowledge of wastewater stabilization ponds in France. They only dealt with "natural" ponds (NSP) for full treatment of urban sewage. Polishing ponds, industrial plants and aerated lagoons were considered as being out of the field. A questionnaire was forwarded to 90 departments (mean area: 6000 km2); the highly urbanized Paris region was excluded from the mailing list. They all answered through the local service in charge of technical advising to plant managers (SATESE).

DEVELOPMENT OF LAGOONING

The very first plants (Le-Grau-du-Roi, 15 ha, 1965) were integrated to touristic equipment along the Mediterranean shore. However ponds were reluctantly considered until instructions from the Ministry of Health gave them an official unrestrictive acknowledgement and defined adapted standards on effluent quality (1976). Then the process developed quickly: 6 new ponds only in 1975, but 115 in 1980 and 207 in 1985 (Fig. 1 and 2). Reported systems (1986) amount to 1289 (total area: 715 ha) with a likely underestimation of 10% as a result of usual delays to reach the SATESE system. It is assumed that there will be about 1500 "natural" stabilization ponds (NSP) in France in the middle of 1987. Such figures represent nearly 20% of all treatment systems and 25% of the new ones in rural areas (over 50% in some departments). The estimated treatment potential reaches 30-40 tBOD/d, only 2-3% of the capacity of all types of plants. The mean number of plants is 13 per department, the median value 5. Mean and median of total plant area are 8.4 ha and 3.3 ha (Fig. 3).

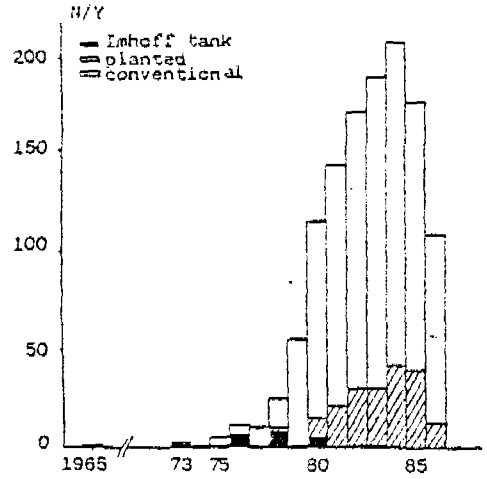


Fig. 1. Total number of new NSP per year

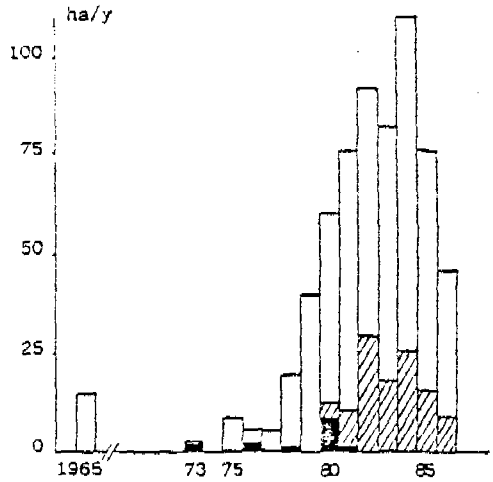
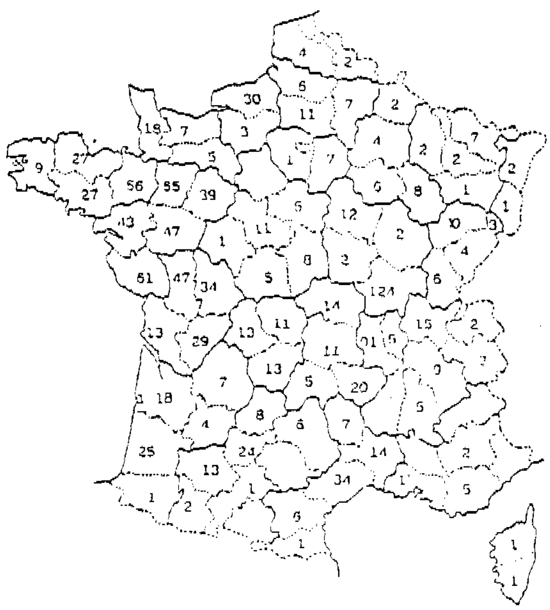


Fig. 2. Total area of new NSP per year

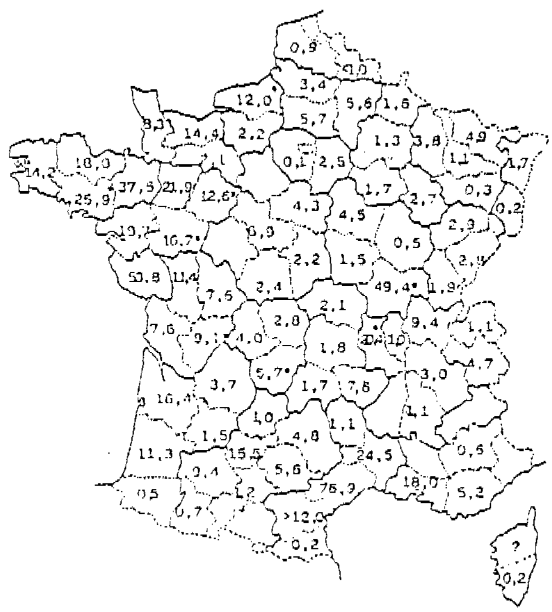
Most NSP are of the conventional (algal) type. 36 out of the older systems had an Imhoff tank, but this combination was never commonly widespread and is now outmoded. Since 1980, increasing interest is shown to ponds partially planted with reeds, rushes and other emergent hydrophytes.

GEOGRAPHICAL DISTRIBUTION OF PONDS

NSP are inequally distributed all over the territory (Maps I and II). Beyond the crowded Paris region, 5 departments only disregard them, whereas more than 40 systems are available in 7 and up to 124 in 1. Ten out of 90 departments receive half the NSP, ten (not always the same) half the whole basin area.



MAP 1. Total number of NSP per department (1986)



MAP 2. Total area of NSP per department (1986)

High numbers were registered on and nearby the old formations of the **Massif Armoricain** and on the North-East border of the **Massif Central**, both regions with scattered human settlements, often supplied by unitary sewerage systems. An exceptional density is reached in the Southern part of the Burgundy vineyards. Outstanding figures were also recorded in the North-West and South-West coasts, and above all on the Languedocian shore, with unusually large systems (up to 18 ha). Besides, NSP are scarce from the Paris basin to Belgium and Germany as well as in the mountainous Alps and Pyrenees and in the touristic **Côte d'Azur**. Industrial, highly populated or hilly areas and high prices for suitable sites appear negative factors. On the contrary, geological and pedological considerations do not seem dissuading.

STATISTICAL DISTRIBUTION OF POND AREAS

The overall mean area of NSP is 5500 m2, **i.-e.** a treatment capacity for 500-600 persons: this figure clearly highlights the rural nature of the process. For further statistical analysis, 12 classes (S1 to S12) were sorted out according to the total plant area (range: 330 m2 - 33 ha; geometric ratio = 1.778). The size is conventionnally described as: very small (330 - 1850 m2), small (1850 m2 - 1.04 ha); medium (1.04 - 5.8 ha) and large (5.8 - 33.0 ha).

The current distribution of total areas may be interpreted as the sum of two log-normal distributions, the first one concerning 97% of the territory and 96% of NSP, the other the Languedocian region only (Fig. 4). For the major group, the mean area is 4800 m2, the median area 3300 m2. Half the systems cover the 2000 - 6500 m2 range, 80% the 1040 m2 - 1.04 ha range. Only 10% of NSP cover an area over 1,04 ha. This last figure corresponds to the median surface registered in the totally different West-Mediterranean sample. At a national level, 9 large plants only (5.8-18 ha) are reported. A 32 ha pond has been planned.

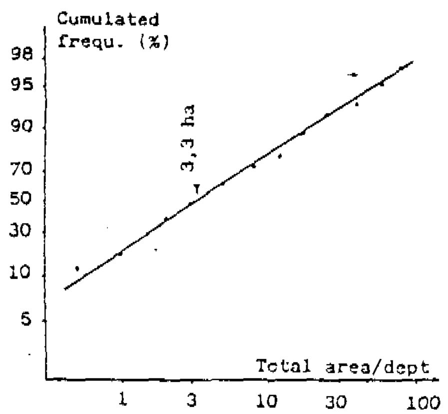


Fig. 3. Statistical distribution of total NSP area per department

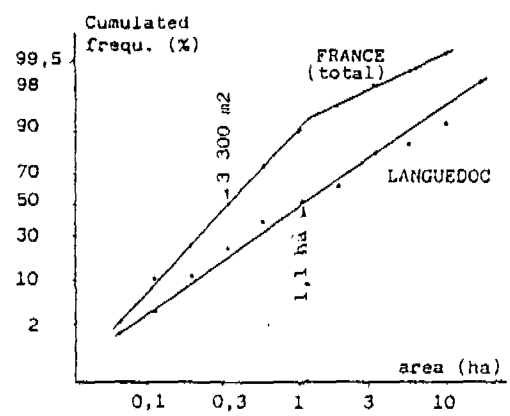
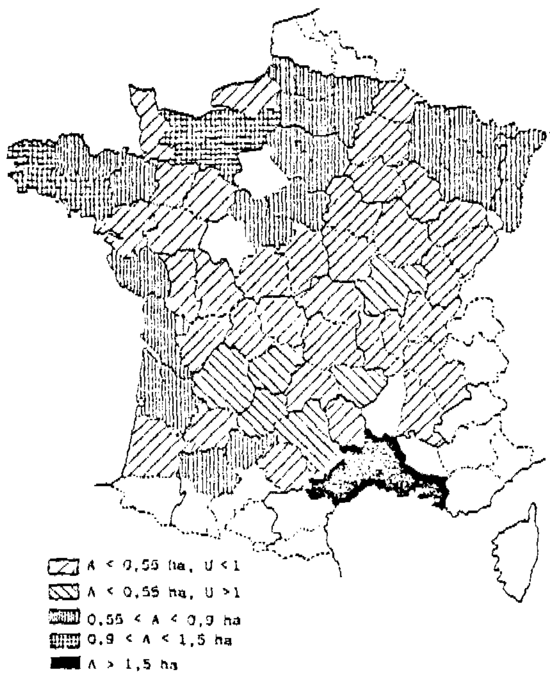


Fig. 4 Statistical distribution of NSP areas (whole France and Languedoc only)

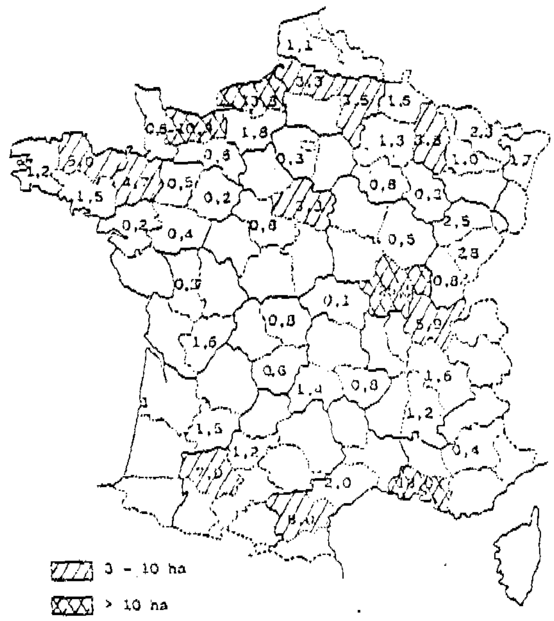
In most departments, pond areas are log-normally distributed. In order not to give an excessive weight to exceptional units, the mean area **A** and the variation coefficient **V** are not directly computed, but deduced from this statistical model. The pair **(A,V)** fully characterizes the departmental distribution. Both parameters were computed for departments with a sufficient number of plants (> 12); several departments were grouped together when this minimal number was not reached. The graphical report of **A** and **V** allows to individualize 4 categories:

- -- Ia: A < 5500 m2 and V < 1,
- -- Ib: A < 5500 m2 and V > 1,
- -- II: 6000 < A < 15000 m2,
- -- III: A > 22000 m2.

The geographical distribution (Map III) indicates a large central strip from Brittany to the Rhone valley (Ia and Ib) with small ponds supplying scattered villages and hamlets. Larger units (II) are predominant on both sides for geological, relief and population reasons. Three Mediterranean departments constitute the last group.



MAP 3. Geographical distribution of NSP according to mean area and variation coefficient



MAP 4. Total area of (partially) planted NSP per department

As far as the size is concerned, no chronological evolution appears. It is impossible to distinguish the distributions of plant areas (at least under 1 ha) for the 1973-80, 1981-84 and 1985-86 periods. As soon as lagooning was accepted as a competitive and efficient treatment system, its possibilities were truly acknowledged, even for large plants. This overall stability persists at the departmental level, and offers thus a powerful tool for local investment planning. In some well-equipped departments, the annual number of new lagoons is currently decreasing because of saturation (Fig. 5). Such an effect may modify the geographical distribution of the process in the years to come.

DESIGN OF ALGAL PONDS

The currently recognized design basis for algal ponds is a surface load of 5 gBOD5/m2 (10 m2 per cap.) except in touristic, seasonally overpopulated areas where 7m2 per cap. are tolerated. Most lagoons have 2 or 3 basins (43% each); the 1-basin and 4 or 5-basin systems are not as widespread (11 and 3%).

Among very small plants, single (23%) and double basins (51%) are more widespread than 3-compartment systems (25%). For small plants, 2 and 3-basin plants are almost as numerous (43 and 48% resp.). The most common plants (71%) are triple plants for medium and large NSP. The median area increases thus with the number of compartments (Fig. 5). Reported figures confirm the wide use of the conventional pattern for 2 and 3-basin ponds (1/2-1/2 and 1/2-1/4-1/4 of the total plant area respectively): in 3 ponds out of 4, the first compartment covers 50-60% of the total area.

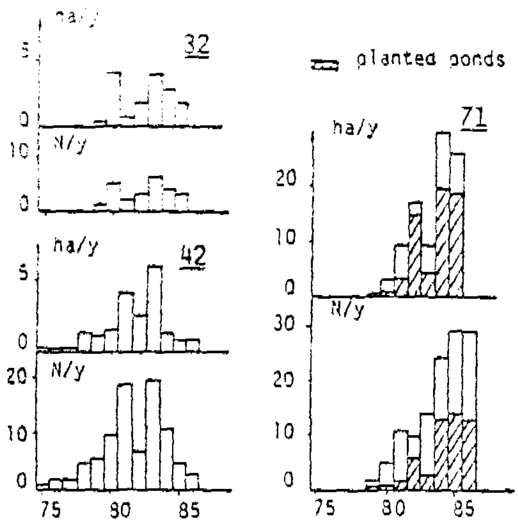


Fig. 5. Number (N/y) and total area (ha/y) of new NSP per year in 3 typical departments

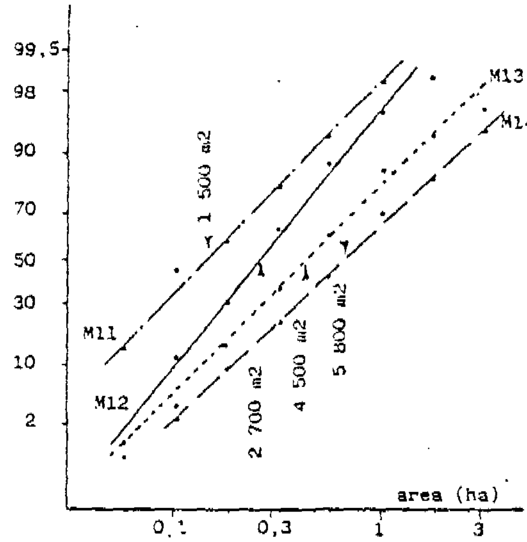


Fig. 6. Statistical distribution of NSP areas according to the number of basins (1 b.:M11, 2b.:M12, etc.)

87% of alternative ponds are between 1.0 and 1.2 m deep, 10% between 1.2 and 1.5 m, with a trend towards a deeper (+ 10-20 cm) first compartment for sludge accumulation.

Widely used in other European countries and despite some recent examples in France, the alternative of an anaerobic pond ahead appears exceptional for fear of odours.

"PLANTED" PONDS

The reported number of planted systems is 203 (17% of all ponds; total area: 147 ha ≈ 21% of the total surface of NSP). They are scattered mainly throughout the North-East half territory (Map IV). Some departments which recently converted to lagooning adopt this technique only, although most regions with a long tradition appear reluctant.

Most ponds of this type associate algal and planted basins: only 35 ha are currently covered by emergent hydrophytes (23% of the whole area of planted systems). Various combinations of algal (AL), totally (MA) or partially (MX) planted basins are encountered. The first basin is hardly ever planted: the usual scheme is AL+AL+MA (56%), but AL+MX (15%), AL+MA (6%), AL+AL+MX and AL+MX+MA (7% each) are also found. Thus planted basins are rather considered as polishing devices, mainly aiming at reducing suspended solids in the effluent. The design basis (10 m2 per cap.) is identical to the current practice for simple algal systems. Compared to the latter, the median (4300 m2) and mean area (7400 m2) of "planted" ponds are higher.

SEWERAGE SYSTEMS - HYDRAULIC LOADING OF PONDS

Since decades in rural areas the trend has been to prefer separated networks when a treatment plant is planned downstream. However, current experience shows that pipes collecting domestic effluent often admit high flows of unwanted waters resulting in effluent dilution and hydraulic overloading. The survey revealed that it is the case for 30% of separated systems but this figure seems underestimated. The drawbacks due to excessive flows are much less awkward with NSP than with conventional biological plants. Given the variability of received flows, the available data are insufficient to evaluate the percentage of plants receiving more water than the design data.

Among sewers feeding natural ponds, unitary systems usually supply smaller units than separated networks. Out of the Languedocian shore, the mean area of NSP receiving waters from unitary and separated sewers is 3850 and 5600 m2 respectively. The reason why seems to be that no treatment plant was initially planned for small communities. NSP later afforded a suitable solution when more stringent requirements were postulated for effluent discharge.

When collection and treatment are planned at once, departments where ponds actually compete with other types of plants do not show any clear-cut advantage for the combined choice of sewerage and treatment systems. There are scarce cases where designers obviously select lagoons for the treatment of flow from unitary systems and prefer more conventional plants for separated collection. Many other factors are obviously interfering.

ORGANIC LOADING

Whereas a fair number of NSP receive excessive hydraulic flows, the results put forward a general organic underloading. From the limited sample for which precise data were available, more than half plants treat less than 50% of their BOD capacity. One out of ten only is overloaded, usually because of industrial effluents.

NSP thus share the common lot of rural wastewater treatment plants in France. It is due to: (a) the population considered, usually anticipating - quite optimistically - the 20 years to come, (b) the organic discharge per capita, less in rural areas than with the conventional values (about 60 gBOD/d) computed from urban surveys, (c) the slowly increasing connection to the sewerage system. On the whole, very small plants better correspond to their design capacity than larger ones and underloading decreases with time.

INVESTMENTS - MANAGEMENT OF PONDS

There is still a lack of in-depth information about investments. Yet, a NSP on a suitable site is generally assumed to cost 20-30% less than a conventional plant. Inadequate sites (expensive land; rocky, sandy or organic soils, etc.) may surcharge comparable or even higher levels and result in a wide dispersion of overall costs.

This process is very popular among rural municipalities mainly because of its easy and cheap maintenance by unspecialized employees. Ponds are usually well maintained. However, some mishaps may occur. They are not all due to slapdash upkeep and are more difficult to solve when they result from inadequate preliminary studies or careless building. Hydraulic underloading and insufficient water-tighness give rise to problems to keep the designed water depth. Duckweed (Lemna ssp.) proliferations may lead to awkward situations when they cover the whole surface: it is then necessary to get rid of them, up to 2 or 3 times from May to October, a rather uneasy operation in large ponds. Hot and dry summers occasionnally bring about deviations due to pink-coloured photosynthetic sulphur bacteria. Such a phenomenon was first described near the Mediterranean coast, but it extended to South-West France and even to Brittany over the last years. NSP are too new and underloaded to provide information about sludge accumulation, desludging periodicity and suitable operational techniques that can be generalized.

EFFLUENT QUALITY

The quality of effluents delivered by NSP was estimated through the statistical interpretation of data collected by the *SATESEs* and gathered by the **Agence de Bassin Loire-Bretagne** (River Loire and Brittany Water Authority). A representative set of 50 plants from 11 departments was constituted (average treatment capacity corresponding to 900 persons, 78% of the plants designed for less than 1000 p.). The mean current organic load is 29 kgBOD5/ha.d (1st basin: 61 kg/ha.d); 78% of the ponds receive less than half the design capacity, i. e. less than 25 kg/ha.d. Results concerning 410 grab samples and 70 full day surveys were available. The overall and seasonal mean values of SS, filtered BOD and COD are given by table 1.

TABLE 1: Effluent Quality from NSP (mg/l)

	Average value	Oct. to Jan.	Feb. to May	June to Sept.
BOD5	23	19	21	31
COD	100	83	94	127
SS	36	31	32	45

Average values for "total" (Kjeldahl) nitrogen and total phosphorus are 23 and 7.8 mg/l respectively.

Effluents are thus more concentrated in Summer. Three factors may justify this seasonal increase: (a) a lesser dilution of raw sewage by unwanted waters entering the sewerage system, (b) higher contents of micro-algae, resulting from a more active photosynthesis, (c) water losses through evaporation, reducing the effluent flow. In the current conditions of NSP underloading in France, SS values are only slightly affected by micro-algae and effluent quality is usually good. Limits of 35 mgBOD5/l, 150 mgCOD/l and 50 mgSS/l are satisfied by 85%, 92% and 86% of samples respectively.

CONCLUSION

After hesitant start, the technique of waste stabilization ponds is now favourably considered in France and so well-appraised by local communities that they too often neglect or underestimate the specific drawbacks and limits of the process. Such an interest results from the flexibility of the system and its capacity to meet the ordinary requirements of the rural world. NSP are also favourably regarded by authorities in charge of the protection of receiving waters, which consider them more reliable on the whole than small conventional plants.

ACKNOWLEDGEMENTS

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EVALUATION OF WASTE STABILIZATION POND PERFORMANCE IN MISSOURI AND KANSAS

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ABSTRACT

Twenty waste stabilization ponds in Missouri and Kansas were evaluated as to their ability to meet the State effluent requirements. It was found that a large number of these ponds were violating the BOD or the suspended solids requirements on many occasions. The performance of these ponds did not correlate well with the traditional design parameters, such as BOD loading rates or mean hydraulic detention time. The use of multiple cells was also not found to be advantageous in all situations. Some of the pond design equations did not provide a valid method of estimating their performance. Effluent polishing methods are necessary to remove excess suspended solids during the critical summer months.

KEYWORDS

Waste stabilization pond, lagoon, performance evaluation, effluent quality

INTRODUCTION

Waste stabilization ponds (lagoons)(WSP) are a common wastewater treatment method in the midwestern U.S. for small systems. Many of these ponds serve small communities, schools, motels and even individual homes. The reasons for their popularity are the low energy usage and low cost of construction and operation, especially in rural areas. Unfortunately, these ponds do not perform as effectively, especially with respect to suspended solids removal criteria, as other secondary biological treatment processes, such as activated sludge and rotating biological contactors. In view of the large number of ponds in service which are not meeting the effluent limitation criteria set up by various states in compliance with federal water pollution regulations, the U.S. Environmental Protection Agency in 1977 relaxed the suspended solids criteria for systems less than 2 mgd (7600 m^3/s) capacity (EPA, 1977). In Missouri the monthly average effluent suspended solids (SS) and biochemical oxygen demand (BOD) should not exceed 80 mg/L and 45 mg/L, respectively. In specific instances, the states could impose a more restrictive standard. It was felt that the relaxed effluent standards would not cause serious damage to the streams and would allow the pond wastewater treatment plants to be in compliance with the regulations.

The study reported herein investigated a number of facultative waste stabilization ponds in Missouri and Kansas for their ability to meet the applicable effluent standards. Out of a total of 40 pond systems considered, only 20 had sufficient detailed information available for further examination and study. Twelve of these ponds are in Missouri and eight are in Kansas. Out of the twenty pond systems studied, there were 6 single-cell units, 6 two-cell units, 5 three-cell units and 3 four-cell units.

In addition, the performances of these ponds in terms of BOD and SS removals were also evaluated in terms of typical design parameters and design equations.

DATA SOURCE

The data used for the WSP performance evaluations were obtained from either Missouri Department of Natural Resources, Division of Environmental Quality, or Kansas Department of Health and Environment. The data for City of Columbia WSPs were obtained from the Department of Public Works files. It is presumed that these analyses were performed in accordance with standard procedures.

PERFORMANCE EVALUATIONS

Table 1 shows the design information and actual performance for the different WSP systems studied. The water depth for these systems was generally about 0.9 m. It can be seen that some of these systems based on the design criteria are hydraulically and organically overloaded, at design capacity or underloaded. There were a few systems for which the performance data was available, but the design data was not available. Table 2 shows the state effluent discharge requirements for the WSP systems.

The BOD and SS performance data for the WSP studied are shown in Figure 1. In the bar diagram the top continuous line is the influent value, the lower continuous line is the effluent value and the dashed line is the effluent discharge requirements. It can be seen that many of the systems do not meet the required BOD and SS effluent standards. Further, there is little relationship between the loading conditions and the performance of the WSP systems. For instance, some WSP systems at design loading conditions or even underloaded conditions are not meeting the standards, while others at overloaded conditions are doing so.

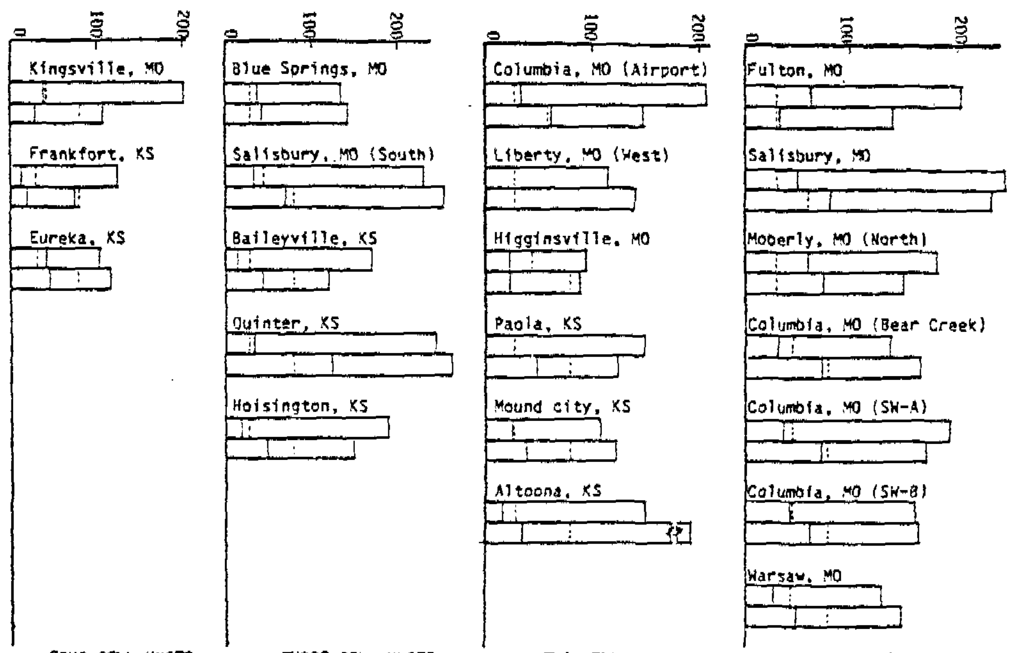

Fig. 1. BOD and SS Removal Performance of WSP Units - Top bar diagram-BOD data; Bottom bar diagram-SS data; Right firm line-influent value; Left firm line-effluent value; Dotted line-State effluent requirements.

Figures 2 and 3 show the relationship of percent BOD and SS removals with BOD loading rate (kg BOD/ha.d) and mean hydraulic detention time (days) for all the WSPs studied. There was no clear relationship evident for BOD removals at different hydraulic detention times and BOD loading rate. It was expected that lower detention time and higher BOD loading rate would lead to poorer performance. This was not the case. In addition, having numerous cells also did not materially improve the situation. The suspended solids removal data was

TABLE 1 Waste Stabilization Pond Design Data and Performance Evaluation

Name	No. of Obs.	Design Basis: Total Surf.Area ha.	Design Basis: Flow Rate m^3/d	Design Basis: Mean Det. Time, d	Design Basis: Areal Loading Rate KgBOD/ha.d	Actual Performance: Flow Rate m^3/d	Actual Performance: Mean Det. Time, d	Actual Performance: Areal Loading Rate KgBOD/ha.d	% BOD Removal	% S.S. Removal
SINGLE CELL UNITS										
1. Fulton, MO (Helms Lagoon) (1/84 - 5/85)	16	0.2	37.85	49	37	34.1	51	35.9	69	78
2. Salisbury, MO (2/83 - 2/85)	8	2.42	601.8	37	49.4	333	67	34.8	80	66
3. Moberly, MO (North Lagoon) (1/83 - 3/85)	27	1.21	227	49	37	283.9	54	40.4	67	50
4. Columbia, MO (Bear Creek-North) (2/82 - 7/83)	39	5.18	1476	31	58.4	2763	17	62.9	75	64
5. Columbia, MO (Swothwest Lagoon)										
A. (4/77 - 2/78)	43	14.32	3550	36	49.4	3785	36	51.6	81	56
B. (3/78 - 1/80)	94	14.32	3550	36	49.4	9614	16	105.5	74	63
6. Warsaw, MO	16	4.37	832	48	38.2	908.4	49	27	78	67
TWO CELL UNITS										
1. Columbia, MO (Airport Lagoon) (1/85 - 12/86)	24	0.17	39.7	39	47	31.8	52	39.3	83	58
2. Liberty, MO (West Lagoon) (1/83 - 1/85)	90	19.3	6370	28	52.7	9235	23	50.5	69	71
3. Higginsville, MO (11/83 - 8 85)	22	11.89	2271	48	38.2	1400	160	11.2	74	73
4. Paola, KS (11/83 - 10/85)	22	--	2460	--	--	2536	--	--	86	59
5. Mound City, KS (1/84 - 10/85)	7	--	484	--	--	1022	--	--	74	67
6. Altoona, KS (1/84 - 10/85)	8	--	302	--	--	530	--	--	88	93
THREE CELL UNITS										
1. Blue Springs, MO (3/83 - 2/85)	84	23.47	4160	52	35.9	10900	24	58.4	70	67
2. Salisbury, MO (S. Lagoon) (2/83 - 2/85)	7	0.57	110	46	40.4	130	85	31.4	86	72
3. Baileyville, KS (2/84 - 10/85)	8	--	49.2	--	--	--	--	--	91	63
4. Quinter, KS (11/83 - 10/85)	8	--	567	--	--	--	--	--	86	53
5. Hoisington, KS (3/84 - 9/85)	7	--	2010	--	--	--	--	--	89	67
FOUR CELL UNITS										
1. Kingsville, MO (3/84 - 8/85)	17	--	193	120	--	162.8	142	--	79	72
2. Frankfort, KS (2/84 - 10/85)	8	--	908	--	--	402	--	--	92	74
3. Eureka, KS (12/83 - 11/85)	7	--	1067	--	--	1968	--	--	60	63

TABLE 2 NPDES Effluent Discharge Requirements for the Waste Stabilization Ponds Studies and Effluent Violations

Name	Effluent Quality Requirements				No. of Obs.	No. of BOD Violations	No. of S.S. Violations
	BOD_5	S.S.	pH Range	Frequency of Monitoring			
SINGLE CELL UNITS							
1. Fulton, MO (Helms Lagoon)	30†	30†	6-9	Quarterly	16	16	8
2. Salisbury, MO (Large Lagoon)	30*	60*	6-10	Monthly	23	21	15
3. Moberly, MO (North Lagoon)	30†	30†	6-9	Monthly	27	26	20
4. Columbia, MO (Bear Creek Lagoon)	45§	80§	6-9	Monthly	39	9	8
5. Columbia, MO (Southwest Lagoon) A.	45§	80§	6-9	Weekly	43	7	15
" " " B.	45§	80§	6-9	Weekly	94	34	38
6. Warsaw, MO	45*	80*	>6	Monthly	16	2	6
TWO CELL UNITS							
1. Columbia, MO (Airport Lagoon)	30*	60*	6-9	Monthly	24	11	10
2. Liberty, MO (West Lagoon)	30†	30†	6-9	Weekly	90	46	56
3. Higginsville, MO	45*	80*	6	Monthly	22	1	0
4. Paola, KS	30*	80*	6-9	Quarterly	22	3	4
5. Mound City, KS	30*	80*	6-9	Quarterly	7	1	1
6. Altoona, KS	30*	80*	6-9	Quarterly	8	1	1
THREE CELL UNITS							
1. Blue Springs, MO	30†	30†	6-9	Weekly	84	61	59
2. Salisbury, MO (South Lagoon)	45*	80*	>6	Monthly	7	2	1
3. Baileyville, KS	30*	80*	6-9	Quarterly	8	0	0
4. Quinter, KS	30*	80*	6-9	Quarterly	8	6	7
5. Hoisington, KS	30*	80*	6-9	Quarterly	7	1	1
FOUR CELL UNITS							
1. Kingsville, MO	45*	80*	>6	Monthly	17	4	0
2. Frankfort, KS	30*	80*	6-9	Quarterly	8	0	0
3. Eureka, KS	30*	80*	6-9	Quarterly	7	2	1

* Monthly Average
§ Weekly Average
† Daily Average

very similar to the BOD data with no clear effect of BOD loading rate, detention time or number of cells on the performance.

Figure 1 indicated that several of the WSP systems did not meet the state effluent requirements even when the entire study period average data (usually two years) was used to estimate the influent and effluent quality. The number of violations during the study period is also reported in Table 2. The violations of the pH limits are not included in Table 2. Table 2 shows that except for two systems studied (Baileyville and Frankfort, Kansas), all violated the state effluent requirements and some did so persistently. Many of the violations occurred despite the relaxed effluent criteria for SS and BOD.

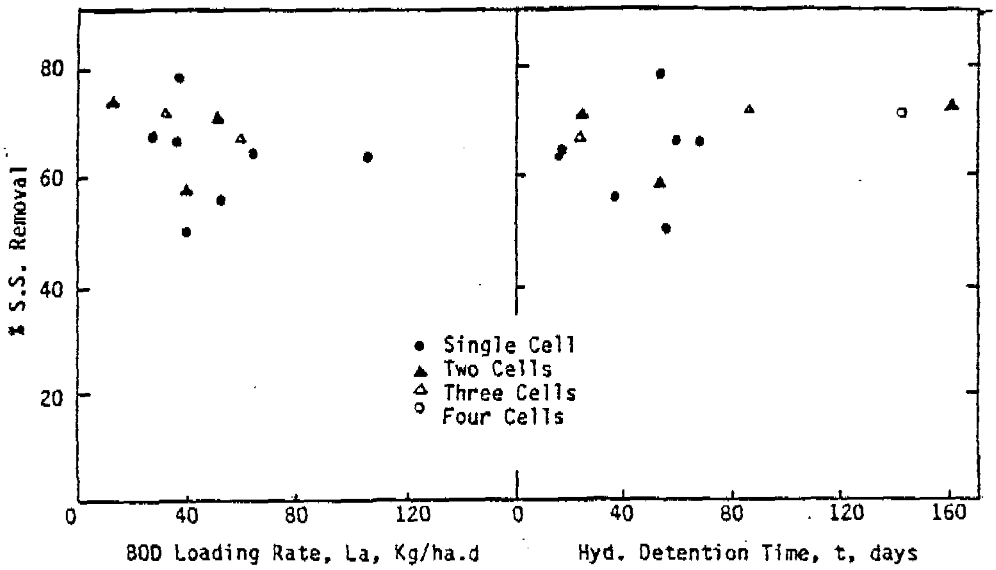

Fig. 2. Relationship between BOD Loading Rate, Hydraulic Detention Time and % S.S. Removal by WSP Units

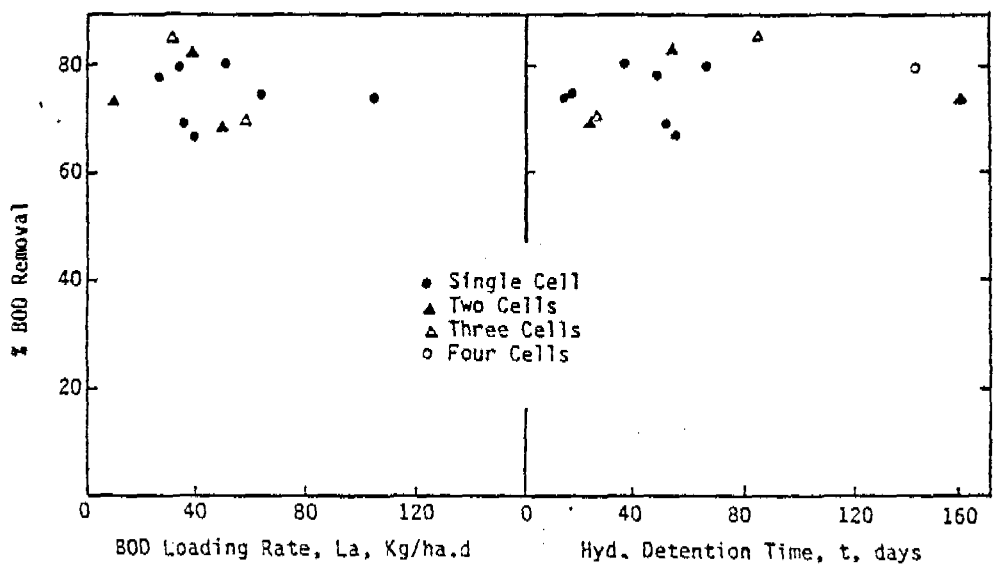

Fig. 3. Relationship between BOD LOading Rate, Hydraulic Detention Time and % BOD Removal by WSP Units

DESIGN EVALUATIONS

As shown in Figures 2 and 3, the performance of the WSP studied did not correlate well with any of the empirical design parameters, such as BOD loading rate or detention time.

Gloyna (1976) has proposed the following empirical equation for determining the volume of facultative WSP:

$$V = 3.5 \times 10^{-5}\ Q\ Lu\ (1.085^{35-T})ff' \qquad (1)$$

V = pond volume, m^3; Q = inflow rate, L/d

Lu = influent ultimate BOD_u, mg/L; T = pond temperature, ^oC

f = algal toxicity factor, and f' = sulfide oxygen demand

The ponds designed on this basis were expected to give 80 to 90% BOD removal efficiency. This equation was utilized to determine the pond volume of selected WSP systems. These calculated pond volumes for 5^o and 10^oC are shown in Table 3. Factors f and f' were assumed to be one. Influent BOD_5 values were converted to BOD_u by multiplying by 1.5. No solar radiation corrections were made in these calculations.

TABLE 3 Waste Stabilization Pond Volume Comparisons

Name	Actual Volume, m	Av. % BOD Removal	Volume as per Eq.(1), m^3 5°C	10°C
1. Fulton, MO - single cell (Helms Lagoon)	1850	69	4233	2816
2. Salisbury, MO - single cell	22205	80	49619	32993
3. Liberty, MO - two cells	176534	69	644073	428226
4. Columbia, MO - two cells (Airport)	1555	83	3971	2640
5. Blue Springs, MO -three cells	214654	70	892344	593351

The data in Table 3 indicates that the pond volumes calculated by Gloyna's equation are much larger than the actual volumes. However, despite much smaller pond volumes, in two cases the BOD removals were in the same range as expected by the use of Equation 1. Therefore, the validity of the equation for the design of pond volume is questionable.

Thirumurthi (1974) has suggested the use of the equation:

$$\frac{Ce}{Ci} = e^{-Kt} \qquad (2)$$

where Ce = effluent BOD, mg/L;
Ci = influent BOD, mg/L;
t = Mean Hydraulic Detention Time
K = First order BOD removal coeffieient, $(day)^{-1}$

for determining the performance of a facultative waste stabilization pond which has a plug flow regime. K value must be corrected for temperature, organic loading rates and the presence of industrial toxic chemicals in accordance with the equation:

$$K = K_S C_{Te} C_O C_{Tox} \qquad (3)$$

where K_S = Standard BOD removal

C_{Te} = Temperature correction factor = $1.036^{(T-20)}$

T = Average temperature in the coldest months, ^oC

C_0 = Organic load correction factor = $1 - \frac{0.083}{K_S} [\log \frac{67.2}{La}]$

L_a = Organic load in Kg/ha.d

C_{Tox} = Industrial toxic chemical correction factor.

The WSPs studied had no artificial mixing and probably were more close to a plug flow than a completely mixed flow regime. Equations 2 and 3 were used to determine the K and K_S values for winter operation of selected WSP systems. Table 4 shows these values.

TABLE 4

Name	Av. Pond Temp. ^oC	C_i/C_e	La Kg/ha.d	t, d	K $(d)^{-1}$	K_S $(d)^{-1}$
1. Fulton, MO	0	2.39	29.5	61	0.014	0.054
2. Salisbury, MO	8.3	6.00	39.2	58	0.030	0.064
3. Liberty, MO	2.0	2.49	50.2	31	0.012	0.034
4. Columbia, MO (Airport)	3.0	4.00	42.2	47.5	0.029	0.069
5. Blue Springs, MO	1.5	11.47	48.1	37	0.066	0.136

The wide variations in the calculated value of K_S indicate that factors other than those considered may influence the pond BOD removal behavior. Finney and Middlebrooks (1980) also found that the calculated K_S values for different ponds had a wide variation. Thus, it is unlikely that one can use equations 2 and 3 to design or predict the performance of WSP systems. In addition, Figure 1 shows that the BOD removal is independent of pond hydraulic detention time, which would contradict Equation 2.

WSP effluent BOD consists of both particulate and soluble fractions. The reported WSP effluent BOD values do not provide an indication of the magnitude of the soluble or particulate component. The particulate component will have BOD contributions from algal cells, which would be high during warmer months. One reason why the present study and others (Finney and Middlebrooks, 1980) have found poor correlation between pond performance in terms of BOD removal and traditional design parameters and design equations is the variable presence of particulate BOD in the effluent samples. If soluble BOD was measured and design criteria were based on soluble BOD, perhaps WSP performance would be more predictable.

The suspended solids escaping in the WSP effluent do cause ecological problems in streams having low flows (King, et al., 1970). Economical suspended solids removal techniques should be developed to prevent water quality problems in the streams. Land application of WSP effluents (Withrow and Bledsoe, 1980), or use of an aquatic marsh treatment system (Bastian, 1982) may provide economical methods to take care of this problem.

In summary, a close evaluation of twenty WSP systems in Missouri and Kansas showed that many of these systems were not meeting the state effluent BOD and SS requirements (even though in some cases these were the relaxed effluent standards). The WSP performance in terms of BOD and SS removals did not correlate well with the traditional design parameters, such as BOD loading rate (kg/ha.d) or mean hydraulic detention time. The use of multiple cells did not always mean better effluent quality. Gloyna's equation for determining the pond volume for a 30-90% BOD removal efficiency was not very accurate in predicting the pond performance. Usually, it gave pond volumes much higher than calculated by other empirical methods. Use of a plug flow equation for pond performance evaluation may not also be quite suitable since the calculated BOD removal coefficient was found to be quite variable. In order to comply with the state effluent quality requirements, it is necessary to develop some economical methods for the removal of suspended solids (floating algae cells) and BOD during the critical summer months.

ACKNOWLEDGEMENT

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WASTEWATER LAGOONS IN A COLD CLIMATE

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ABSTRACT

Severe climate, intermittent rivers and availability of land make facultative lagoon systems the method of choice in treating primarily domestic sewage from smaller municipalities. The lagoons are designed on a recommended maximum load of 55 $kgBOD_5/ha\ d$ to first cell, while the second cell provides storage. The discharge is twice annually and the occurrence of the spring ice-break up odor period is one of the primary criteria limiting this load. Based on full scale performance data, it is demonstrated that, from the standpoint of odor nuisance, the load to the first cell should be kept equal to or less than 35 kg/ha d. Full scale studies of an overloaded lagoon systems show the futility of under-ice aeration for odor control. Mechanism of natural odor control during ice break up is elucidated. Upgrading of the overloaded systems or lagoons receiving significant industrial contribution is best achieved by construction of a 3-5 m deep aerated lagoon preceding the two or more facultative cells in series.

KEYWORDS

Facultative ponds, aerated lagoons, low temperatures, performance, odors, pond design, organic load.

INTRODUCTION AND OBJECTIVES

Located between the 49th and 60th parallel of latitude, Manitoba has a continental climate. The temperature extremes are from -45°C to +40°C, with an average annual temperature of 2.5°C. The frost-free period is approximately 130 days. As land is generally available, lagooning is the single most popular method of waste treatment used in the province. It proved to be the most cost effective method in the agricultural prairie provinces, which currently have 95% - 99.3% of the population serviced by waste treatment. For comparison, the more industrialized provinces of Quebec and Ontario have only 6.2% and 83.5% of population, respectively, serviced by waste treatment facilities (Pearse et al. 1985).

This paper will present: the current lagoon design criteria, results of full scale performance of lagoons serving municipalities with and without industries and results of full scale research on upgrading the performance during the spring ice-break up in an overloaded facility. Although the main emphasis is on facultative (non-aerated) systems, some attention will be directed to the upgrading of overloaded facultative lagoons with a preceding aerated lagoon.

FACULTATIVE LAGOONS

Design Criteria and Operation

The design of facultative lagoons is based on intermittent discharge and treatment in two cells in series, a primary cell and a secondary or storage cell. The maximum permissible organic loading for facultative lagoons is 55 kg BOD_5/ha d. however lower loads are recommended. These loads are the basis of design for the first or primary cell. Hydraulic loading is not considered in the design of the primary cell. The maximum water depth in all facultative cells is 1.5 metres, and is based on a balance between sunlight penetration and the effect of ice formation. Overall hydraulic retention time is based on two factors. Firstly, many of the lagoons are located in areas with intermittent streams, often providing no dilution for discharges and, more importantly, winter discharges freeze in the dry water courses, creating ice dams which will float on the spring run-off and damage the water courses and bridges during the spring flooding. Secondly, there is very little stabilization of the organic wastes under ice cover and they must be stored until they have stabilized in the spring. In combining these two factors, no discharge is permitted from November 30 of any year until May 15 the following spring (196 days). Storage is provided in subsequent cells, however one half of the volume of the primary cell may be considered for storage capacity. The precipitation and evaporation are ignored as they generally balance. The lagoon is required to be impervious, and unless liners are installed, the soil must be compacted to have a hydraulic conductivity of less than 10^{-7} cm/s. Facultative lagoons, depending on the receiving waters, are discharged once or twice a year. The procedure (which may last up to two weeks) involves closing the valve between primary and secondary cell, sampling and analyses of BOD_5 and Coliform MPN and then discharging the secondary cell if discharge permit limitations are met. This is followed by opening the interconnecting valve, transferring the waste from the primary cell and discharging from the secondary cell again, after the primary cell is isolated.

Winter Performance of Facultative Lagoons

Winter performance is determined by the thickness of ice cover, existence or lack of snow cover, organic load, and influent wastewater temperature. The most significant concern in the application of the facultative lagoons is the probability of odor nuisance during the spring ice break-up. The presence of snow cover on ice eliminates the production of photosynthetic oxygen. This, coupled with significantly increased concentration of wastewater under ice due to freeze out provides ideal conditions for incomplete anaerobic degradation resulting in the production of odorous volatile fatty acids (VFA) and mercaptans, as well as gaseous hydrogen sulfide (H_2S). Spring ice break-up releases these odorous compounds. The intensity and duration of odor emissions is associated with the overall organic load to the lagoon and weather conditions during breakup. Practical experience has placed a value of 55 kg BOD/ha d as the maximum load to the first cell, above which the odor emissions are excessive. As will be shown later design load of 35 kg/ha d is recommended as it does not result in significant odor nuisance during the ice break-up.

Low intensity aeration. An experiment with aeration of full scale overloaded lagoons to maintain and control odors will be described. The 1.5 m deep lagoon system consisted of two parallel lagoon trains: 1. cell 1 followed by cell 3 - with aeration. 2. cells 2 and 4 - without supplemental aeration. The aeration was by a 15 kW blower supplying 5 Nm^3air/kg BOD_5 introduced. Both trains were overloaded, receiving an average of 381 mg/L BOD_5, partly due to large proportion of the town's wastewater originating in a vegetable processing industry. The primary cells in the two trains worked under a load of 128 kg BOD_5/ha d (cell 1) and 98 kg/ha d, (Cell 2). Fig. 1 illustrates the performance of the aerated train, series of cell 1 and 3. The bar NO AIR indicates the period when the aeration system broke down. Fig. 2 illustrates the non-aerated train, cells 2 and 4. The non aerated train had an overall load to both cells of 56 kg BOD_5/ha d, while the aerated train was loaded at 78 kg/ha d. The difference is manifested by a more rapid BOD_5 concentration drop in cells 2 and 4 after April 1, following ice break-up. The maximum freeze-out concentration increase in cell 1 was much higher and lasted longer than in cell 2. The secondary cells exhibited similar difference in the under-ice behavior. Figure 3 illustrates the total sulfides concentrations in the liquid in the two parallel secondary cells 3 and 4. The sulfides concentration in the liquid is less in the aerated cell 3 than in the non-aerated cell 4. This was a result of venting the sulfides through holes created in the ice by the escaping air. Site visits, however, did not indicate any improvement in odor intensity due to aeration. The aeration of the cell 1-3 system was found practically ineffective at the applied rate of 5 N m^3/kg BOD introduced. The problem was aggravated by thicker than usual ice formation (70-100 cm in 150 cm deep lagoon) and significantly increased BOD_5 concentration due to freeze-out. In summary, the train of cell 1 and 3 was found to perform comparably to cells 2 and 4.

Ice and spring odor relationship. Another full scale experiment was performed on two parallel lagoons without any influent, acting only as winter storage. The objective was to determine if the lagoons would freeze completely and what is the mechanism of spring odor control in low-loaded lagoon. The lagoons A and B, were 1.5 m deep and each had a volume of 1000 m^3. They were filled on Jan. 3 (A) and Jan. 5 (B). The ice cover developed very rapidly, and the anaerobic degradation started depressing the pH and generating gaseous hydrogen sulfide. The lagoons did not freeze to the bottom.

The evolution of hydrogen sulfide is primarily controlled by pH. The equilibrium equation indicates that at pH = 6, almost 100% of sulfides are in gaseous form, and will cause odors, while at pH > 9.0, most of reduced sulfides will be in the HS^- and $S^=$ forms, which are not volatilized easily. Anaerobiosis was the result of increased BOD due to freeze out and onset of acidogenic fermentation, coupled with sulfate reduction to sulfides. Spring thaw began in March by developing a layer of water on top of ice. This layer was well aerated by natural oxygen transfer, and had very low BOD. Ice core samples melted in the laboratory, demonstrated a BOD_5 concentration of less than 10 mg/L and low content of total solids. This concentration increased only insignificantly toward the bottom of the ice cover. Thus, melting is a natural well aerated barrier to odor emissions. Early spring algal blooms both above and often under the ice, add a significant quantity of oxygen, speeding up the removal processes. The occurrence of spring odor period must then be the result of organic overloading, where an excessive amount of odorous organics (volatile fatty acids, mercaptans) overtaxes the oxidation capacity of the melting water layer.

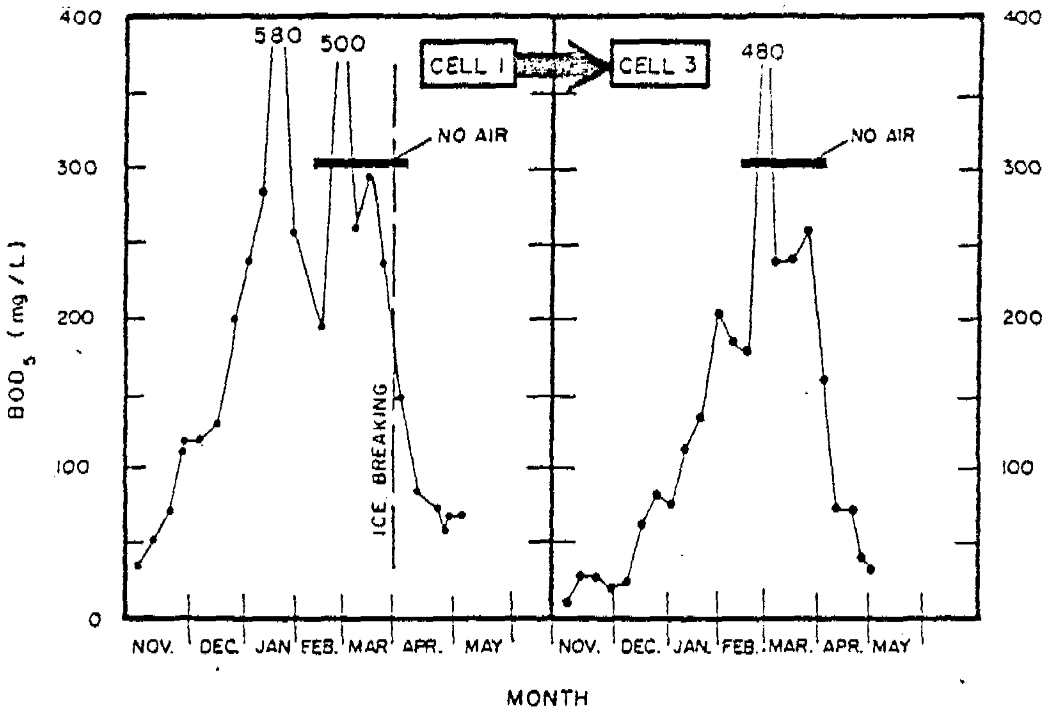

Fig. 1. BOD concentrations in two facultative lagoons in series, aerated in winter.

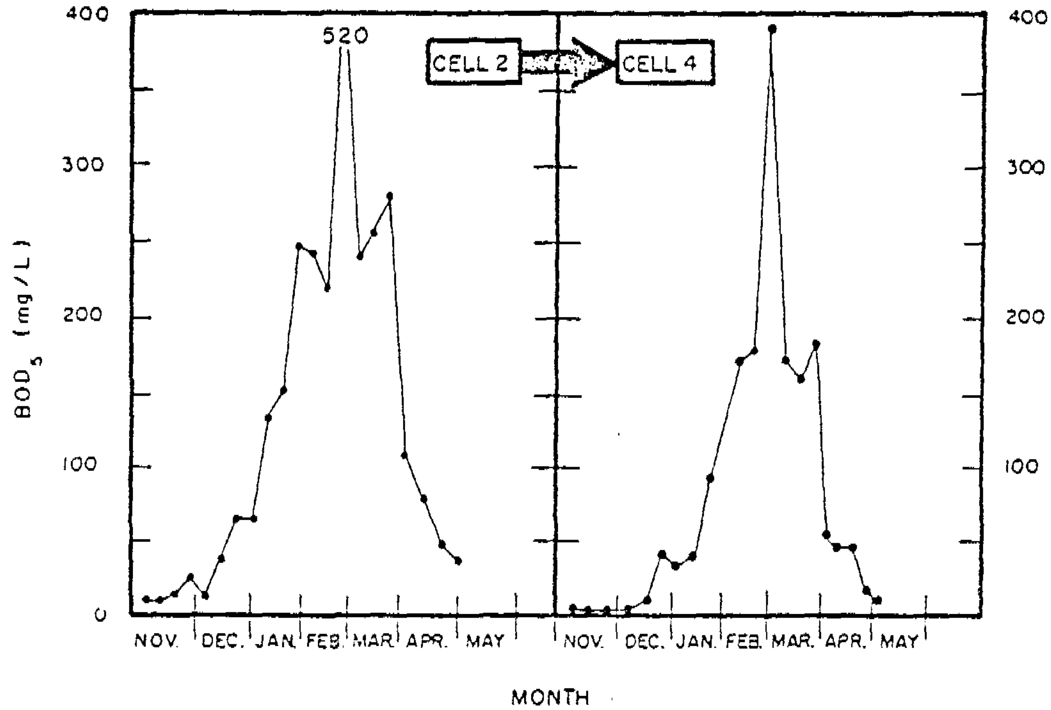

Fig. 2. BOD concentrations in two facultative lagoons in series. No aeration.

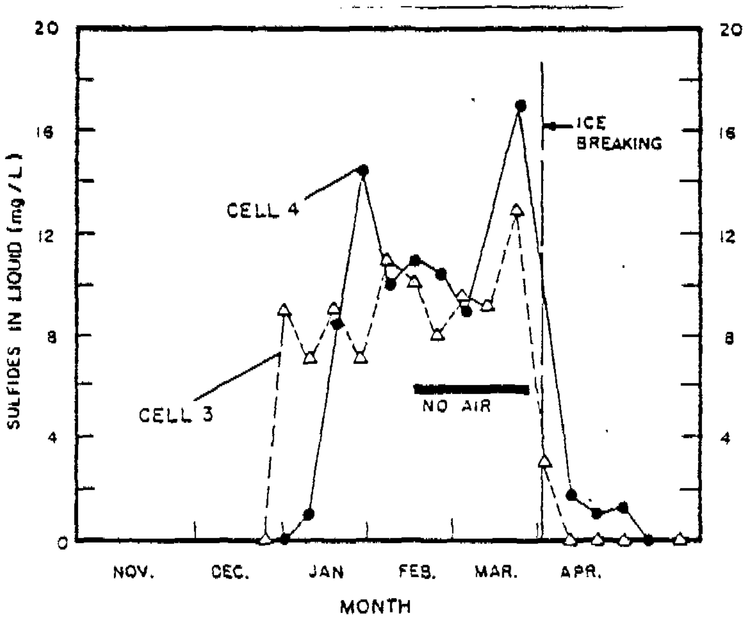

Fig. 3. Sulfides in liquid in secondary cells of the two parallel trains.

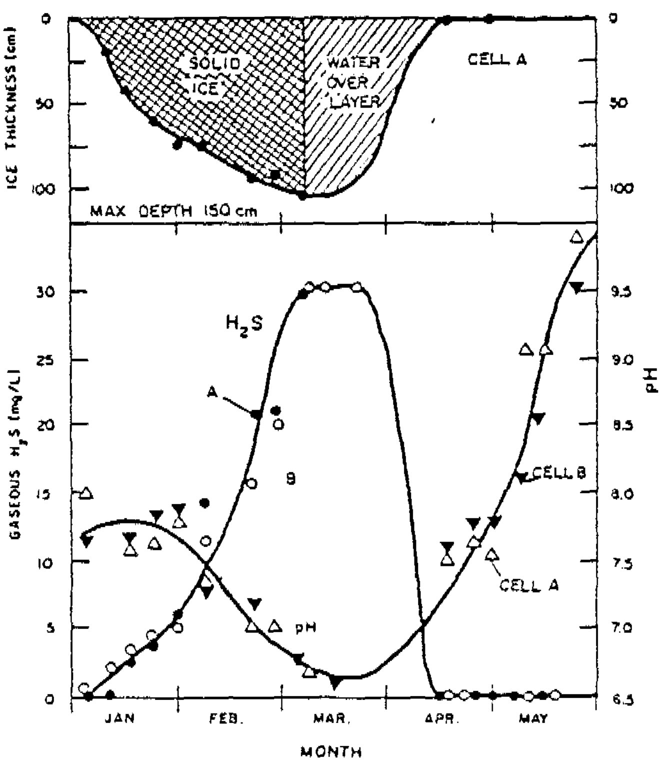

Fig. 4. Effects of ice formation on the spring odor period.

Fig. 4 illustrates the occurrence of gaseous H_2S when the pH is depressed due to acidogenic fermentation under ice. The rapid increase of pH due to algal consumption of CO_2 and generation of oxygen in March and April (when daylight lasts longer than darkness) drive the sulfide concentration rapidly to zero. Change in BOD_5 during the winter season was monitored. Cell B was filled on Jan. 7 with sewage of BOD_5 = 340 mg/L (pH 7.65). The BOD_5 concentration changed as follows: March 20, 1000 mg/L (pH 6.55); April 24, 500 mg/L (pH 7.70); May 1, 110 mg/L (pH 7.55); May 8, 20 mg/L (pH 8.10).

The study showed that holding lagoons do not freeze to the bottom. It was also demonstrated that odors are not emitted at low loads, presumably due to ice melting from the top and the development of an aerobic over-layer. It was found feasible to plan discharge of sewage as early as mid-May.

Full scale performance of facultative lagoons

The two criteria of correct performance are quality of the final effluent and lack of, odors. Since lagoons are discharged intermittently the quality is assured by holding the waste till complete stabilization occurs. The odor issue then becomes the deciding factor in selecting the proper organic load.

Performance of twelve lagoons receiving mainly domestic sewage is presented in Fig. 5 (circles). Due to large distances in the Province, which is roughly 500 km by 1200 km, and infrequent inspections there are no formal threshold odor number (TON) determinations performed on site. The data in Fig. 5 is for towns of 600 to 2500 inhabitants with one town of 12000. Odor perception level, translated to numbers from 1 to 10, have been assigned to these lagoons, based on the following descriptions provided by the town and in most cases corroborated by the visiting inspector: 1 - none reported, 2 - few odor complaints, 4 - infrequent odors, 5 - moderate odors, 8 - severe odors, 10 - very severe odors. The low odor nuisance may sometimes pass unreported by the community as upgrading may mean increased expenditures. Fig. 5 lists also data for municipalities with high proportion (more than 40 to 50% of BOD load) of agricultural processing wastes (triangles). The data appear to be distinctly different from the primarily domestic sewage lagoons. The odors are particularly acute at lagoons loaded with cheese plant effluent (No. 2, 4 and 5) and meat packing (No. 1) effluents. Lagoon system No. 3 has inputs from vegetable processing (80% by load); No. 6 from poultry processing; and No. 7 from vegetable oil manufacturing.

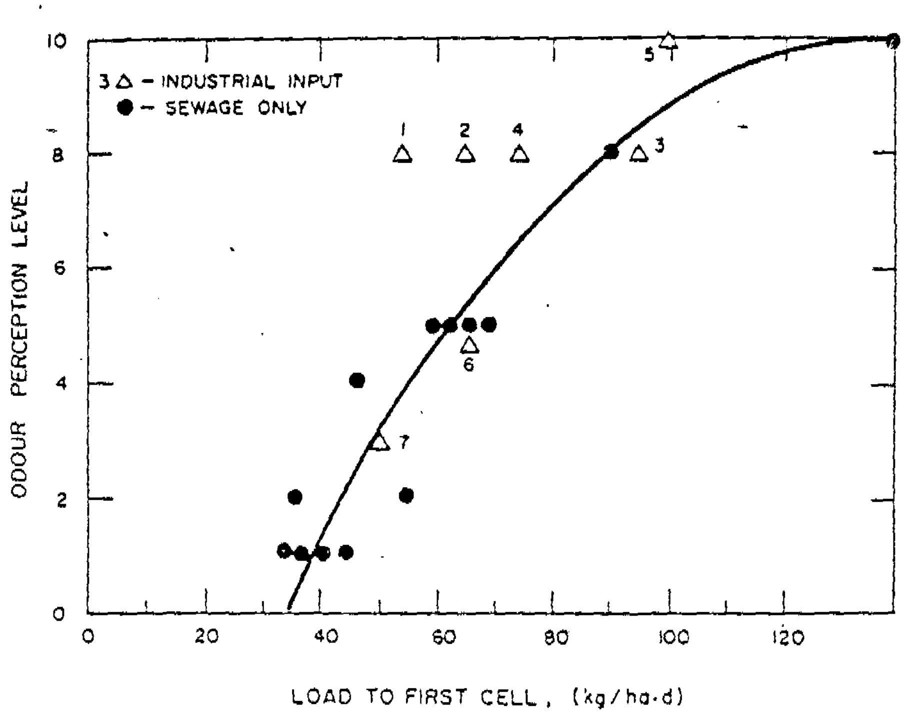

Fig. 5. Relationship between odors during spring ice break up and BOD_5 load to the first cell.

The authors stress the subjective nature of the assigned numerical odor perception levels. The occurances of odor were recorded, without taking into account duration of the spring break-up, prevailing wind direction, nor distance between the lagoon and the municipality.

Based on Fig. 5 it may be recommended that the design loads be kept well below the 55 kg/ha d. The load of 35 kg/ha d to the first cell appears to warrant odor-free operation of lagoons during the spring ice break up. Using the design procedure outlined above, for a 196 days storage volume, the area and volume requirements were calculated for the 55 and 35 kg/ha d first cell loads. The difference in volumes for flows from 100-1000 m^3/d was only 5-6.3%. The lagoons that were overloaded have now been enlarged to lower the load to the first cell. The lagoons receiving significant quantity of industrial wastes have been converted to a system consisting of aerated cell followed by a series of two facultative cells.

AERATED LAGOONS

Facultative lagoons are usually designed for municipalities below 2000 inhabitants, without significant inputs from industry. For larger municipalities costs of land become prohibitive. Municipalities with industries often find that the spring break-up and odor period lasts much longer and is often more severe. Variable discharges from industry are not well-tolerated by the facultative lagoons, and, in these cases, aerated lagoons are often used to pretreat the more concentrated raw wastes and equalize the incoming load.

The usual set-up involves a series of an aerated cell followed by two facultative cells. The design of the aerated cell is based on a conventional first-order removal mechanism, utilizing k rates adjusted for winter temperatures (Bemister, 1977). The facultative cells are designed operated and discharged as described before.

Table 1. Raw waste characteristics (mg/L) and final effluent quality for town A.

Sample	Date	NFR	BOD_5	TOC	TKN	NH_3	P_{tot}	pH
RAW WASTE	1976	560	760	650	60	—	20	7.2
Cell 3	7.V.76	150	17	42	15	—	5	8.7
EFFLUENT	13.V.77	—	19	55	10	—	5	7.8
EFFLUENT	26.V.77	—	56	87	15	—	7	8.5
Cell 2	8.VI.77	—	—	68	10	—	3	9.2
Cell 3	24.VIII.77	12	11	45	12	—	6	8.4
Cell 3	15.V.78	15	12	34	8	—	4	9
EFFLUENT	19.X.78	71	13	65	15	—	8	8.2
Cell 3	5.VI.79	18	10	30	11	10	5	8.6
EFFLUENT	20.VIII.81	16	4	30	11	4	2	8.4
Cell 2	9.V.84	18	26	95	26	23	7	8.1
Cell 3	9.V.84	43	23	100	24	16	5	8.4
Cell 3	22.V.84	15	10	54	7	4	3	9.3
Cell 3	16.VII.84	19	9	56	8	5	5	8.3
Cell 3	17.X.85	15	5	—	3	0	2	8.7

An example of an aerated lagoon-facultative lagoon system is presented in Table 1. The data is based on grab samples except as noted. The lagoons provide an overall volume of 223,500 m^3 and the discharge is twice annually. The aerated lagoon is 3 m deep and has an area of 0.72 ha. The following two facultative cells are 1.5 m deep and have an area 6.88 ha-cell 2 and 7.12 ha-cell 3. During the discharge, both facultative cells are emptied through cell 3, during a period of 2 weeks. The loads to the wastewater treatment plant based on 1976 data were in the order of 523 kg BOD_5/d to 765 kg BOD_5/d, with peak loads in excess of 850 kg/d. Influent wastewater consists of domestic sewage and a vegetable oil processing plant effluent and contains a significant fraction of nonbiodegradable organics, as evidenced by high COD/BOD_5 ratio of 2.8. This ratio is much higher in the effluent, in the range of 5 to 7. The town is required to obtain a permit for each discharge. The data shows significant stability of effluent (or cell

3) BOD_5, which with the exception of one data point remains well below the 30 mg/L requirement. There is considerable nitrification accomplished, some 60 to 90% TKN removal. Phosphorus removal is accomplished at similar levels. Significant algal activity accounts for nutrient uptake in the spring and summer, as indicated by the pH increase.

Review of the aerated lagoon design and performance in the Province has revealed that they operate at range of loads from 0.02 to 0.5 kg BOD_5/m^3d. The usual depth is three meters. Aeration devices include a variety of compressed air devices such as the static tube aerators, and in at least five plants floating surface aerators are used. For larger municipalities in excess of 3000 inhabitants located on flowing streams, the flow-through aerated lagoons are used without storage in the facultative cells.

SUMMARY AND CONCLUSIONS

Due to severe winters and the ephemeric nature of many water courses in Manitoba, lagoons remain the most cost effective method of waste treatment where adequate land is available. Design criteria presently used are a maximum load of 55 kg BOD_5/ha d to the first cell of a facultative lagoon system with 196 d retention time. Experience and data on odor nuisance during the spring ice breakup presented in the paper suggest that loads not exceeding 35 kg/ha d should be used. Maintenance of low loads is the only available safeguard against the odor nuisance as remedial under-the-ice aeration or chemical addition have been shown to be either ineffective or too expensive. The decrease in design load to the first cell will increase the overall volume required by 5-6.5%.

The mechanism of a facultative lagoon spring ice break up was found to work towards elimination of any odors if the load is appropriately selected. Freezeout concentrates contaminants under a thick layer of ice, where the concentration may rise many times that of the incoming raw wastewater. Thawing ice in spring is covered by very well aerated layer of water of very low organics concentration. Vigorous sun action results in first algal blooms in late March or early April, which drive the DO beyond saturation and increase pH, thus making the sulfides insoluble and oxidizing other reduced products of under-the-ice anaerobiosis. At higher BOD_5 loads, equal to or larger than 55 kg/ha d to the first cell, the odors are not contained due to overtaxing of the oxidizing capacity of the system.

The odors are particularly acute in case of lagoons receiving significant inputs of organics from the agricultural industry. The facultative lagoon system was found poorly suited to the handling of variable loads of organics often poorly degradable at low temperatures (e.g. lipids). The majority of these municipalities are now refitted with a highly loaded continuous flow aerated lagoon preceeding the two facultative cells in series. Such system is able to provide an acceptable effluent BOD_5 on an intermittent discharge basis.

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THE DEVELOPMENT OF LAGOONS IN VENEZUELA

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ABSTRACT

The operational experience of early lagoons is outlined. The construction of a new generation of lagoons of 2000 PE to 1.000.000 PE capacity and associated practical difficulties are described. Those with innovative and space saving features treated in some detail. One includes an anaerobic baffled reactor with sludge draw-off facilities which entered service in september 1986 and which is being monitored for compliance with "Engelberg Requirements". Cost information and space requirements for the different systems under construction are included. It is concluded that lagoon systems properly designed and sited are the most appropriate and indeed only financially viable sewage treatment option for Venezuelan circumstances.

KEYWORDS

Wastewater stabilization lagoons; Low cost sewage treatment; Appropriate technology; Anaerobic reactors.

INTRODUCTION

Venezuela, an OPEC country located on the north coast of South America, has a population of 15 million rising at 3,2% per year.

The National Sanitary Works Institute (INOS) is the government body responsible for water supply and sewage collection and treatment in urban areas of over 5000 population; the Ministry of Health being responsible for smaller rural systems.

According to the 1981 census, 83% of the population had a piped water supply in or near their house; 56% had a sewer connection; 15,5% a septic tank; 12,5% a latrine whilst 16% had no sanitary facilities at all.

Of the sewage arising in urban areas, it is estimated that 3% is discharged through long outfalls into the Caribbean and less than 2% of the remainder discharged inland receives any treatment at all. This situation is however, not at all uncommon in Latin America.

Unfortunately, the population is concentrated in the northern part of the country, where water resources have become scarce and some rivers consist almost entirely of raw sewage and industrial waste during the dry season (January to April).

Two years ago, the government set aside funds for the construction of outfall sewers and sewage treatment systems as part of part of a special three year public works programme to ease unemployment. Particularly critical receiving waters which will receive attention include Lake Valencia, Lake Maracaibo and the River Tuy, some of which is taken for water supply to the Capital, Caracas. Another area which is receiving attention is the eastern seaboard and the Island of Margarita, which have become popular with international tourists.

Practical solutions had to be found fast if the oportunity was not to be lost and the politicians lose interest!

EARLY LAGOON SYSTEMS

INOS began lagoon construction in 1969 and by 1973, seven systems had been built to serve populations varying from 10.000 to 25.000. These systems were generally rectangular with a dividing wall down the centre longways, and a cast iron inlet pipe extending 15m inwards from the bank across the water surface. The banks and bottom were usually lined with clay and the water depth ranged from 1 to 1,5 metres.

These lagoons presented one or more of the following problems:

- Vegetation invaded the banks and floor causing smells and vector breeding. Maintenance budgets would not cover the expense of clearing and the lagoon was left to itś own devices.
- Initial sewage flow was insufficient to combat evaporation, the lagoon never filled, dense vegetation took hold increasing losses through evapotranspiration.
- The inlet sewer was badly designed or vulnerable to washout by floods.
- Unplanned urban expansion reached to within 10m of the lagoonś water's edge. The lagoon became overloaded, and smells and insects affected the nearest houses.

When effluent analyses have been carried out, it has generally been found that in terms of BOD, the lagoons perform well when not overloaded and particularly well when full of emergent vegetation. In bacterial terms however they fail due to lack of light penetration. The vegetation must be removed in any case for odour control and anti-malarial reasons, a job which must be done twice a year to acheive any measure of control.

The early lagoons had 0,5 mts. freeboard which allows the embankment's capilliary zone to rise to the surface allowing a prolific growth of weeds around the perimeter. The maintenance of these lagoons was very costly particularly the operation of dragging out the emergent vegetation from the bottom.

RECENT LAGOON CONSTRUCTION

Effluent standards were promulgated for certain areas in 1978 and those for the whole country in 1985. This has meant that sewage treatment has received attention only recently, despite many years of lobbying by conservationists.

A large number of industrial waste treatment plants have been built along with many so called 'Compact' extended aeration type plants to serve isolated housing estates and apartment blocks. Few of these operate satisfactorily for more than a few months after the last property has been sold.

The author has been searching for more appropriate solutions for sewage disposal in Venezuela since 1974, and has been involved in the design and construction of the municipal systems listed in table 1.

TABLE 1 STATUS OF MUNICIPAL SEWAGE TREATMENT SYSTEMS

TOWN	DESIGN POP. STAGE 1	TYPE	YEAR DESIGNED	YEAR PUT IN SERVICE
BOCA DE RIO	4.500	AL	1979	1980
LOS MARITES	15.000	AL	1979	1981
" "	40.000	AS		1987 (Upgraded)
PORLAMAR	200.000	AS	1981	1987
MATURIN	225.000	ANL	1983	1987
VALENCIA (EAST)	750.000	ANL	1983	1987
MOCHIMA	1.000	ANL	1982	1983
EL TIGRE	60.000	FL	1982	1986
PARIAGUAN	10.000	ANL	1984	1986
CUMANA	100.000	AS	1984	1987
MARACAIBO (SOUTH)	1.000.000	ANL	1986	1987
JUANGRIEGO	50.000	AS	1985	1987

AS: Simplified activated sludge; AL:Aerated lagoon; ANL: Anaerobic- Facultative- Maturation L.;FL:Facultative- Mat. L.

All the systems have been planned in stages with an initial capacity of no more than twice or three times the existing gauged flow in order to avoid the 'no-fill' situation mentioned earlier, and also to reduce initial investment. Almost without exception, considerable capital cost was involved in getting the flow together and then to the treatment site. In some cases the cost of the outfall sewers was considerably more than the cost of the first stage of the treatment system.

It was found necessary to site the lagoon systems further away from existing urban ares than would have been the case with a conventional plant, in order to find suficient land at a reasonable price. However the cost of the extra sewer length was always compensated for by the reduction in capital and operational costs of the lagoon alternative. Existing INOS norms tend to give outfall sewers an uneconomic amount of spare capacity. Previous projects were reviewed and capacity provided in stages according to demand.

The interior banks of all the new lagoon systems have a concrete lining 5-7cm thick with continuous mesh reinforcement. Linings with expansion joints have not been satisfactory as vegetation sprouted from them requiring constant attention. Asphalt linings broke up at the water line due to wave action and vegetation again has taken hold.

The cost of the lagoon systems built recently, including land aquisition varies from US $ 12 per Population Equivalent (PE) for systems of around 30.000PE to $ 5 for the very large systems.

SYSTEMS WITH ANAEROBIC REACTORS

Several of the lagoon systems are in situations requiring economy of space and in these cases, anaerobic reactors have been incorporated,saving about 40% of the space requirements of a facultative- maturation series. Because effluent will be used mainly for irrigation, sizing of the systems has generally been to the recommendations of World Bank Technical Paper No. 7 (Arthur,1983), the "Engelberg Report" (IRCWD,1985) and World Bank Technical Paper No.51 (Shuval et al,1986).

Two such systems, albeit small ones are now in operation and two very large ones are under construction. The Mochima system, shown in Fig. 1 was designed for 1000 people in a small fishing village at the head of an enclosed bay which forms part of a National Park.

The anaerobic reactor consists of two baffled concrete tanks in parallel with a roof consisting of aluminium mosquito netting stretched over wooden frames. The tanks have hopper bottoms with valved sludge drawoff pipes discharging to an adjacent drying bed.

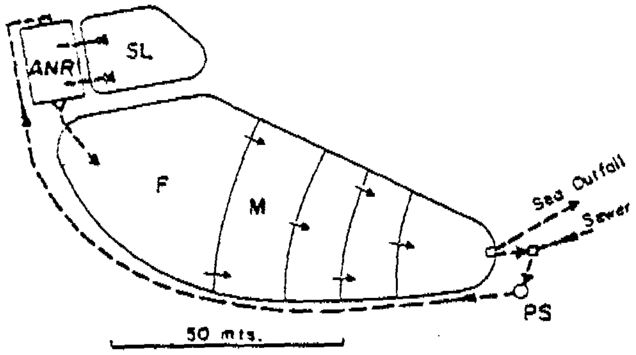

FIG. 1 MOCHIMA SYSTEM

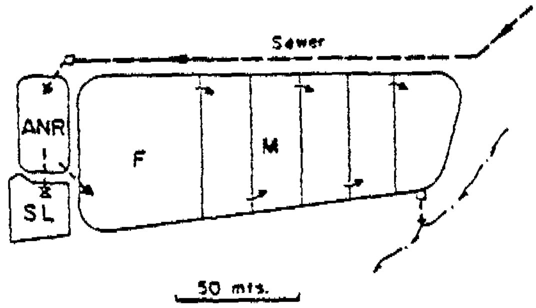

FIG. 2 PARIAGUAN SYSTEM

ANR. Anaerobic Reactor., SL. Sludge Lagoon, FL. Facultative Cell, M. Maturation Cells

The final effluent weir from the lagoon has two 'V' notches, one at 90^{o} for discharge to a sea outfall and the other at 53^{o}-08' for recirculating 50% flow to the inlet sewer screening chamber. This has allowed the formation of a green photosynthetic layer on the surface of the anaerobic reactor, reducing odours to a minimum.

The small maturation cells encouraged mosquito breeding due to lack of wave formation. This was cured by introducing 5000 fish ('Guppies') and the problem disappeared within 48 hours.

Unfortunately no monitoring of performance has been carried out, but the local community are very pleased with their lagoon and the nuisance conditions which previously existed along the town's shoreline have disappeared entirely.

The Pariaguán system, shown in Fig. 2, was designed for 10.000 in a small provincial town. The inlet sewer discharges to a control structure with a flume for restriction of flow to up to 3X design average daily flow, the excess overflowing to the receiving watercourse. This was necessary due to the number of older houses with roof and internal patio drainage connected to the sewer.

The primary treatment stage in this case consists of an anaerobic baffled reactor with 5 compartments in series with more or less upward flow, the final one being for settlement and is provided with a hopper bottom and valved sludge draw-off pipe to an adjacent drying bed.

The anaerobic reactor has a total design retention time of 0,7 days and the six facultative and maturation cells, a total time of 8,5 days at the design average flow of 23 L/s. The system started filling in October 1986 and by the end of January the faecal coli count was below 1000/ 100ml and detergent foam, initially an embarrassment in the effluent channel, disappeared.

Microscopic examination of influent and effluent for compliance with the 'Engelberg Requirements' ("No more than one viable intestinal nematode egg per litre, nor 1000 Faecal coli per 100ml as a geom. mean") has been undertaken from the start. Whilst eggs and cysts of Taenia Sagginata, Ascaris Lumbricoides Ancylostoma and amoeboe have been found routinely in the influent, none has been found in any of the 25 effluent samples examined to date.

Fly breeding on the scum of the first cell of the anaerobic reactor was initially controlled by sprinkling lime. Later, recirculation with a portable pump was carried out for three weeks in December. The scum has since turned green and operation has stabilized; the original reduction of 0,3 of a pH unit between inlet and outlet has disappeared and the gas rising to the surface has no apreciable odour.

DISCUSSION

Two very large systems are under construction in Maracaibo and Valencia (see Figs. 3 and 4), both designed along the lines of the anaerobic systems describ-ed. In addition, they will incorporate effluent recycle for odour and pH control, and sludge recycle in order to inoculate the influent with appropriate anaerobic biomass in a similar manner to activated sludge. In both cases effluent will be used for irrigation for nutrient removal as well as to conser-ve dwindling water resources.

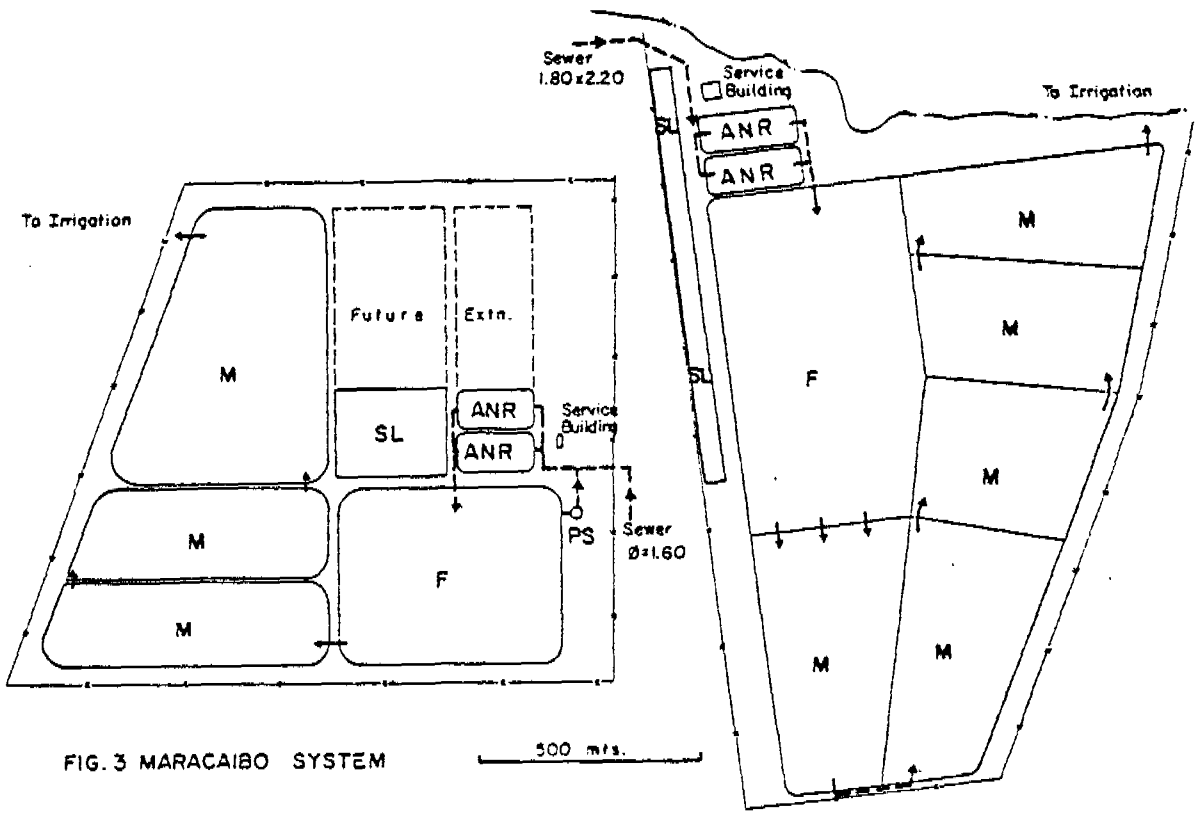

FIG. 3 MARACAIBO SYSTEM

FIG. 4 VALENCIA SYSTEM

ANR. Anaerobic Reactor , SL. Sludge Lagoon., FL. Facultative Cell., M. Maturation Cells

It is realized that the scale up is considerable but the coincidence of the availability of funds and enthusiasm on the part of politicians could not be missed. An economic but robust solution had to be found quickly and despite considerable pressure from salesmen of foreign equipment, schemes based on lagoons were adopted with the anaerobic modifications described above for economy of space. The Maracaibo system for 1.000.000 PE for example will occupy a 130 Ha. site which includes a 50m buffer strip around the periphery.

It has been found that the capital cost of 'conventional' systems is from 4 to 6 times as much as lagoon systems and require expensive electrical energy and maintenance by scarce technicians who are in great demand in other industr-ies.

CONCLUSIONS

Lagoon systems have been found capable of producing effluents complying with 'Engelberg' requirements for use in irrigation at minimum capital and operational costs, without the use of imported mechanical equipment.

The anaerobic systems built so far generate less odour than feared and econ--omise considerably on space. For public relations purposes however, they should be sited as far as feasible from urban areas, where land values are lower at the expense of additional sewer length. At today's rates of interest, outfall sewer capacity may be most economically provided in stages according to the evolution of the demand.

In hot countries such as Venezuela, lagoon systems combined where possible with effluent irrigation, provide really the only technically viable and financially defensible alternative for sewage treatment as they make most use of the available heat and solar energy for digestion, photosynthesis and ultraviolet disinfection.

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THE PERFORMANCE OF A SERIES OF FIVE DEEP WASTE STABILIZATION PONDS IN NORTHEAST BRAZIL

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ABSTRACT

The performance was studied of a series of 2.2 m deep anaerobic, facultative and maturation ponds, which had an overall retention time of 25 days, and a mean mid-depth temperature of 25°C. The mean percentage removals of BOD, COD, suspended solids and faecal coliforms were 88, 69, 83 and 99.97 respectively; there was only 4 percent reduction of ammonia and 6 percent of total phosphorus. The results are compared with those previously reported for a similar series of 1 - 1.2 m deep ponds at the same site.

KEYWORDS

Waste stabilization ponds; lagooning; deep; performance.

INTRODUCTION

Waste stabilization ponds are an efficient means of wastewater treatment in many parts of the world wherever suitable land is available at reasonable cost. Their many advantages include low costs (both capital and operation and maintenance), a very high degree of pathogen removal, simple maintenance, and no requirement for external energy (other than solar energy). They do require, however, significantly more land than more other wastewater treatment processes, and this is often a limiting factor in the adoption of pond technology, especially in large metropolitan areas. The usually recommended depth range for facultative and maturation ponds is 0.9 - 1.5 m (Marais and Shaw, 1961); but clearly if pond depths could be increased, and pond performance maintained (or increased), then less land would be required and ponds could become more widely used for large populations (> 100,000 people). In this paper we present the results of our research into the performance of a series of 2.2 m deep pilot-scale ponds in northeast Brazil; this extends our earlier work on deep ponds which concentrated more on the removal of excreted bacteria and viruses (Oragui et al., 1987).

EXPERIMENTAL METHODS

Pilot-scale ponds

A series of five pilot-scale ponds (Figure 1) was built at the Federal University of Paraíba's wastewater treatment experimental station EXTRABES (Estação Experimental de Tratamentos Biológicos de Esgotos Sanitários) in Campina Grande in northeast Brazil (latitude 7°13'11" south, longitude 35°52'31" west; altitude 550 m above m.s.l.). The series comprised an anaerobic pond (coded A7), a secondary facultative pond (F9) and three maturation ponds (M7, M8 and M9), and its overall mean hydraulic retention time (= volume/flow) was 25 d; the dimensions (m) of each pond were 10.00 x 3.35 x 2.20. Raw municipal wastewater was pumped

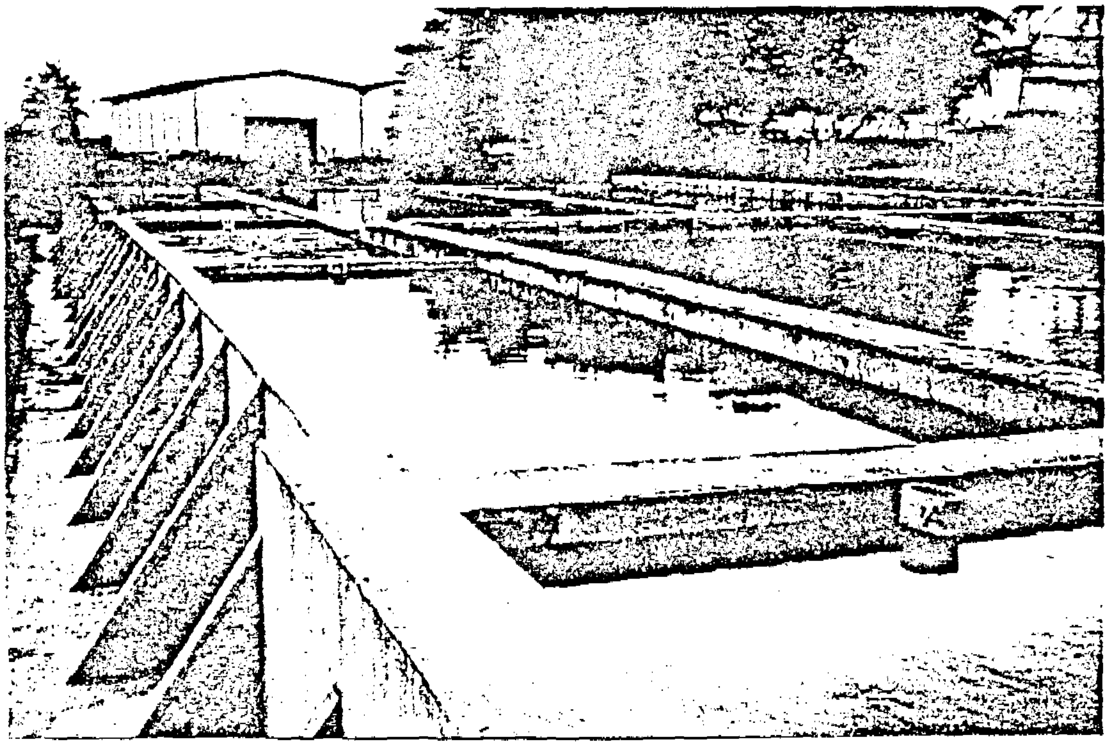

Fig. 1. The series of five deep pilot-scale waste stabilization ponds at EXTRABES

from a trunk sewer running adjacent to the site into pond A7 at a constant rate of 614 l/h using a Watson Marlow (Falmouth, England) model HRSV variable speed peristaltic pump. Pumping commenced on 4 June 1985.

Sampling and analysis

Sampling commenced at the beginning of August 1985, and essentially followed the procedures detailed in Silva (1982). These involved the preparation of weekly composite samples, which were analysed according to APHA (1980) for BOD_5, COD, suspended solids, ammoniacal nitrogen and total phosphorus. Grab samples, taken twice weekly at 0800 h, were analysed for faecal coliforms (Oragui et al., 1987), intenstinal nematode eggs (Mara and Silva, 1986), chlorophyll a (Pearson, Mara and Bartone, 1987), sulphide and pH (APHA, 1980). Temperature was recorded as the monthly mean of the daily mean temperature determined by suspending a maximum-and-minimum thermometer in the raw wastewater flow and at mid-depth in each pond.

EXPERIMENTAL RESULTS AND DISCUSSION

Throughout the twelve-month sampling period August 1985 - July 1986 the mean pond temperatures averaged 25.0°C within the narrow range 23.6 - 26.3°C. The mean physicochemical and microbiological results are presented in Tables 1 and 2 respectively. The anaerobic pond A7 removed only 46 percent of the BOD and COD; this was much less than previously reported for anaerobic ponds at EXTRABES (68 - 83 percent: Silva, 1982; Mara, Pearson and Silva, 1983; Oragui et al., 1987), and probably reflects the much lower strength of the wastewater during this sampling period (128 mg BOD_5 l^{-1} compared with 215 - 289 obtained previously), which resulted in a corresponding low volumetric loading of only 26 g BOD_5 m^{-3} d^{-1} (compared with 35 - 300 in previous experiments). However the final effluent quality, in terms of BOD_5/SS was very good (16/32). The removal of nutrients (N and P), on the other hand, was essentially zero; in comparison the series of five shallow (1.0 - 1.2 m deep) ponds at EXTRABES (A1, F1, M1, M2 and M3) described by Silva (1982) effected removals of 32 - 81 and 36 - 54 percent for N (ammonia) and P respectively when operated at overall retention times of 8.5 - 29.1 d.

The removal of intestinal nematode eggs was lower than previously reported for ponds at

TABLE 1 Mean concentrations, and percentage removals, of physicochemical parameters in raw wastewater (RW) and pond effluents (A7 - M9)

Parameter	RW	A7	F9	M7	M8	M9	Percentage removal
BOD_5, mg l^{-1}	128	69	45	28	20	16	88
COD, mg l^{-1}	357	193	162	137	111	109	69
Suspended solids, mg l^{-1}	184	44	41	46	33	32	83
Ammonia, mg N l^{-1}	27	28	28	27	26	26	4
Total phosphorus, mg P l^{-1}	4.1	3.8	4.0	4.1	3.9	3.8	6
Sulphide, mg S l^{-1}	2.0	7.8	6.8	4.6	3.0	1.6	-
Chlorophyll a, μg l^{-1}	-	-	94	170	112	233	-
pH	7.4	7.2	7.3	7.4	7.6	7.6	-
Temperature, °C	26.5	25.2	24.9	25.0	24.9	25.0	-

TABLE 2 Mean bacterial and parasite egg numbers*, and percentage removals, in raw wastewater (RW) and pond effluents (A7 - M9)

Organism	RW	A7	F9	M7	M8	M9	Percentage removal
Faecal coliforms	$1.2x10^7$	$1.8x10^6$	$4.8x10^5$	$6.0x10^4$	$2.5x10^4$	$4.0x10^3$	99.97
Ascaris	332	63	13	2	3	0	100
Trichuris	7	0	0	0	0	0	100
Hookworms**	992	108	58	57	53	53	94

* Bacterial numbers per 100 ml (geometric mean), parasite egg numbers per litre (arithmetic mean).

** Hookworm numbers include both eggs and larvae.

EXTRABES (Mara and Silva, 1986), and this indicates the need for a more thorough investigation into the relationship between egg removal and pond geometry and hydraulics, especially in view of the recent "Engelberg" guideline (IRCWD, 1985) for no more than 1 viable egg per litre in treated wastewaters that are to be used for crop irrigation.

The removal of faecal coliforms varied from 64 to 86 percent in each pond, with an overall reduction of 99.97 percent, which is the same as that reported by Oragui et al. (1987) for a series of five 3 m deep ponds at EXTRABES (A6, F8, M4, M5 and M6) which had an overall retention time of 21 d. This removal was similar to that achieved in the series of five shallow ponds (A1 - M3) (Silva, 1982) operated at an overall retention time of 17 d (99.96 percent), but less than that achieved by A1 - M3 when operated at an overall retention time 29.1 d (99.99993 percent, and 99.999 percent after 23.3 d). Interestingly, in all these series (A1 - M3, A6 - M6 and A7 - M9) the percentage removal of faecal coliforms in the anaerobic pond was as high as, or higher than, that in some (but not all) of the facultative and maturation ponds; this, together with the other data herein, highlights the need for further research into faecal coliform removal in ponds, in particular into the mechanisms and relative efficiencies of removal in anaerobic and photosynthetic ponds.

CONCLUDING REMARKS

Whilst it is true that the series of shallow ponds A1 - M3 reported by Silva (1982) were able to produce an effluent with up to two orders of magnitude fewer faecal coliforms than the deep series A6 - M6 and A7 - M9 at more or less the same retention time, they would require 2 - 3 times as much land to treat the same wastewater flow. Thus there is a trade-off to be made at the design stage beween pond performance and pond land requirements. The better conservation of nutrients (N and P) in the deep series A7 - M9 is an additional point of importance when the effluent is to be used for crop irrigation.

ACKNOWLEDGEMENTS

We wish to express our gratitude to the following organizations for the provision of research facilities at EXTRABES: the Universidade Federal da Paraíba, the Companhia de Águas e Esgotos da Paraíba, the Superintendência do Desenvolvimento do Nordeste, the Financiadora de Estudos e Projetos and the Fundo de Incentivo à Pesquisa Tecnico-Científica of the Banco do Brasil S.A.

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THEME 2

INDUSTRIAL WASTES AND TERTIARY TREATMENT

POND TREATMENT OF RETTERY WASTEWATERS

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ABSTRACT

Rettery wastewaters were subject to anaerobic and aerobic treatment. Anaerobic treatment yielded efficiencies of BOD_5 and COD removal as low as 20%. The treatment process conducted under aerobic conditions in aerated and stabilizing ponds arranged in series took from 18 to 20 days and gave efficiencies of BOD_5 and COD removal amounting to 90%. The experimental results were interpreted by virtue of the Eckenfelder equation. Excess activated sludge was subject to aerobic stabilization in a separated tank. A new technology was suggested for the existing obsolete industrial treatment plant.

KEYWORDS

Rettery wastewaters, anaerobic treatment, activated sludge pond treatment.

INTRODUCTION

Flax and hemp are homegrowing textille raw materials. The total annual production of the 35 rettery plants operated in Poland amounts to 30100 Mg. The rettery technology involves biological and thermal processes. The volume of wastewaters discharged, predominantly soak and wring effluents, approach 26.50 m^3/Mg flax and hemp processed.

Raw wastewater contains organic matter lixiviated from stalks. The influent concentrations of pollutants are as follows: COD, 2500 to 4600 g O_2 m^{-3}, BOD_5, 1100 to 2800 g O_2 m^{-3}, total nitrogen, 58 to 129 g $N \cdot m^{-3}$, and phosphates, 3 to 90 g PO_4 m^{-3} / Bartoszewski K., et.all 1982/. While the chemical composition of rettery effluents suggests agricultural uses, the relatively long winter season in Poland fails to encourage on developing such methods of utilization. It is conventional to treat rettery wastewater by lagooning or, sometimes, by the activated sludge process. / Mainck et.all 1968, Benefield at.all 1980/

In recent times, the increased demands made on industrial effluents have raised serious problems in many wastewater treatment plants, specifically in the obsolete ones. This holds primarily for treatment plants involving anaerobic processes, e.g. stabilization or even facultative ponds.

A radical change in the treatment technology by substituting anaerobic for aerobic processes will solve the problem. The aerated pond method gives fair promise to be simple, inexpensive and energy saving.

Treatment by lagooning

The daily volume of wastewater discharged by the rettery plant under study ranges between 300 and 350 m^3. Although the concentration of pollutants is high, it does not differ from a typical rettery effluent. The waste stream is passed to an obsolete industrial treatment plant which operates seven in series connected ponds. They have a depth of 1.2 m each and a total volume of 7000 m^3. Retention time amounts to about 27 days, and is too short to cope with the high concentration of organic substances present in the wastewater. The anaerobic conditions existing in the ponds are responsible for the low efficiency of the treatment process. The removal of COD and BOD_5 is equal to, or less than, 20% which is far below the removal efficiency required.

Table 1 Results of anaerobic treatment

Constituents	Unit	Concentration	
		Influent	Effluent
pH	pH	5.2 - 6.3	6.0 - 6.2
BOD_5	g O_2 m^3	1330	1060
COD	g O_2 m^3	2409	1921
Suspended solids	g m^3	273	82
Dissolved solids	g m^3	2098	1982

To upgrade the quality of the effluent it is necessary to apply much longer retention times and pond volumes seven or more than seven times as large as they are. The alternative is to replace the anaerobic treatment method by an aerobic one. This concept was subject to laboratory tests.

Testing method

Laboratory investigations were carried out under through flow conditions for six months. The technological system consisted of two pairs of in series connected aeration and stabilization ponds /System A and System B/. System A involved a long aeration time and a short time of stabilization. In System B, inversely, aeration time was short and stabilization time was long. The technological parameters of the process are shown in Table 2. In System A and System B the wastewater was treated under full aerobic conditions and then sent to the stabilization ponds. Full aerobic conditions were generated in the first pond of System A and in the first pond of System B by medium-size bubble aeration. The total volume of the ponds amounted to 50 dm^3.

Table 2 Technological parameters of rettery wastewater treatment

Technological parameter	Unit	Value	
		Set A	Set B
Retention time for aeration pond	days	19.6	8.43
Retention time for stabilization pond	days	3.34	10.52
Total retention time	days	22.94	18.95
Hydraulic load of aeration pond	m^3/m^2d	0.053	0.117
Hydraulic load of stabilization pond	m^3/m^2d	0.299	0.035
BOD_5 load of aerobic pond	g O_2/m^3d	72.25	161.3

Excess activated sludge was treated in an aerobic stabilization batch reactor of a 50 dm^3 volume. Aeration involved pressed-air. Temperature ranged from 293 to 300 K throughout the tests.

Results

The treatment effects are good when the process involves aerobic conditions. Thus, the efficiency of COD and BOD_5 removal amounted to 90% and 98% respectively. The concentration of suspended solids never exceeded 40 g m^{-3} and 80 g m^{-3} for System A and System B, respectively. The difference in removal efficiency between COD and BOD_5 suggests the contribution of high refractive substance concentrations.

Table 3 Chemical composition of rettery effluent

Constituents	Unit	Raw sewage	Set A Aerobic pond	Set A Stabilization pond	Set B Aerobic pond	Set B Stabilization pond
pH		5.26	8.28	8.10	8.24	8.06
BOD_5	g O_2/m^3	1375	9.5	14.9	13.2	25.6
COD	g O_2/m^3	2395	216	236	259	298
Alkalinity	g $CaCO_3/m^3$	445	440	459	489	511
Total nitrogen	g N/m^3	49.7	10.1	19.4	9.4	17.8
Nitrates	g $N\text{-}NO_3/m^3$	0.0	2.25	0.35	0.55	0.22
Phosphates	g PO_4^{-3}/m^3	55	21.6	30.0	22.6	38.6
Dissolved solids	g/m^3	2247	1181	1171	1245	1240

Humic substances occurred at concentration of 50 to 170 g m^{-3} and inked the effluent with a dark brown colour. Further testing has revealed a high resistace of humic substances to treatment by coagulation and chlorination.

Activated sludge /which settled in the stabilization pond/ was prone to degradation under anaerobic conditions, and the degradation products has an adverse effect on the quality of the effluent. While COD, BOD_5 and concentration persisting in the effluent were only slightly higher, the concentration of total nitrogen were nearly seven times higher that those in the effluent from the aerobic pond. The decreased concentration of nitrates indicates the denitrification has occurred.

Table 4 Efficiency of rettery wastewater treatment by different methods

Element	Unit	Concentration: Natural fertilizers	Concentration: Waste sludges agriculture utilizied	Concentration: Aerobic stabilizied rettery sludge
Nitrogen	% sm	2.0 - 2.3	0.5 - 12.5	36.7 - 38.6
Phosphorus	% sm	0.42- 0.47	0.13 - 1.52	0.80 - 0.95
Potassium	% sm	1.53-2.52	0.08 - 1.1	0.94 - 1.024
Antimonium	µg/g sm	-	5	-
Chromium	µg/g sm	-	200 - 1000	100 - 240
Manganese	µg/g sm	3.5 - 870	500 - 1000	90 - 130
Ferrum	µg/g sm	40 - 2800	120 - 26000	4900 - 10500
Lead	µg/g sm	15	300 - 1000	140 - 170
Cadmium	µg/g	0.8	10 - 50	-
Cupprum	µg/g	1.4 - 62	400 - 1500	200 - 350
Molibdenium	µg/g	0.05 - 5.5	20 - 50	-
Nickle	µg/g	30	100 - 200	70 - 100
Zinc	µg/g	15 - 650	1500 - 3000	600 - 1100

Activated sludge concentration in aerated ponds varied from 300 to 680 g dry wt·m^{-3} in System A and from 580 to 812 g dry wt·m^{-3} in System B.

Aerobic stabilization of excess sludge ran fairly fast. After 16 to 24 days, the efficiency of organic matter removal approached 80 to 90% the initial concentration. Nitrogen and phosphate content in aerobically stabilized rettery sludges was higher than that in natural fertilizers or industrial sludges. Heavy metal concentrations ranged lower than in sewage sludges processed for agricultural uses.

Discussion

The reaction, by which organic matter present in the rettery effluent is degraded, displays a monomolecular nature and runs in a complette mix reator with no recirculation. The reaction rate constant / k / can be calculated by virtue of the Eckenfelder equation.

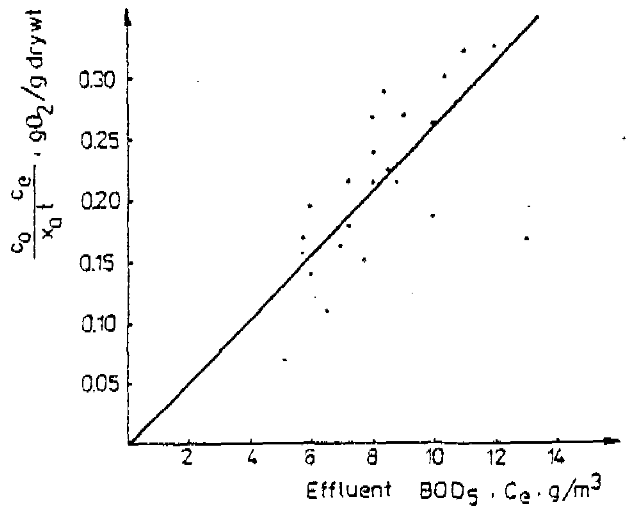

Fig. 1. BOD_5 removal from rettery wastewater according to Eckenfelder's model.

$$\frac{c_o - c_e}{x_a t} = k\ c_e$$

where:

c_o, c_e = influent BOD_5 and effluent BOD_5/without/ SS/, g $O_2 \cdot m^{-3}$

x_a = activated sludge concentration, g dry wt$\cdot m^{-3}$

t = aeration time

The experimental value of k was 0.025 d^{-1} at 303 K, the temperature coefficient Θ amounting to 1.089. Activated sludge growth /calculated in terms of BOD_5 removed/, which is described by the maximum cell yield coefficient / Y_T /, amounted to 0.82 g dry wt$\cdot g^{-1} O_2$. The calculated autooxidation rate coefficient equaled 0.077 d^{-1}.

Organic matter removal by aerobic stabilization of excess sludge followed an exponential pattern. The sludge destruction rate coefficient takes the form

$$k_o = 1/t \ln \frac{x_t}{x_o} = 0.09 ,$$

where x_o, x_t denotes organic matter concentration in the sludge at the beginning of the stabilization process and after time t.

Summarizing comments

The results of laboratory tests and the calculated coefficient of the treatment process made it possible to estimate the technological parameters for the aerobic pond treatment of rettery wastewaters which was conducted in the existing industrial plant.

The technology proposed anticipates that aerobic treatment should be carried out in three in series connected ponds, whereas the stabilization process should be run in the remaining four ponds. The first of the four stabilization ponds should act as a settling tank. As the predicted concentration of biodegradable organics are low, the remaining three stabilizing ponds may act as facultative ponds. The total volume adopted for the aerobic ponds is 2900 m^3, and the optimal retention time approaches 9 days. Sedimentation and stabilization should take 12 days. Calculated oxygen demand and power demand amounts to 455-500 kg $O_2 \cdot d^{-1}$ and $W \cdot m^{-3}$, respectively. Aeration should involve a rotor aerator.

The anticipated average effluent BOD_5 and COD is 20 g O_2 m^{-3} and 275 g O_2 m^{-3}, respectively. The removal efficiency predicted for the winter season is half that for the summer season.

Daily growth of excess sludge is expected to reach 215 kg dry wt$\cdot d^{-1}$. This calls for frequent pumping, to pass the sludge from the settling pond to the separate stabilizing tank. Good aerobic stabilization of the sludge will be achieved after 16 to 24 days of retention. The final operation involves seasoning in the open.

It is worth noting that the tank for sludge stabilization and the fields for sludge seasoning are the only objects to be build in the existing rettery treatment plant.

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TREATMENT OF SLAUGHTERHOUSE WASTE WATERS BY STABILIZATION PONDS

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ABSTRACT

A sampling programme was devised to assess in terms of 5-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD) and suspended solids (SS) removal the performance of one anaerobic, one facultative and two maturation ponds in series, for treating the waste waters resulting from a slaughterhouse killing approximatelly 625 porks/week. The results show that, in spite of poor maintenance which has been causing bank erosion and macrophyte infestation, the system has coped with large variations in flow and organic load, reducing to a minimum the impact of the effluent discharge in the receiving creek.

KEYWORDS

Slaughterhouse waste waters; stabilization ponds; anaerobic ponds; facultative ponds; maturation ponds

INTRODUCTION

A slaughterhouse planning to kill 625 porks/week, equivalent to a gross weight of about 46.10^3 kg, and operating a complete processing system which includes manufacturing, smoking, curing and packing started up in March 1985.

Given the availability of land, the biodegradability of the effluent, its suitable mean temperature of 25ºC and the financial shortages for supporting high capital costs, a set of stabilization ponds consisting of one anaerobic, one facultative and two maturation ponds, in series, has been proposed to treat the plant waste waters.

The lagoons design was based on the predicted water consumption for meat processing activities of 250 l/pork which allowed to estimate a daily waste water volume of 30 m^3. In addition to that, the predicted washing water flow from the pens (5 m^3/d) and the floors (5 m^3/day) and also the waste water from the sanitary facilities (10 m^3/d) totalled a design flow of about

50 m^3/day. The physico-chemical characteristics of the waste water such as BOD_5, COD, and SS were assumed 1200, 2000 and 300 mg/l, respectively.

The dimensions of the lagoons which are presented in Table 1 were obtained from the mid-depth area and calculated following basic design principles.

TABLE 1 Dimensions of the Lagoons

	Anaerobic Pond	Facultative Pond	Maturation Pond
Lenght, m	10	78	33
Width, m	10	26	11
Water depth, m	3.5	1	1
Volume, m^3	350	2000	360
Surface Area, m^2	100	2000	360
Slope	Vert.	1:1	1:1
Clearance, m	0.5	0.5	0.5

The anaerobic pond was designed for a conservative 7-day hydraulic retention time (HRT) and 50% BOD_5 removal assuming temperatures in the range 15-20ºC. Furthermore, the volumetric load was subjected to a check in order to verify that would not exceed 400g BOD_5/m^3 d avoiding the release of objectionable odours (Mara, 1976).

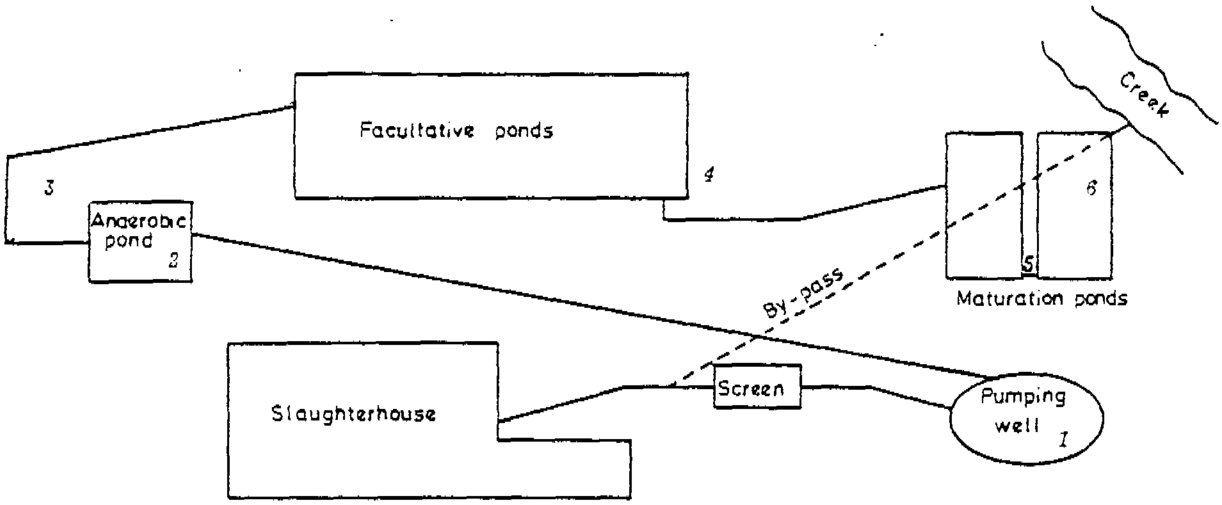
The dimensions of the facultative lagoon were calculated assuming that they behave as completely mixed reactors with a first order kinetics for BOD_5 removal (Marais and Shaw, 1961). The first order rate constant was assumed 0,3 d^{-1} at 20ºC and corrected for the temperature of 15ºC (Mara, 1976). For a 90% BOD_5 removal the calculated HRT was 37.5 days.

A set of two maturation ponds of 7 days HRT each, completes the system for the treatment of the slaughterhouse waste water.

EXPERIMENTAL RESULTS

The operation of the ponds started up in March 1985 and their performance was followed during two periods of time: from March to August 1986 and from January to March 1987. The sampling points are shown in Figure 1, which represents a schematic description of the treatment plant.

The sampling procedure included measurements of pH, temperature, SS, soluble and total COD and dissolved oxygen (DO) twice a week and BOD_5 once a week. While the pork killing was usually on Mondays and Wednesdays, the meat processing was from Monday to Friday.



Sampling points: 1,2,3,4,5,6

Fig. 1 Schematic diagram of the treatment plant

The values for the parameters used for physico-chemical characterization of the raw waste waters from the plant, are presented in Table 2.

TABLE 2 Characterization of the raw waste waters

	Mean value	Median	Std. dev.	Number of samples
BOD_5, mg/l	1308	1235	589	14
COD, mg/l	1905	1635	1256	34
Sol. COD, mg/l	1147	910	852	34
SS, mg/l	395	270	402	33
pH				
Morning	7.5	7.6	0.5	20
Afternoom	7.9	7.8	0.7	19
Temperature, ºC				
Morning	20.0	19.0	4.1	17
Afternoom	22.9	22.7	2.6	16

The results obtained for the sampling points 1, 3, 4 and 6, during the sampling programme are presented in Figure 2 to 5. The characteristics of the final effluent are shown in Table 3.

It worth pointing out that during a significant period of time, as shown in Figures 3 to 5, the treatment plant was out of order due to pump failure. During that period, the raw waste water was discharged straight into a receiving creek.

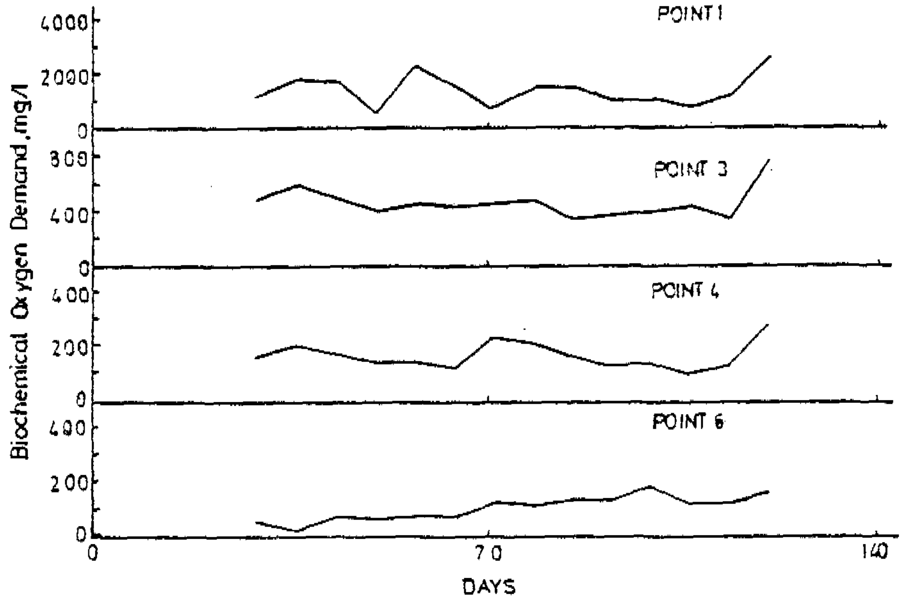


Fig. 2 Observed BOD_5 variation for the different sampling points.

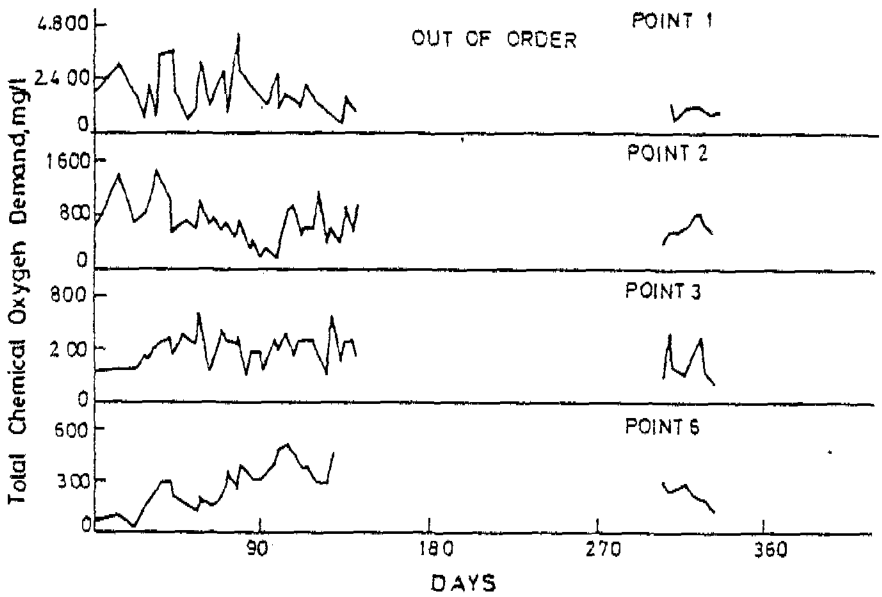


Fig. 3 Observed total COD values for the different sampling points

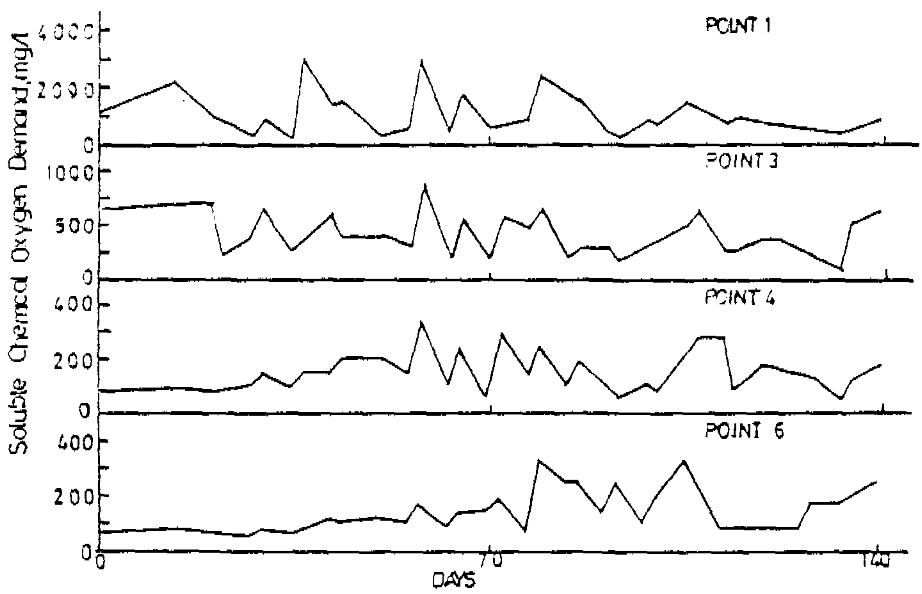


Fig. 4 Observed soluble COD values for the different sampling points

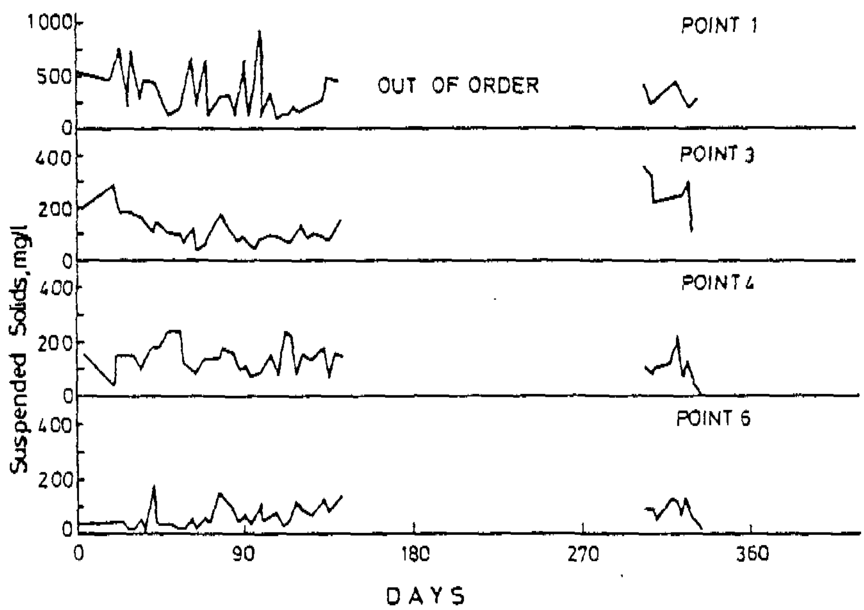


Fig. 5 Observed SS values for the different sampling points

TABLE 3 Characterization of the treated waste waters

	Mean value	Median	Std. dev.	Number of samples
BOD_5, mg/l	101	115	45	14
Sol. BOD_5, mg/l	62,5	55	29	12
COD, mg/l	269	250	150	40
Sol. COD, mg/l	183.5	140	128	34
SS, mg/l	69.7	60	42.5	36
pH				
Morning	8.3	8.1	0.9	19
Afternoom	9.8	9.4	1.3	21
Temperature, ºC				
Morning	18.6	21.0	6.1	16
Afternoom	28.2	28.9	4.7	18
DO, mg/l				
Morning	4.4	3.7	2.9	11
Afternoom	12.7	13.3	4.0	17

The results obtained allowed to assess the performance of the individual treatment units through the estimation of the BOD_5, COD, Soluble COD (SCOD) and SS removal for each pond (anaerobic, facultative and both maturation ponds) and also of the treatment system as a whole. Such results are shown in Table 4.

TABLE 4 Treatment efficiency expressed as percent removal

	Anaerobic pond	Facultative pond	Maturation ponds	Total
BOD_5	66	66	34	92
COD	62	44	30	85
SCOD	57	67	-3	85
SS	62	11	43	81

DISCUSSION AND CONCLUSIONS

The observed performance of the treatment system, in terms of organic matter removal, is in general agreement with the assumed design values. Based on the experimental results presented in Table 2, the volumetric organic load applied to the anaerobic lagoon can be estimated as 185 g BOD_5/m^3 d whereas the surface organic load applied to the facultative lagoon is 0,225 g BOD_5/m^2 d. The above referred volumetric load is bellow the maximum suggested by Mara (1976) for anaerobic lagooning of domestic waste waters and no obnoxious odours were ever noticed in the vicinity of the lagoons.

Regarding the facultative lagoon, and although the BOD_5 removal in the anaerobic lagoon is higher than expected, its predicted removal was never achieved. Taking advantage of a sligth difference in colour of the liquid at the entrance of the facultative lagoon, visual inspection suggested that a high degree of short circuiting must be occurring. The actual HRT is lower than the design value and therefore a system of bafles should be installed in order to minimize the problem.

Although a maintenance program has been suggested, since the lagoon has been performing according to what the plant manager expected, no measures have been taken to avoid macrophyte growing and bank erosion. In the long run this aspects will present problems of expensive solution but for the time being the lagoons, when in operation, have removed most of the impact of the plant waste water at a extremmely low running cost.

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INTENSIFICATION OF WASTE WATER TREATMENT IN STABILIZATION PONDS.

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ABSTRACT

Higher Aquatic Plants (HAP) provide more effective tertiary waste water treatment in biological ponds. They also accelerate the clean-up process with simultaneous increase of hydraulic loading, There was observed not only high clean-up efficacy for domestic organics, but also for various universal pollutants such as oil, synthetic surface active substances and phenoles.Our findings have demonstrated that the HAP stabilization ponds may be used successfully for industrial waste water treatment. We have also shown it expedient to apply the method in biological, biochemical and vitamine production plants and sugar refineries. HAP of reed, cane and rush type proved to be most effective in intensification of this process.

KEYWORDS

Tertiary waste water treatment; Higher Aquatic Plants (HAP); intensification; efficacy.

To intensify the process of waste water treatment in stabilization ponds HAP are widely used. Elodea, Phragmites, Tupha, Seirpus, Stratiotes and alike planted artificially into ponds promote effective extracting diluted organic substances, phenoles, oil, synthetic surphace active substances and also decrease water mineralization. That's why HAP stabilization ponds appear to be considerable potential in industrial and city sewage treatment(Seidelet al, 1967; Volga et al.,1977; Markov et al.,1978; Reed et al.,1982).

We have studied the ways of intensification of biochemical and vitamine production plants sewage treatment and that from sugar refinery in HAP stabilization ponds(Kravets et al.,1986). The efficacy of biochemical and vitamine production plants waste water treatment in stabilization ponds with Phragmites (70%), Tupha (14%), Carex (4%), Sparganium (2,5%), Butomus (2%), Scirpus (1,5%), Acorus (1%), Bidens (1%) showed 98-99% of initial pollution level.

After waste water had been kept in the ponds for 6 days Biochemical Oxygen Demand(BOD) showed 99,5% reduction, Chemical Oxygen Demand (COD) - 96,9%, suspended substances - 99,1%, ammonium nitrgen - 92,8%, phosphates - 99,8%, and general mineralization - 32,7% respectively.

The long-term studies on involving HAP in the process of waste water treatment at sugar refinery was aimed at its utility in reverse watersupply systems. The technological process of sugar refinery effluent clean-up includes preliminary mechanical treatment (foam-suppression, clarification in settling-tanks) and profound biological clean-up in HAP biopoпds. The HAP bioponds loading averaged 120 - 160 m^3 of effluents per a hectare daily. The capacity of every biopond was about 150 m^3.

For about 3 years we were controling treated water quolity parameteres including physico-chemical, hydrobiological and sanitary - bacteriological ones.

As had been corraborated by the trials HAP bioponds may be also effectively used in sugar refinery effluents treatment. The suggested method was found to intensify the process of tertiary waste water treatment ant its filtration into soil.

For the purpose we have chosen the plants of reed type as far as they promote proper water demineralization and intensify its clean-up process. In winter (December - February) when vegetation halts, the plants give about 3 -4 new shoots, because the growth of new suckers in winter time supports constant sorption of nutrients by roots.The fact vividly demonstrates that the intensive process ofself-purification in HAP filtration fields is all-year-round, for in our area the major flowing occurs in October - January.

AS the process of filtration is 2,5 times accelerated the bioponds can be filled to 1 meter depth. So its loading is then 2,5 times that noted before with excellent refinement quality in summer time (August - October). Depending on season it takes about 6 - 15 days to treat waste water at 80-90% effficacy rate with its further filtrating into soil. Water that had been treated in HAP bioponds meets all the requirements of technical water.

From our standpoint, the method of HAP bioponds provides extracting biogenic elements in the course of tertiary waste water treatment befor it enters natural reservoirs or is filtrated into soil. The method may be recommended for wide use as the cheapest and most economic technique to avoid eutrophication of natural reservoirs.

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WATER QUALITY IMPROVEMENT OF SECONDARY EFFLUENT BY AN OXIDATION POND WITH SUBSEQUENT SAND FILTRATION TREATMENT

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ABSTRACT

A series of oxidation ponds and subsequent sand filtration experiments was performed to improve the water quality of secondary effluent containing organics and nutrients. The results show that the main reaction mechanism in the oxidation pond is the fixation of inorganic nutrient by algae. T-N and T-P concentrations in pond effluent decrease to 80 % and 77 % of influent respectively owing to the sedimentation of the particulate fraction. The sand filtration experiment of the pond effluent was conducted. At a filtration rate higher than 5 m/day, physical filtration mainly takes place, and the clogging of sand filter occurs within 1-3 days. However, at a rate lower than 1 m/day, the process of decomposition of deposit is also observed, and the run length becomes longer (> 20 days). The removal efficiency of SS is very high, ranging from 97 to 100 %. The total removal efficiency by an oxidation pond with sand filtration treatment is 24 to 37 % in T-N and 32 to 75 % in T-P. The most rational filtration rate for good performance seems to be 0.5 m/day because of its long run length. The effect of fish (*Rhinogobius brunneus*) on this treatment becomes obvious, prolonging the run length by 20 % or more due to feeding on the deposit.

KEYWORDS

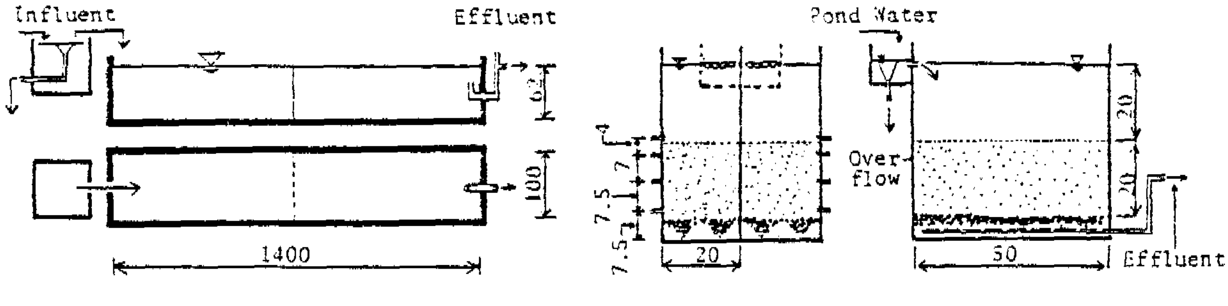
Oxidation pond, sand filtration, nutrients removal, tertiary treatment, fish.

INTRODUCTION

In the oxidation ponds used as tertiary treatment for water quality improvement and the removal of residual organics and nutrients, the carry-over problem of algae often occurs and reduces the removal efficiency of the nutrients (Somiya *et al.*, 1984). The objective of this paper is to clarify the removal performance of nutrients and organics contained in secondary effluent by the oxidation pond and the subsequent sand filtration treatment, and to establish effective operation conditions for sand filtration treatment. Of the many methods introduced to remove algae from pond water (Middlebrooks *et al.*, 1974, etc), the sand filtration process is chosen as a brush-up technique in this study. The sand filtration bed is installed separately from the oxidation pond to acquire the performance of each treatment. However, if we could prepare the filter on the pond bottom, additional area would not be required, which would be more advantageous in maintenance and management.

EXPERIMENTAL PROCEDURES AND METHODS

A series of experiments was performed at the Biwako Water Pollution Research Laboratory located on the shore of Lake Biwa. The schematic layout of a test oxidation pond and filtration equipment is shown in Fig. 1. The length, width and depth of the pond are 14 m, 1 m and 0.62 m, respectively. The secondary effluent from an activated sludge process was introduced as influent of the oxidation pond at a flow rate Q = 2.63 m^3/day, so that the detention time was about 3.3 days. The oxidation pond experiment started on June 11 and finished on November 18.



(a) Oxidation pond (cm) (b) Sand filtration equipment (cm)

Fig. 1. Layout of the test oxidation pond and sand filtration equipment

The sand filtration equipment consisted of two chambers. Each chamber had a water zone and a sand zone. The dimensions of the chambers were 50 cm in length, 20 cm in width, 20 cm in depth for the water zone and 20 cm in depth for the sand zone. Pond effluent was introduced at the inlet of the equipment and passed through the narrow pipes into water zone to reduce the disturbance. The effective size of the sand prepared for the filter bed was 0.65 mm. The uniformity coefficient and density of the sand were 1.46 and 2.72 g/cm^3, respectively. The normally filled-up porosity was 0.42. Eight filtration experiments (Run 1 - Run 8) were conducted at several filtration rates ranging from 0.5 m/day to 10 m/day. Table 1 shows the conditions of each experiment. Fish (*Rhinogobius brunneus*) were brought into the water zone in 2 of 8 runs. The filtration rate was controlled by the height of the outlet. Every experiment except Run 8 was terminated when the prescribed flow rate could not be maintained by this control owing to the clogging.

Table 1 Conditions of the experiment and the period of each run

Run No.	filtration rate (m/day)	period	run length (days)	total loading (m^3/m^2)	remarks
Run 1	10	Jul. 9, 8 A.M. - Jul.10, 8 A.M.	1.0	10.	
Run 2	5	Aug. 3, 6 P.M. - Aug. 6, 0 A.M.	2.3	11.	
Run 3	3	Aug.28, 6 P.M. - Sep. 6,10 A.M.	8.7	26.	
Run 4	1	Sep. 28 - Oct. 22	24.	24.	
Run 5	1	Sep. 28 - Oct. 27	29.	29.	fish
Run 6	0.5	Sep. 16 - Nov. 18	63.	31.5	
Run 7	0.5	Jun. 11 - Nov. 10	152.	76.	fish
Run 8	0.5	Jun. 11 - (Aug. 4)	(55.)	(27.5)	stopped by accident

Each water sample was taken at the inlet and outlet of the oxidation pond and at the outlet of the filtration equipment. The water quality, in terms of water temperature, pH, SS and chlorophyll-a, was measured daily, while that concerning nitrogen and phosphorus was measured weekly.

PERFORMANCE OF THE TEST OXIDATION POND

The results of the oxidation pond experiment are summarized in Table 2. These values show the averages over the whole experiment period (170 days). Temperature varied from 9.7°C (Nov. 13) to 30.8°C (Jul. 20) with an average of 22.3°C. The pH value increased intensively from 6.8 of influent to 9.6 of effluent. DO concentration also increased intensively from 3.43 mg/l to 16.55 mg/l. The increase of these indices shows the high algal activity in pond. The variation in the SS concentration in the influent and effluent is shown in Fig. 2. The effluent concentration of SS increased intensively. Its average was 4 times higher than that of the influent. The main component of the influent SS was supposed to be the debris of activated sludge. The variations of chlorophyll-a and SS in effluent show a similar pattern. The ratio of chlorophyll-a against SS was 0.011. This ratio is similar to that of phytoplankton (e.g. *Scenedesmus obliquus* 0.021, Tsuda,1964), so that the main component of SS in effluent is considered to be phytoplankton.

Table 2 Results of the oxidation pond experiment

	Temp. °C	p H -	D O mg/l	Chl.a mg/l	SS mg/l	P-COD mg/l	S-COD mg/l	T-N mg/l	P.Or-N mg/l	In-N mg/l	T-P mg/l	S-P mg/l	PO_4^{3-}-P mg/l
Influent	24.8	6.8	3.43	0.0	4.4	4.41	10.09	12.05	0.33	11.25	1.004	0.795	0.672
Effluent	22.3	9.6	16.55	0.201	17.9	21.66	11.35	9.58	1.31	7.60	0.773	0.459	0.370

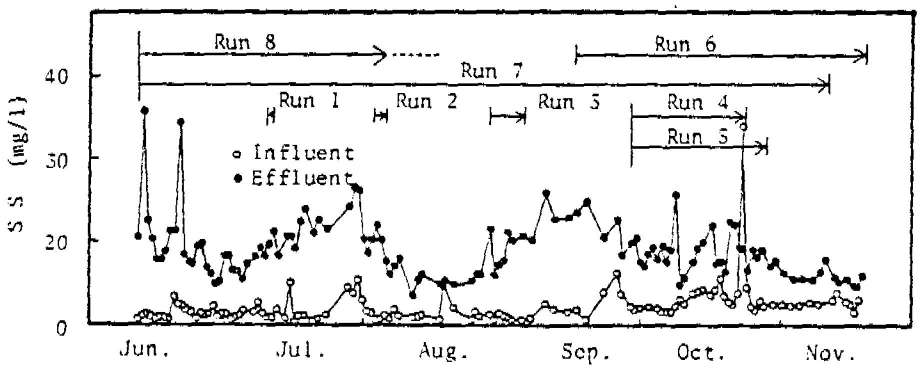


Fig. 2 Variation of SS concentration in the pond experiment and run periods of the sand filtration experiment

The observation of phytoplankton in pond water was performed with a microscope several times per month. Predominant species of algae were *Golenkinia sp.* and/or *Scenedesmus quadricauda* as shown in Table 3. The percentages of the main soluble components on nutrients (N, P) and CODcr, such as NO_3^--N / T-N, PO_4^{3-}-P / T-P and soluble CODcr (S-CODcr) / T-CODcr were 90.9 %, 66.9 % and 69.6 %, respectively. Using the pond treatment, inorganic nutrients such as NO_3^--N and PO_4^{3-}-P were reduced, and suspended materials such as P.Org-N, P-P and P-CODcr were increased. In pond treatment with 3.3 days of detention time, some of the nutrients were solidified by algae and removed from the pond water by sedimentation, so that T-N and T-P concentrations in pond effluent became approximately 80 % and 77 % of influent, respectively.

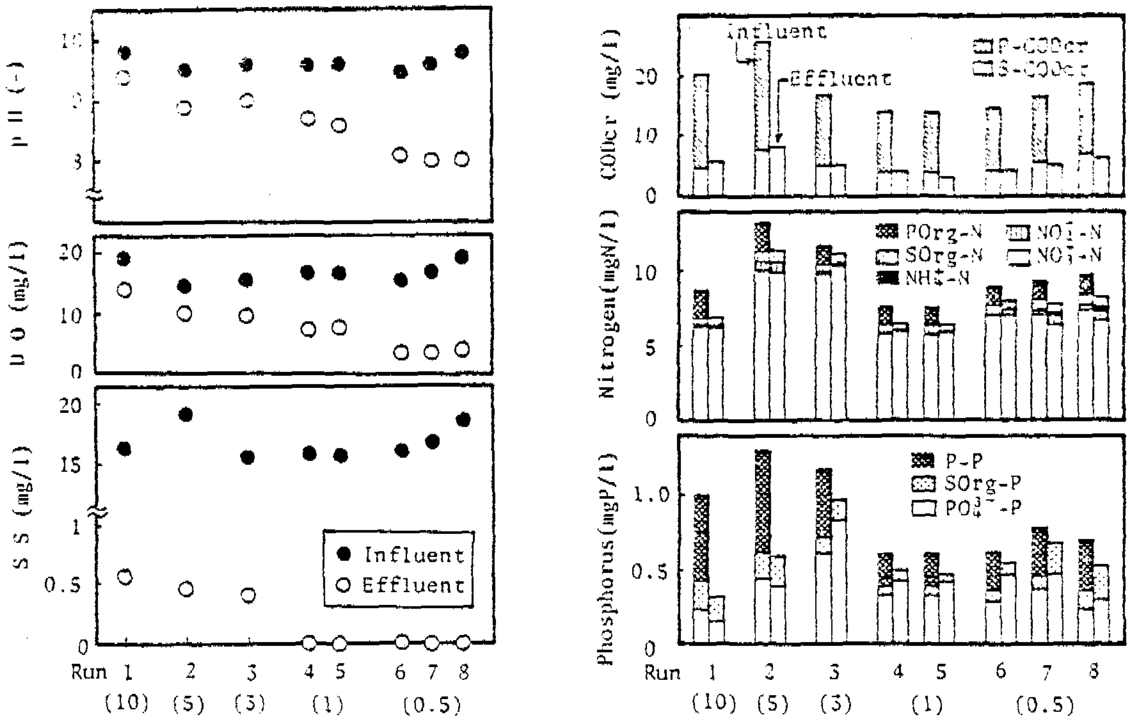
Table 3 Main algal species in the oxidation pond

	Jun. 15	Jul. 20	28	Aug. 24	26	Sep. 9	13	29	Oct. 3	5	16	26	Nov. 7
Golenkinia sp.		+++	++	+++	+	+++	+++	+++	+++	+++	+++	+++	+++
Scenedesmus quadricauda	+++	+++	++	+++	+++	+++	+++	+++	+++	++	++	++	++
Oocystis sp.		+++	+++					+	++	++	++	++	+
Nitzschia sp.		+			+						+		+
Pediastrum biwae									+				

+++ major ++ minor + trace

PERFORMANCE OF THE SAND FILTRATION

The results of eight sand filtration runs are summarized in Fig. 3. This figure shows the change in average water quality concentrations. The effluent sample from the sand filter layer contained little SS material, so that total concentration of the effluent sample was almost the same as the soluble concentration.



(a) SS, DO and pH (b) CODcr, nitrogen and phosphorus
Fig. 3 Results of sand filtration experiments (filtration rate, m/day)

DO, pH The values of both indices increased in the pond, while they decreased in sand filtration. This change is mainly caused by the reactions of algal respiration and bacterial decomposition. The lower DO concentration was measured at a slower filtration rate. The variation profiles of those indices are shown in Fig. 4. At a slower filtration rate, the value of pH and the concentration of effluent DO decreased to some extent after the start of the experiment and then became relatively constant.

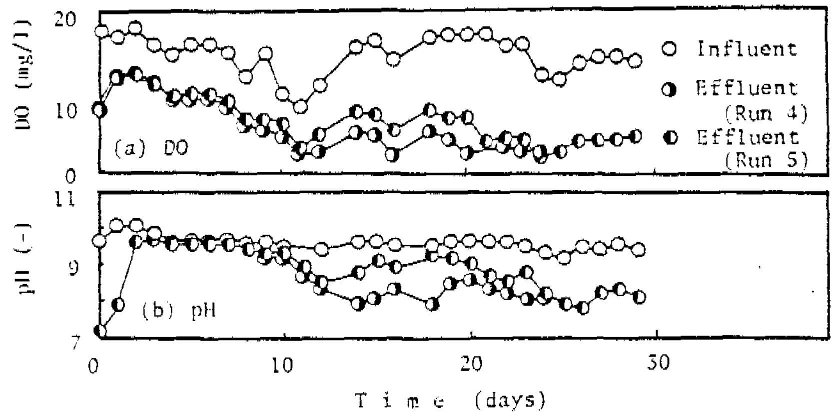


Fig. 4 Variation of pH and DO in sand filtration experiments

SS The SS concentration of the effluent was very low in every run, demonstrating the high performance of sand filtration for the removal of algae. When the filtration rate was higher than 3 m/day, suspended material slightly passed through the sand layer, but the average SS concentration was less than 1.0 mg/l, and the SS removal efficiency was higher than 97 %. The removal efficiency of the sand filter was very high, compared with 30-70 % SS removal efficiency in intermediate filtration of single-cell oxidation pond effluent (Morgan *et al.*, 1981).

CODcr, N, P Particulate CODcr was almost completely removed by filtration treatment, while a slight reduction of soluble CODcr was observed at a rate lower than 1 m/day. The average CODcr concentration was less than 17 m/l in every case. Nitrogen was reduced by 1 - 2 mg/l in the filtration process due mainly to the removal of particulate nitrogen (P.Or-N). At a low rate, the concentration of NO_3^--N decreased, and that of NO_2^--N increased slightly. At a rate higher than 5 m/day, the average concentration of total phosphorus in effluent was lower than that of soluble phosphorus in influent, while it was higher at a rate lower than 1 m/day. It seems that there are two processes for the increase and decrease of PO_4^{3-}-P concentration in the filter. One is adsorption on sand at the higher flow rate, while the other is the release from the decomposed deposit at the lower rate.

<u>Head loss</u> The variation in head loss is shown in Fig. 5. At a high rate treatment (10 m/day), head loss increased gradually and developed to 10 cm thickness of sand layer. The loss at a low rate (0.5 m/day) was very small in the beginning, and then it began to increase progressively. The head loss occurred mainly at the top thin layer below the surface. The head loss in the other cases (F.V. 1 - 5 m/day) showed the intermediate pattern of both examples.

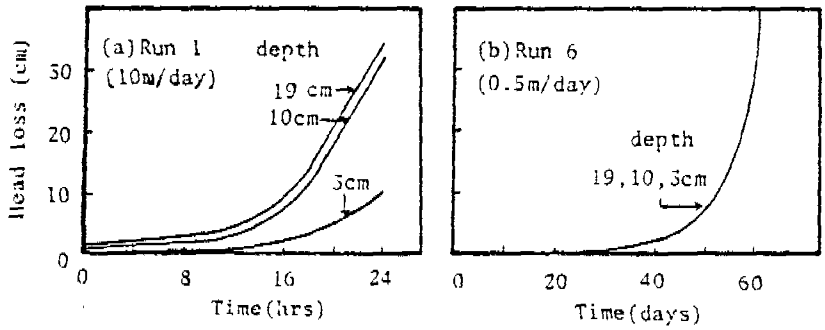


Fig. 5 Variation in head loss

<u>Run length</u> When the head loss reached a certain level and the filtration rate became lower than that prescribed previously, each run was stopped. The run length was only 24 hours at the rate of 10 m/day, while at the rate of 0.5 m/day, it was more than 60 days. The run length of the experiment with fish was 20 % or more longer than that without fish. From these results, it is shown that fish cultivation in the pond prolongs the run length of filtration.

DISCUSSION

<u>Solidification by algae</u> The data from the oxidation pond experiment clearly shows that the main reaction mechanism in the pond is the fixation of inorganic nutrients into organic suspended material as algal bodies. As long as algal activity exists in the oxidation pond, DO concentration increases intensively, and T-CODcr concentration increases in effluent. However, the biodegradable fraction of CODcr in the influent decreases easily in the pond. Some nutrients solidified by algae are able to be removed from the pond water by sedimentation, so that the T-N and T-P concentrations of pond effluent decrease. The efficiency of removal by sedimentation is not so high. To achieve a much higher removal efficiency of nutrients and organics, a higher removal of suspended material is necessary.

<u>Reactions in the sand layer</u> As shown in the previous section, various reactions, such as the physical straining of suspended material, chemical and biological reactions, take place in sand layer. At a high filtration rate, the run length is very short so that time is not sufficient for microbes to decompose the trapped organics. Consequently, a slight reduction in DO and pH, and a slight increase in S-CODcr are observed. This might also be related to the respiration or excretion of phytoplankton. When the filtration rate is slower, the biological reaction in the sand layer becomes more active. The DO concentration in the effluent decreases intensively to about 3 mg/l, and pH also decreases intensively. Nitrate decreases and nitrite increases. This shows there are some concurrent reactions of denitrification and nitrification in the layer. The increase in ortho-phosphate might be caused by the dissociation of coagulated ortho-phosphate owing to the decline of pH in effluent. S-CODcr decreases slightly at a rate slower than 1 m/day. This shows that there is the formation of a biofilm in the sand layer and the bacterial decomposition of soluble organic

Table 4. The effect of filtration rate on water quality change
(experiments without fish)

	filtration rate (m/day)				
	10	5	3	1	0.5
Run length (days)	1.0	2.3	3.7	24	63(,>55)
Total loading (m^3/m^2)	10	11	26	24	31.5(,>27.5)
Distribution pattern of deposit	on sand surface layer and in the layer 0-10 cm in depth	on sand surface layer and in the layer 0-3 cm in depth			mainly on sand surface layer
Decomposition of deposit (%)	8.1	8.0	15.9	31.3	44.7
Head loss	← increase progressively			very low in first half period then increase catastrophically →	
DO	(still supersaturated)		decrease		(3mg/l) →
pH	(a little)		decrease		(fair) →
S-CODcr	← a little increase				a little decrease →
SS(,P-CODcr PON,P-P)	← almost complete removal(>97%) →				
Nitrogen					a little denitrification →
Phosphorus	← adsorption of phosphate				release of phosphate →

matter. The results of the experiments without fish are summarized in Table 4.

Deposit Particle material is trapped as deposit by the filtration process in the sand layer. At the end of experiment, all deposit was collected and the profile of the SS distribution was measured. The results are shown in Table 5. At 10 m/day, the deposit is distributed as far as 10 cm depth, while at 0.5 m/day, most of the deposit remains within 3 cm of the sand surface layer. The fraction remaining on the surface area increases from 21 % in Run 1 to 98 % in Run 7 with the decrease in the filtration rate. From the mass balance analysis (Table 5), the ratio of decomposed material against trapped material is shown to increase from 8 % of Run 1 to 45 % of Run 6. However, some of the decomposed mass in Run 5 and 7 is contributed by fish feeding. These results show that the decomposition reaction in sand layer is especially important at a slow filtration rate and prolongs the run length.

Table 5 Distribution of deposit and SS loading (g/m^2)

Run (F.V. m/day)	Run 1(10)	Run 2(5)	Run 3(3)	Run 4(1)	Run 5(1)	Run 6(.5)	Run 7(.5)
On sand surface	44	103	274	327	339	327	781
At 0- 3 cm depth	68	141	152	20	37	21	6
At 3-10 cm depth	89	35	31	5	21	3	3
At 10-20 cm depth	4	9	5	2	4	3	3
Sum (A)	205	288	462	354	401	354	793
SS loading (B)	223	313	549	515	615	640	1684
100*A/B (%)	91.9	92.0	84.2	68.7	65.2	55.3	47.1
Remarks					fish		fish

Selection of a rational filtration rate As shown in Table 4, the effluent quality is not significantly different for each filtration rate. The run length of filtration at 0.5 m/day is considerably longer than that of the other cases, and the total hydraulic loading of each run is much higher at 0.5 m/day than that of the others. Thus, it is concluded that the filtration rate of 0.5 m/day is considered to be the most rational value for the sand filtration operation. The oxidation pond used for this treatment has a depth of 0.63 m depth and a detention time of 3.3 days, so that the surface loading rate is about 0.19 (= 0.63/3.3) m/day. If we choose a filtration rate of 0.5 m/day, the bottom area needed for sand filtration is estimated to be less than half of the total bottom area of the oxidation pond. Thus, it is easily recognized that the filtration rate of 0.5 m/day is a realizable value.

Fish   Fish (*Rhinogobius brunneus*, length:2-3cm) were brought into the upper storage chamber to reduce the deposit and to prolong the run length. The results show that this purpose was achieved in the experiment. The run length of the experiment with the addition of fish is prolonged by 20 % or more than that without fish(table 6). The percentage of deposit to influent SS loading is lower at Run 5 than at Run 4, and also lower at Run 7 than at Run 6. These results show that fish eat the deposit or help bacteria to decompose it. The total deposit remaining at the end of the experiment with fish is more abundant than that without fish. The difference in the remaining deposit mass might be related to the actions of fish, such as the mixing and disturbance of sand surface layer. Consequently, the run length is extended. Thus, it is believed that the effect of organic and nutrient control that fish have is good.

CONCLUSION

A series of oxidation ponds and subsequent sand filtration experiments was performed to clarify the removal performance of nutrients and organics contained in secondary effluent by the oxidation pond and the subsequent sand filtration treatment, and to establish the effective operation condition of sand filtration treatment. The results are summarized as follows:

1. Some nutrients solidified by algae are removed from the pond water by sedimentation, so that T-N and T-P concentrations of pond effluent treated under 3.3 days' detention time decrease to 80 % and 77 % of influent, respectively.

2. At the sand filtration with a filtration rate higher than 5 m/day, physical filtration mainly takes place and the clogging of the bed occurs within 1-3 days, while at that of a rate lower than 1 m/day, the decomposition process of the deposit is also observed, and the run length becomes longer (> 20 days). The removal efficiency of SS is very high, ranging from 97 to 100 %. The total removal efficiency by an oxidation pond and a sand filtration treatment is 24 to 37 % in T-N and 32 to 75 % in T-P.

3. The filtration rate of 0.5 m/day is considered to be a rational value for sand filtration operation. At this rate, the run length is considerably longer and total hydraulic loading is much higher than at the others, though the effluent quality is not significantly different for each filtration velocity.

4. The percentage of deposit to influent SS loading is lower in the experiments with fish than in those without fish. Moreover, the total deposit remaining at the end of experiment using fish is more abundant than that without fish. Consequently, the run length of the experiment with fish is longer by 20 % or more than that without fish. It can be surmised that fish have a good effect on the sand filtration of pond effluent.

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THEME 3

MICROBIOLOGY AND PHYSICOCHEMISTRY

NITROGEN TRANSFORMATIONS AND REMOVAL IN WASTE STABILIZATION PONDS IN PORTUGAL: SEASONAL VARIATIONS.

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ABSTRACT

Nitrogen Transformations that can occur in WSP depend on pond and waste characteristics and nature and are influenced by climatic factors, like temperature and precipitation. Experiments described had been performed in a treatment system, with an anaerobic, one facultative and one maturation pond, in a serial set treating domestic sewage. In this paper we intend to clear up wich N transformations processes happen in each one and seasonal variations in N - removal or bioconversion during the year.
Results proved that in anaerobic pond there was an important N - organic removal, mainly by mineralisation and not only by sedimentation.
Some of the processes of N - transformation observed in the ponds were more strongly influenced by weather conditions than others. In some cases, the increase of biological activity, that was induced by the increase of air temperature, was masked by the reduced precipitations observed, what made less diluted the treated effluents. These climatic factors can explain some of the variations observed along the year, in what concerns nitrogen compounds concentrations.

KEYWORDS

Waste Stabilization Ponds; Domestic Sewage; Nitrogen Transformations; Nitrogen Removal; Season Variation of Nitrogen Transformations and removal.

INTRODUCTION

The utilization of waste stabilization ponds for the treatment of industrial and domestic effluents in Portugal is a low cost technology of great interest. In fact they provide a simple and effective method of waste treatment at low energetic costs and with minimal operations and maintenance requirements.

Climatic conditions, specially sun radiation and thermal amplitude, seem to be favourable for the development of biological species involved in this type of waste treatment.

According to Oliveira and Sousa (1986) there was 53 pond systems working in Portugal at that moment, 23 on domestic sewage and 30 on industrial effluents. Other 40 systems were under construction.

Waste stabilization ponds system at Frielas has been the first one built with the purpose of making research on ponds performance at Portugal. The experimental results obtained in some of these ponds, treating domestic sewage, were choosen to investigate Nitrogen transformations and removal.

NITROGEN TRANSFORMATIONS IN WASTE STABILIZATION PONDS

In waste stabilization ponds there are, as it is generally known, a number of processes that promote Nitrogen transformation. The main processes are:
- Biological hydrolysis of Organic Nitrogen, with the release of Ammonia Nitrogen
- Assimilation of Nitrogen by bacteria and algae, to built up cellular material
- Nitrification of Ammonia, to Nitrite and Nitrate
- Denitrification of Nitrate, to Nitrogen gas.

Some points must be put in evidence in this introduction. Nitrification is an aerobic process, so it do not occur in anaerobic ponds. Denitrification requires anoxic conditions, so it can occur only in anaerobic ponds or in the bottom of facultative ponds. Ammonia volatization is another possible removal process (in some specific physico-chemical conditions), but where N - transformation do not occur.

A mass balance between the concentration of each species of Nitrogen compounds in the influent and in the effluent of a pond permit to determine their percent removal or increase. Some weather factors, such as temperature and rainfall, can affect removal or increase of N - parameters efficiency.

The present work intends to show the variations that can occur in Nitrogen transformations, according to the season of the year, in each different type of pond.

THE PONDS

The three ponds choosen, A1 (anaerobic), F1 (facultative) and M1 (maturation) form a serial set, which means that the effluent of A1 is the influent of F1, and the effluent of F1 is the influent of M1. The influent of A1 is raw sewage .

Table 1 shows some of the ponds functioning characteristics such as mean detention time (t), flow rate (Q), pond volume (V), pond depth (D) and surface area (A).

TABLE 1 - Ponds Functioning Characteristics

	Pond A1	Pond F1	Pond M1
t (day^{-1})	1.7	17.3	9.7
Q (m^3/day)	233.3	95.0	46.7
V (m^3)	396.6	1643.5	453
D (m)	3	1.1	1.1
A (m^2)	256	931	324

MATERIALS AND METHODS

The sampling programme have been carried on since September 1984 until October 1985. Sampling points were: 1 - Arrival of Raw sewage to the system; 2 - Effluent from A1; 3 - Effluent from F1; 4 - Effluent from M1.

Samples were colleted one to four times a month, with sample collectors, for a period of twenty--four hours. After that, all simple samples, of each sampling point, were mixed, in order to obtain composite samples.

Composite samples undergone laboratory analysis to determine Total Kjeldhal Nitrogen (TKN), Soluble Nitrogen (SKN), Ammonia Nitrogen(N-NH3+N-NH4 $^+$), Nitrite (N-NO_2^-) and Nitrate (N-NO_3^-).

Kjeldhal Nitrogen was determined by the kjeldhal technique after filtered (SKN) and unfiltered (TKN) samples mineralization with Selenium (ISO - DIS 5663)

Nitrite was analysed by the diazotization method (ISO - DIS 6777)

Ammonia Nitrogen and Nitrate were determined by specific electrodes (ORION Research, Cambridge, Massachusetts).

Total Organic Nitrogen (TON) and Soluble Organic Nitrogen (SON), were calculated by direct subtraction between Total Kjeldhal Nitrogen and Soluble Kjeldhal Nitrogen and Ammonia. Suspended Organic Nitrogen (Susp. O.N.) is equal to Total Organic Nitrogen, minus Soluble Organic Nitrogen.

RESULTS AND DISCUSSION

Annual mean percentual variation of nitrogenous species along the pond system is presented in table 2.

TABLE 2 - Annual Mean Percent Variation of Nitrogenous Species Along the Pond System
(Mean Concentration of Raw Waste = 100%)

%	Pond A1	Pond F1	Pond M1
TNK	-10.5	-40.3	-53.3
SNK	-8.6	-43.4	-57.3
$N-NH_4^+$	+10.7	-35.7	-52.4
TON	-28.6	-44.4	-54.2
SON	-34.7	-53.3	-62.6
Susp.O.N.	-24.1	-37.0	-43.2
$N-NO_2^-$	-29.6	+11.1	+285.2
$N-NO_3^-$	- 0.4	+20.3	+50.2

Figure 1 represents schematically data from Table 2.

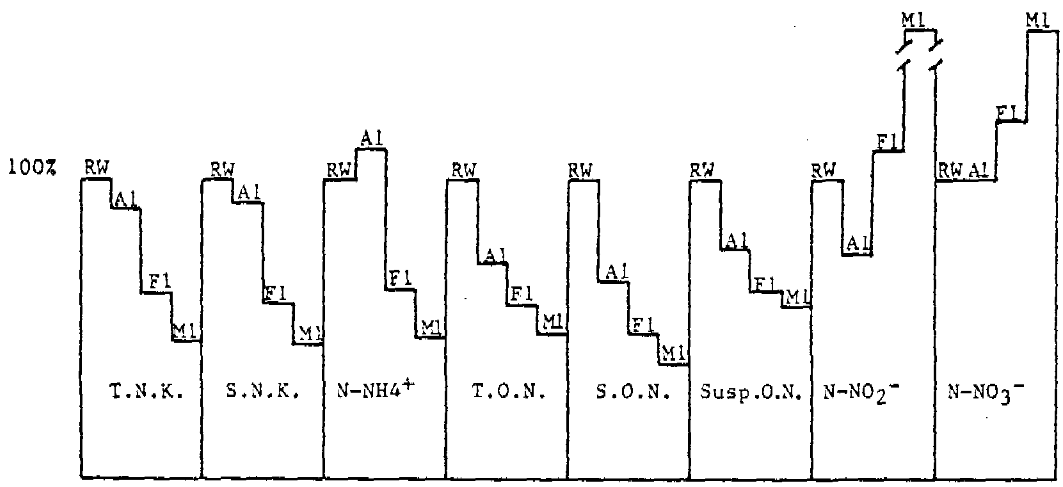

Fig. 1 - Annual percent variation of nitrogenous species along ponds system
(Mean concentration in raw waste = 100%)

As it can be seen the higher mean percentual removal of Organic Nitrogen occured in the anaerobic pond (A1) because of anaerobic mineralization and sedimentation. Mineralization happening was proved by the increase of Ammonia Nitrogen content in this pond.

Ammonia removal was larger in facultative pond F1, because it has been assimilated by bacteria and algae, as Nitrogen source, and nitrified by nitrificant bacteria to Nitrite and Nitrate.

Nitrification had been also important in maturation pond M1.

Denitrification processes could not be proved by these results.

Table 3 presents the mean percentual variation of Nitrogen compounds in each season of the year.

TABLE 3 - Mean Percentual Variation of Nitrogen Compounds
Along the Pond System, at each season of the Year
(Raw Waste = 100%)

	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Ap.	May.	June	July	Aug.	Sept.
	A1	F1	M1	A1	F1	M1	A1	F1	M1	A1	F1	M1
TKN	-12.7	-33.6	-48.3	-11.2	-33.8	-41.0	-12.8	-50.3	-61.6	-12.0	-54.4	-68.0
SKN	-7.3	-40.2	-51.6	-8.8	-43.8	-49.0	-10.5	-46.9	-61.7	-8.6	-52.4	-66.3
$N-NH4^+$	+8.0	-29.1	-46.0	+5.6	-30.2	-41.1	+9.6	-41.6	-59.0	+15.2	-46.3	-65.3
TON	-30.9	-37.7	-50.4	-23.5	-35.6	-40.9	-29.9	-57.9	-63.8	-38.1	-62.3	-70.7
SON	-31.7	-49.0	-60.6	-25.8	-46.6	-53.9	-37.8	-54.2	-65.4	-46.2	-61.9	-67.7
Susp. O.N.	-30.9	-23.5	-37.6	-33.2	-46.1	-34.2	-21.8	-64.9	-60.9	-26.0	-62.7	-75.1
$N-NO2^-$	-51.8	-50.0	+129.6	-47.0	+117.6	+611.8	-28.6	+85.7	+842.9	+33.3	+75.0	+250.0
$N-NO3^-$	+2.3	-108.8	+20.8	+3.2	+19.6	+51.9	-92.3	+10.6	+52.2	+1.5	+59.8	+108.6

Figures 2 to 7 represent the percentual variation of T.K.N., S.K.N., $N-NH4^-$, T.O.N., S.O.N. and Susp. O.N. in Autumm (season 1), Winter (season 2), Spring (season 3) and Summer (season 4).

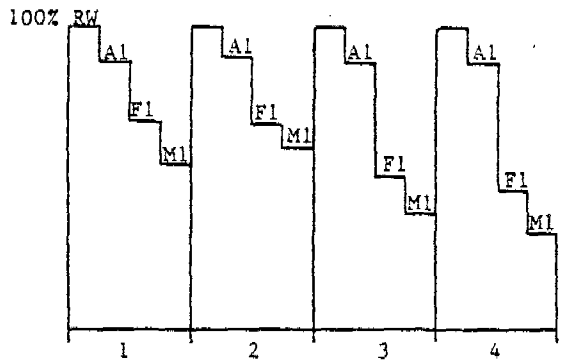

Fig. 2 - Variation of TKN along the pond system at the four differents seasons of the year (Raw Waste = 100%)

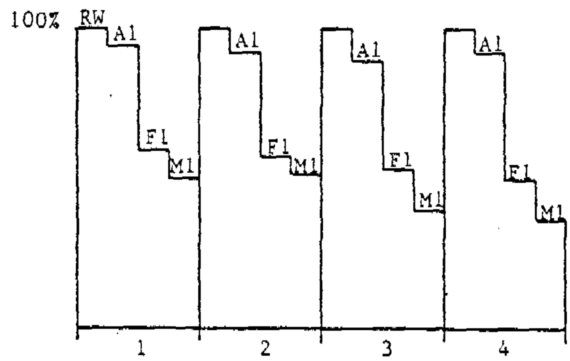

Fig. 3 - Variation of SKN along the pond system at the four differents seasons of the year (Raw Waste = 100%)

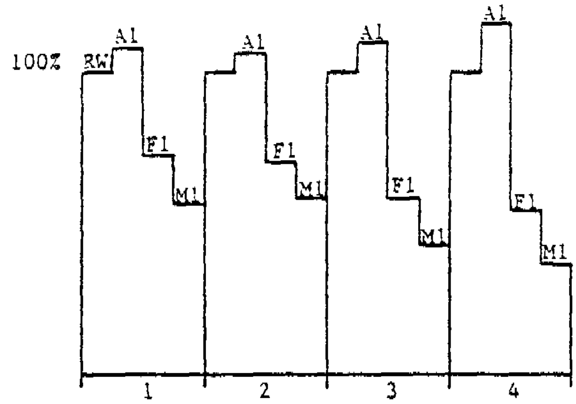

Fig. 4 - Variation of N-NH4$^+$ along the pond system at the four differents seasons of the year (Raw Waste = 100%)

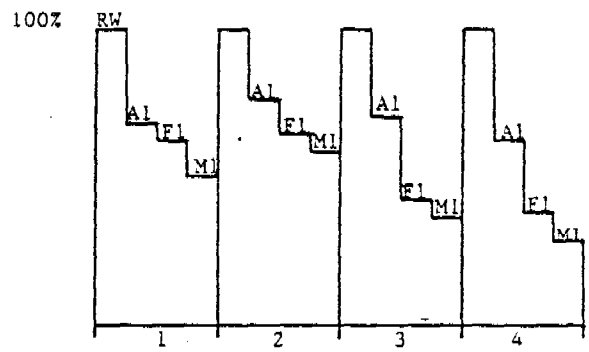

Fig. 5 - Variation of T.O.N. along the pond system at the four differents seasons of the year (Raw Waste = 100%)

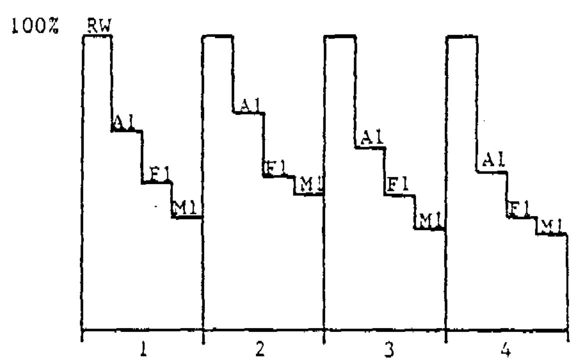

Fig. 6 - Variation of S.O.N. along the pond system at the four differents seasons of the year (Raw Waste = 100%)

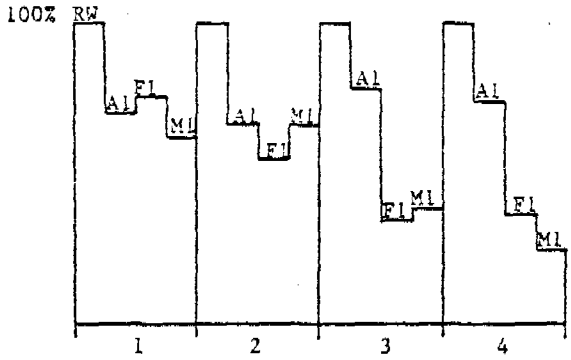

Fig. 7 - Variation of Susp. O.N. along the pond system at the four differents seasons of the year (Raw Waste = 100%)

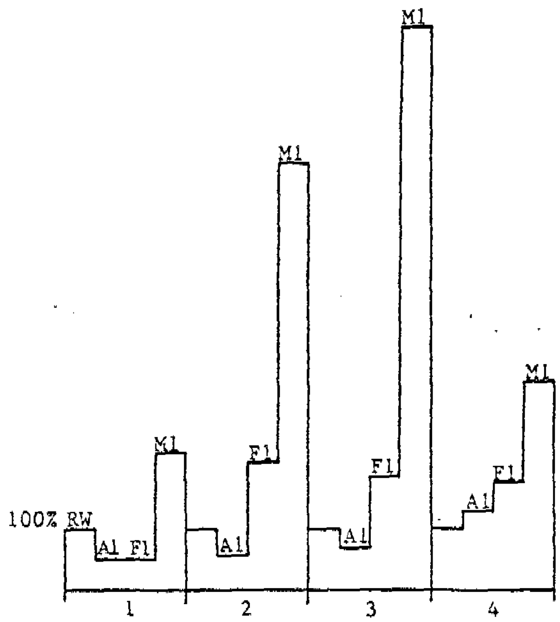

Fig. 8 - Nitrite Variation along the pond system at the four differents seasons of the year (Raw Waste = 100%)

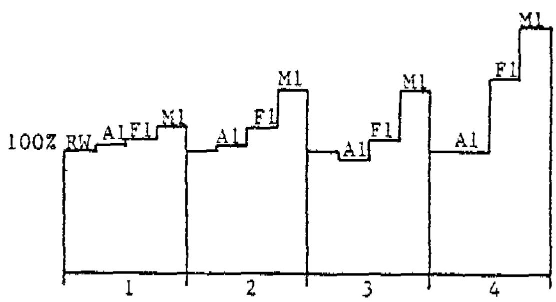

Fig. 9 - Variation of Nitrate along the pond system at the four differents seasons of the year (Raw Waste = 100%)

As it can be seen the variation of all Nitrogen compounds was not identical in all the seasons Generally, removal and bioconversion had been greater in summer and smaller in winter. The reason for this is temperature dependence of biological processes.

Pond effluents dilution by rainfall during Autumm and Winter, increase apparent removal observed wich is higher than biological activity would explain, masking the effect of the low temperatures ragistered in the same periods. The influence of rainfall had been demonstrated by correlating it with nitrogenous compounds concentration in the pond effluents. The function that more often translated these correlations was the hyperbolic one with correlation coefficients (r) of 0,62 to 0,99, and Snedecor Variables (F) of 8 to 15. To exemplifie, it can be said that Soluble Organic Nitrogen concentration in pond effluents can be correlated with rainfall (monthly average $1/m^2$) by the following equations:

$$[S.O.N.]_{A1} = 0{,}099 \text{ Rain} - 0{,}021 \quad (r = 0{,}93; F = 12)$$

$$[S.O.N.]_{F1} = 0{,}121 \text{ Rain} - 0{,}011 \quad (r = 0{,}92; F = 11)$$

$$[S.O.N.]_{M1} = 0{,}161 \text{ Rain} - 0{,}013 \quad (r = 0{,}88; F = 9)$$

Soluble Organic Nitrogen mineralization had been the most temperature influenced process in pond A1; the mean concentration of $N-NH4^+$ during summer season is the parameter whose increase is higher. The correlation between air temperature (ºC) and $N-NH4^+$ concentration in A1 effluent could be expressed by a straight line (correlation coefficent r = 0,92; Snedecor variable F=71) whose equation is:

$$[N-NH4^+]_{A1} = 1{,}891 \text{ Temp.} + 6{,}747$$

Removal of $N-NH4^+$ in pond F1 had been greater during spring and summer probably on account of the increased phytoplancton assimilation and bacterial nitrification.

Pond M1 registered a high population of Daphnia Spp. at the end of spring and during summer. These organisms, eat a great part of maturation pond algae; this process have influenced the results of $N-NH4^+$ and Suspended Organic Nitrogen concentration in this pond effluent.

Nitrification process assumed a very interesting behavior. In fact there was an accumulation of Nitrite in pond M1 during Winter and Spring. This Nitrite is oxidized to Nitrate in Summer and Autumm. It is hard to understand the increase of Nitrite in pond A1 during the Summer. May be it was a consequence of some Ammonia oxidation in the pond surface layers.

Nitrate concentration in F1 and M1 effluents increased significantly in Summer.In fact Nitrate concentration in F1 and M1 effluents could be correlated with air temperature (monthly average ºC) by a modified inverse function expressed by the following equations:

$$[N-NO_3^-]_{F1} = -0{,}023 \text{ Temp.} + 0{,}764 \quad (r = 0{,}86; F = 24)$$

$$[N-NO_3^-]_{M1} = -0{,}019 \text{ Temp.} + 0{,}614 \quad (r = 0{,}82; F = 15)$$

The comparison between fig. 8 and 9 enable us to conclude that Nitratation (Nitrite oxidation to Nitrate) is more dependent on temperature then Nitritation (Ammonia oxidation to Nitrite).

CONCLUSIONS

Waste stabilization ponds provides a good removal of Nitrogen Compounds, especially Organic Nitrogen and Ammonia. The introduction of an Anaerobic pond at the System head is interesting when the raw waste contains great amounts of Organic Nitrogen. Removal of Ammonia Nitrogen in Facultative and Maturation ponds depends on many factors such as temperature and phytoplancton concentration. Everything that could affect algae population density, affects also the Ammonia removal, for the specific populations of these ponds.

The existence of a maturation pond at the end of the system is desirable when a significant nitrification is wanted.

It may be concluded that pond performance, in what respects Nitrogen transformations and removal, is different in each season of the year, but the influence of temperature is not identical for all the processes that may be identified in these antrofic ecosysthems.

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MICRORGANISMS REMOVAL IN WASTE STABILIZATION PONDS IN PORTUGAL

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ABSTRACT

In this study we evaluated the removal of the traditional biological indicators of faecal pollution and two new indicators: Pseudomonas aeruginosa and Clostridium perfringens, in waste stabilization ponds installed at the waste treatment plant of Loures, at Frielas, near Lisbon. The efficiency of removal was 10/100 ml in the anaerobic pond A1 for indicator organisms, except one of them, and less in A2. In facultative ponds the removal were 10/100 ml. In maturation pond M2 the efficiency of removal was higher for the tradicional indicators than for the new indicators; in pond M1 the removal was lesser than in M2.

KEYWORDS
Anaerobic, facultative, maturation pond; Pseudomonas aeruginosa; Clostridium perfringens; heterotrophic bacteria; classical system; yield; K 20.

INTRODUCTION

The main purpose of the bacteriological examination of water and wastewater is to detect human or animal faecal pollution. The presence of faecal pollution constitutes a danger to health from intestinal infections caused by excreted pathogens. Traditionally faecal coliforms and faecal Streptococci have been used as indicators organisms to detect human and animal faecal pollution.

The minimum requirement for a biological indicator is that it must be a biotype that is prevalent in sewage and excreted by both humans and animals. Futhermore, it should be present in higher numbers than the pathogenic bacteria; be incapable of proliferation in the extra-intestinal environment; be more resistant to chemical agents used for water and wastewater treatment; it should be cultivated and enumerated by simple and rapid laboratory procedures (Buttiaux and Mossel, 1961; Bonde, 1963, 1966;Oliveira 1986). However, it is unlikely that any one particular organism chosen as an indicator fulfill all these requirements. For this reason others bacteria and yeasts have been investigated as new indicators. We chose Clostridium perfringens and Pseudomonas aeruginosa.

The purpose of our study was to evaluate the removal of faecal coliforms, faecal Streptococci, total heterotrophic bacteria, Clostridium perfringens and Pseudomonas aeruginosa in waste stabilization ponds installed at the waste treatment plant of Loures, at Frielas, near Lisbon (Oliveira and Sousa 1986).

MATERIAL AND METHODS

Colection of samples and bacterial enumeration. Raw sewage, influent and effluent of the ponds A1 and A2, F1 and F2 and M1 and M2, were collected in sterile glass bottles and transported to the laboratory in iced boxes.

Faecal coliforms, faecal Streptococci and Clostridium perfringens were enumerated on membrane filters (Milipore; type HAWG 04750).

For enumeration of faecal coliforms pads saturated with 0,1% Lauryl sulphate broth were used. Incubation at 43°C for 24h.

Faecal Streptococci were counted on membranes incubated on KF Streptococcus agar. Plates were incubated at 44.5°C for 48 h.

Clostridium perfringens was enumerated on Perfringens agar supplemented with oleandomycin, polymyxin B, sulphadiazine (Oxoid): incubation was conducted anaerobically in Oxoid Gas Generating Kit jars, incubated at 37°C for 48 h.

The method used to count total heterotrophic bacteria was pour plate in Peptone - yeast extract agar at 20°C for 72 h.

For the enumeration of Pseudomonas aeruginosa the appropriate portions of the decimal dilution were spread on the surface of plates of Pseudomonas agar (Oxoid) supplemented with cetrimide and nalidixic acid, with sterile L - shapped glass rod. Incubation took place at 25°C for 48 h.

Sterile quarter-stregth Ringer's solution was used to prepare 10 - fold serial dilutions of sewage and also used for the filtration and final rinsing of the filter funnel.

RESULTS AND DISCUSSION

Table 1 presents the efficiency of removal and the geometric mean of the influent and the effluent of six ponds: A1 and A2; F1 and F2; M1 and M2.

TABLE 1 - Removal of Five Bacteriological Parameters in Ponds A1 and A2; F1 and F2; M1 and M2

PARAMETERS	Geometric Mean		η (%)	Geometric Mean		η (%)
	Influent	Effluent		Influent	Effluent	
	A1			A2		
Faecal Coliforms(1)	2.1×10^7	6.0×10^6	77.1	2.1×10^7	1.4×10^7	66.7
Faecal Streptococci(1)	5.1×10^6	8.7×10^5	79.6	5.1×10^6	0.9×10^6	85.6
Total Heterotrophic(2)	1.2×10^7	5.4×10^6	76.0	1.2×10^7	4.3×10^6	-
Pseudomonas aeruginosa(2)	1.4×10^4	3.5×10^3	72.6	1.4×10^4	3.1×10^3	75.0
Clostridium perfringens(1)	5.7×10^5	1.2×10^5	78.2	5.7×10^5	2.0×10^5	81.2
	F1			F2		
Faecal Coliforms(1)	6.0×10^6	1.0×10^5	93.8	6.0×10^6	2.6×10^5	94.2
Faecal Streptococci(1)	8.7×10^5	1.1×10^4	95.7	8.6×10^5	1.7×10^4	94.4
Total Heterotrophic(2)	5.4×10^6	2.8×10^6	76.3	5.4×10^6	9.5×10^6	-
Pseudomonas aeruginosa(2)	3.5×10^3	1.7×10^2	90.6	3.5×10^3	1.2×10^3	83.3
Clostridium perfringens(1)	1.2×10^5	1.6×10^4	85.6	1.2×10^5	1.2×10^4	82.6
	M1			M2		
Faecal Coliforms(1)	1.0×10^5	2.2×10^3	96.4	1.0×10^5	7.9×10^2	91.9
Faecal Streptococci(1)	1.1×10^4	1.0×10^3	91.0	1.1×10^4	1.1×10^2	90.5
Total Heterotrophic(2)	2.8×10^6	6.9×10^5	82.4	2.8×10^6	8.1×10^5	-
Pseudomonas aeruginosa(2)	1.7×10^2	6.4×10^1	77.7	1.7×10^2	5.4×10^1	86.4
Clostridium perfringens(1)	1.6×10^4	8.3×10^3	74.1	1.6×10^4	4.5×10^3	83.5

(1) Colonies number/100 ml
(2) Colonies number/ml

The data show that indicator organisms were removed 10/100 ml in the pond A1 and less in A2, except Clostridium perfringens. Slight reduction in this organism may be explained by the great resistance of the spores. In the facultative ponds the removal were 10/100 ml, except in total heterotrophic bacteria, as will be expected (they are not bacteria indicator). In the maturation pond M2 the efficiency of removal was from 10^3/100 ml for faecal coliforms to 10^2/100 ml for faecal Streptococci to 10/100 ml for the others bacteria indicator. In pond M1 the removal was lesser.

Table 2 show some stastic values of yield of "classical system" ponds A1, F1 and M1.

TABLE 2 - Global Yield of Classical System A1, F1, M1

Statistic Values	Bacteriological Parameters				
	Faecal Coliforms	Faecal Streptococci	Total Heterotrophic	Pseudomonas aeruginosa	Clostridium perfringens
$\bar{X}$	99.940	99.840	88.310	98.530	96.300
X_{max}	99.999	99.999	99.710	99.928	99.957
X_{min}	99.468	97.666	61.538	84.040	81.875
σ_X	0.001	0.005	0.117	0.033	0.046
cv_X	0.001	0.005	0.133	0.034	0.047

The results show that global yield was higher in faecal coliforms and faecal *Streptococci* than in the others bacteria.

Table 3 presents the values of interception and the slope in experimental linear correlaction between ln Kt and $\frac{1}{T}$ (equaction van't Hoff - Arrhenius) and the values of θ were determined (θ = temperature factor).

TABLE 3 - Variation with Temperature of the Constante of Faecal Coliforms Removal (equation van't Hoff - Arrhenius)

Ponds	Temperature					
	$5 \leqslant T < 15^{o}C$			$15 \leqslant T < 25^{o}C$		
	a	b	θ	a	b	θ
A1	-37802.52	132.628	1.60453	-6582.27	22.972	1.07980
F1	- 4768.31	17.554	1.06146	-20061.35	69.834	1.26366
M1	15459.59	-53.443	0.73495	24621.19	-83.562	0.75036

The data show that in pond A1 and F1 the values of θ were higher than 1 and in pond M1 were lower than 1. In ponds F1 and M1 the values of θ increase with the increase of temperature which was not expected.

Table 4 presents some stastic values of η and K20.

TABLE 4 - Some Statistic Values of and K20

Statistic Values	Pond A1		Pond F1		Pond M1	
	Parameters					
	η(%)	K20(day^{-1})	η (%)	K20(day^{-1})	η (%)	K20(day^{-1})
$\bar{X}$	77.1	15.607	94.00	28.809	96.4	9.949
X_{max}	90.0	200.300	99.99	301.800	99.9	65.830
X_{min}	57.0	0.395	56.20	0.079	91.0	0.005
σ_X	10.2	38.388	9.10	66.889	3.2	30.426
cv_X	13.2	2.459	9.60	2.322	3.4	3.058

The values of K20 were independent of the temperature and these values depend of the kind of pond and show small values in pond M1.

CONCLUSIONS

The microbiological quality of the final effluent of "classic system" in M1 didn't achieve the water quality requitements, and couldn't be used in irrigation even restrict FC 10^3/100 ml, parameters of WHO and EEC. It will be necessary another maturation pond in order to improve the microbiological quality of the final effluent.
In what concerns the scheme including M2 pond the quality is already acceptable for irrigation.

The quality of the final effluent depends strongly of the retention time and eventually on hydraulic patterns in reactors, beeing essential a careful control of their characteristics.

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THEME 4

POND BENTHOS

SEDIMENTATION AND DIGESTION ON POND BOTTOMS : AN ATTEMPT TO ESTABLISH A SHORT TERM MATERIAL BALANCE

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ABSTRACT

Sedimentation and digestion have been measured in situ on the bottom of the first (facultative) and the last (maturation) pond of a wastewater stabilization pond system during six periods of one week. Calculated dry solids balances of the ponds bottoms showed that most of the collected solids would have an endogenous origine. Identification of the collected solids by digestion in the laboratory and supplemental in situ measurements in winter, when gas production was low, allowed the conclusion that the collected solids actually were fairly stabilized bottom sediments which are resuspended by the vigorous gas production. The establishment of a short term sedimentation-digestion balance appears to be a practical impossibility on an actively fermenting pond bottom.

KEYWORDS

Wastewater stabilization pond ; sludge accumulation ; digestion.

INTRODUCTION

The accumulation of sediments in wastewater stabilization ponds can affect their performances by reducing the pond volume and shortening the hydraulic retention time (Schneiter et al., 1984), by the feedback of soluble organic matter (Bryant et al., 1984) and by the resuspension of particulate matter, increasing turbidity and suspended solids concentration in the overlaying water (Parker et al., 1968). As a consequence, periodic desludging of the ponds is necessary.

The frequency of desludging depends on the sludge accumulation rate which is the result of the deposition and the decomposition of particulate matter. The rates of both processes are not well known.

The aim of this study was to develop a methodology to quantify deposition and decomposition rates on basis of short term observations (weeks).

It was interesting to study these phenomena in the first and the last pond of a four ponds system (La Chapelle Thouarault, France) since Carré et al. (1986), noticed that the sludge accumulation rate in the fourth pond (a maturation pond) was of the same order of magnitude as the one in the first pond (a facultative pond), which was preceeded by an Imhoff-tank. This seemed surprising, but could be caused by :

- a high algae production in this last pond, or
- a low digestibility of the algae organic matter.

Indeed Golueke et al., (1957) noticed methane production from algae to be lower than from raw sewage sludge when expressed as ml CH_4/g VS.

Deposition and decomposition rates now were evaluated by placing sludge collectors and gas collectors on various sites in the ponds. Decomposition was furthermore studied by digestion of sediment samples in the laboratory.

MATERIALS AND METHODS

The pond system
The pond system studied has been described earlier by Carré et al., (1986). The areas of the first and the fourth pond are respectively 2 156 m^2 and 2 855 m^2.

The period of study
After a period of preparation of sampling techniques, the field measurements have been realized from the end of Mai until the beginning of July. The weather was sunny and dry ; sediment temperature varied from 15°C to 17°C. Some supplemental field measurements were realized during winter when the sediment temperature was 6°C.

Sampling

Fresh sediment. Sediment collectors consisted of 0.159 m^2 circular PVC discs with a 4 cm high wall. The collector could be closed before lifting with a perforated cover allowing air and water to escape without disturbing the collected sediment.
The collectors were left for six periods of one week, placed on the longitudinal axes of the ponds. Each week three samples were taken in the first pond and two in the last pond. They were transferred to bottles without air inclusion and stored at 4°C in the laboratory for further analysis.

Old sediment. Old sediments were collected with transparent plexiglass tubes (Ø 60 mm), like described by Zehnder et al., (1980). The tubes had 5 mm diameter perforations every 2 cm, allowing for taking subsamples anoxically with a syringue. These perforations were covered with adhesive tape during sampling and transport to the laboratory.

Gas. The gas collectors - adopted from Magruder, (1984) - consisted of an inverted plastic funnel (aerea 0.045 m^2) connected to an inverted spheric glass vessel (550 ml), provided with openings for purging of air during positioning and for purging of water during sampling. The glass cover was covered with aluminium foil to prevent photosynthesis. The whole system was fixed to a vertical stand placed on the pond bottom.
In the first pond two collectors were left near each other for seven 3 to 4 day periods. One collector had the funnel placed on the sediment layer (closed collector) ; the other had the funnel at about 20 cm above the sludge layer (open collector). Two identical pairs of collectors were left for six 1 week periods in the fourth pond.

Digestion in the laboratory

Fresh sediment and suspended solids. Digestibility has been tested in the laboratory in 300 ml serum vials filled with 200 mg (dry solids) of fresh sediment or pond inflow suspended solids (concentrated by centrifuging at 3 000 g during 20 mn). The vials were flushed with a N_2/CO_2 mixture in order to remove oxygen and to maintain a constant pH and also to remove pore water methane (Kiene et al., 1985). They have been incubated at 20°C in dark. Digestion was followed by periodic CH_4 determination in the head-space gas.

Core segments. Known quantities of sediment of the cores were transferred to 60 ml serum vials under N_2 flushing. They were incubated at 16°C ; the sediment temperature in the ponds. Methane production was followed like described above.

Analysis
Sediment dry solids were determined directly on the samples and inflow total suspended solids after filtering (Millipore prefilter, AP20). Volatile solids were determined after ignition at 55°C during one hour.
Gas composition has been determined with a GIRDEL 30 gaschromatograph.

RESULTS AND DISCUSSION

Sedimentation
Dry solids deposition rates have been calculated from the sludge quantities collected during six one week periods (Table 1). The mean deposition rates are 52 g DS/(m^2. day) and 19 g DS/(m^2. day) for respectively the first and the fourth pond.
When compared to the loading rates which can be calculated from the inflow and the outflow data of both ponds (Table 2), their seems to be some internal dry solids production in the ponds which exceeds 6x to 5x their inflow.

TABLE 1 Dry solids deposition rates in g DS/(m^2. day)

Collector		27/5	3/6	10/6	16/6	24/6	30/6	Mean
1st pond	1	51	40	33	49	42	61	50
	2	56	36	39	53	25	46	53
	3	35	41	122	69	16	34	53
4th pond	4	18	17	32	15	19	10	19
	5	15	15	14	51	10	9	19

TABLE 2 Estimation of the internal dry solids production (g DS/(m^2. day)

	1st Pond	4th Pond	
Surface area	2 156	2 855	m^2
Inflow	120	103	m^3/day
(TSS)	157	97	g DS/m^3
Loading rate	8,7	3,5	g DS/(m^2. day)
Outflow	114	95	m^3/day
(TSS)	200	62	g DS/m^3
Loading rate	- 10,6	- 2,1	g DS/(m^2. day)
Measured deposition rate	52	19	g DS/(m^2. day)
Internal production rate	53,9	17,6	g DS/(m^2. day)

The algae production has not been measured in the field, but the potential algae production has been estimated by the formule proposed by Oswald et al., (1957) :

$$W = \frac{\Lambda \times 10^4 \times 0,06}{6.10^3}$$

where :

W	:	algae production (g DS/(m^2. day)
Λ	:	solar radiation received (cal/(cm^2. day))
10^4	:	cm^2/m^2
0,06	:	efficiency of solar energy utilization
6.10^3	:	calories required for synthesis of 1 g DS

Local mean intensity of solar radiation was 480 cal/(cm^2. day) during June. Thus potential algae production was 48 g DS/(cm^2. day), which is a same order of magnitude as the estimated internal production in the first pond, but much higher than the one in the fourth pond where, a priori, the algae production is the most important. So their might be an other mechanism responsible for the observation of an internal production.

Digestion

Gas production rates and methane production rates have been calculated from the volumes and the compositions of the gases collected during seven halfweek periods for the first pond and five one week periods for the last pond (Table 3).

The composition of the gas is like reported by Brockett (1975). CO_2 concentration is low due to the dissolution in the overlaying water. N_2 concentrations observed do not reflect a denitrification which is considered a minor phenomenon by several authors (Pano et al., 1982, Ferrara et al., 1982, and Reed, 1985) but simply is a consequence of diffusion of water dissolved N_2 to the collected gas.

TABLE 3 Measured gas production, gas composition and calculated CH_4 production rate and volatile solids loss rate

	1st Pond		4th Pond		
Gas collector	closed	open	closed	open	
Gas prod. rate	3 100	3 400	720	990	ml/(m^2. day)
Gas composition					
% CH_4	75	80	77	73	
% CO_2	8	4	3	2	
% N_2	17	16	20	25	
CH_4 prod. rate	2 325	2 720	577	723	ml/(m^2. day)
VS loss rate	41	48	10	1,3	g VS/(m^2. day)

The gas quantities collected look to depend on the position of the gas collector funnel : on or above the sludge layer. In the fourth pond only one time out of five, the collector placed above the sludge layer collected less gas than the collector on the sludge. This difference was less evident in the first pond. Their may be two not exclusive explanations for this phenomenon.
First, the N_2 collected does not enter the vessels by bubbling (like CH_4) but by diffusion through the water/gas interface in the vessel. This was demonstrated in the case of clogging of the tubes between funnel and vessel : CH_4 collection was severely reduced but N_2 transfer continued resulting in N_2 concentrations of 53%. Diffusion of N_2 now can be more important for open collectors than for closed ones.
Secondly the high position of the collector funnel allows dry solids to continue to settle under the funnel and they will be partly transformed in methane. The low funnel however prevents arrival of new matter during the sampling period.
The methane production rates have been converted to volatile solids loss rates according to : 1 g COD = 375 ml CH_4 (20°C, 1 atm) and 1 g VS = 1,5 g COD. The results figure in Table 3.

Sedimentation/digestion balance
A short term sedimentation/digestion balance can be established on basis of the collected data. The calculated dry solids deposition rate represents the sedimented dry solids minus the losses due to the digestion of a fraction of them during the sampling period. The volatile solids losses calculated on basis of the data of the high placed gas collectors represent the losses due to the digestion (1) of sediments present before the placing of the collector and (2) of a part of the fresh sediment deposited during the sampling period. Thus, to avoid double counting of the losses of a part of the fresh sediment the data from the low placed collectors has to be taken into account. The results of the calculation figure in Table 4.

TABLE 4 Calculation of net accumulation of dry solids as a result of sedimentation and digestion

	1st Pond	4th Pond	
Deposition rate	52	19	g DS/(m^2. day)
Loss rate	4,1	1	g VS/(m^2. day)
Net accumulation rate	48	18	g DS/(m^2. day)
Mean ultimate density *	160	240	kg/m^3
Sludge depth increase	11	2,7	cm/year

* Data from Carré et al. (1986).

The calculated mean yearly sludge depth increase rates are 11 cm/year and 2,7 cm/year for the first and the fourth pond. These values are much higher than those found by Carré et al. (1986) for the same ponds on basis of bathymetric measurements after nine years of operation : 1,7 and 1,8 cm/year.
Schneiter et al. (1983) measured sludge productions of 0,27 m^3/(1000 i.e. day) and 0,69 m^3/(1000 i.e. day) in the first cells of multiple cell facultative ponds. In our study we estimated a production of 0,65 m^3/(1 000 i.e. day) in the first pond which is high considering the presence of an Imhoff tank.
Two facts could explain the apparent overestimation of the sludge accumulation : an underestimation of the methane production rate and an overestimation of the deposition rate. The underestimation of the methane production rate is highly improbable as the summer data was used for the estimation of the yearly accumulation. The overestimation of the deposition rate can be due to an experimental artefact : not only inflow suspended solids and algae are retained in the collectors but also resuspended sediment. In order to check this last hypothesis some supplemental experiments were realized.

The origine of the retained sediments.

Methane production in core segments. The methane production measured in incubated core segments clearly shows the high activity of the upper layers containing solids with a relatively high VS/DS ratio. A typical example is shown in Table 5 and Figure 1, indicating that about 75% of the methane is produced (in summer) in the upper 4 cm.

TABLE 5 Methane production in core segments of the first pond sludge layer

Level (cm)	g DS/kg	g VS/kg	% VS/DS	ml CH_4/(g VS. day)	ml CH_4/(kg. day)
0-4	37	18	49	5,7	103
	55	23	42	6,3	145
4-6	104	23	22	1,0	23
	79	26	33	1,3	34
8-12	95	20	21	0,4	8
	101	26	26	0,4	10
12-16	166	16	10	n.d.	n.d.
	163	20	12	n.d.	n.d.
16-18	298	15	5	n.d.	n.d.
	241	12	5	n.d.	n.d.

n.d. : not detected.

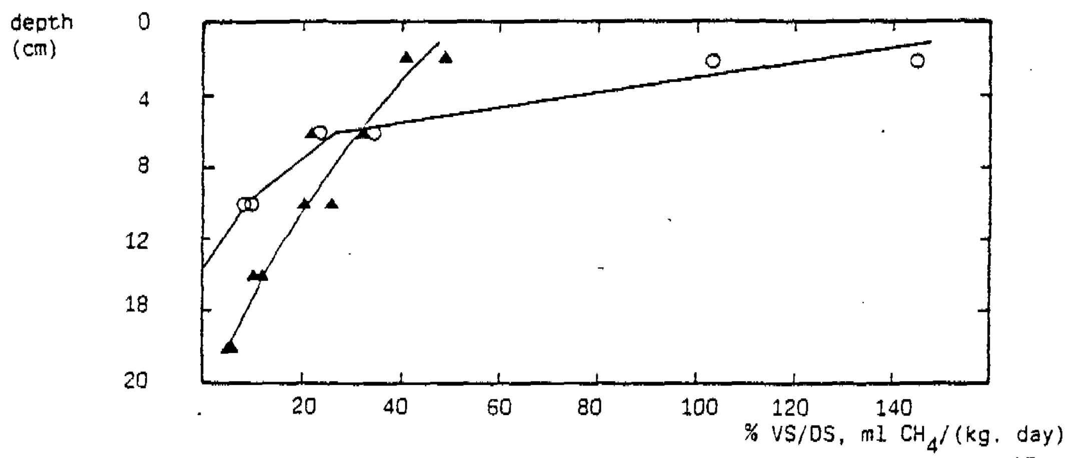

Fig. 1. Methane production (o) as ml CH_4/(kg sludge. day) and VS/DS ratio (▲) in the first pond.

Methane production from fresh sediments in ponds. Gas collectors were placed on sludge collectors in both ponds and methane production was measured after one week. The measured methane production was more than 2 700 ml $CH_4/(m^2$. day) in the first pond and 520 ml $CH_4/(m^2$. day) in the fourth pond. Thus methane production rates on the sludge collectors were of the same order of magnitude as those measured on the pond bottoms, indicating the fresh sediment is actively participating in methane production, even when no contact between pond bottom and fresh sediment exists. This seems surprising, specially in the case of the fourth pond where fresh sediment mainly would consist of dead algae.

Methane production from inflow suspended solids and fresh sediments in the laboratory. Inflow suspended solids of both ponds were concentrated by centrifuging and incubated at 20°C. Freshly collected sediment was incubated in the same conditions. No methanogenic inoculum was added.
The freshly collected sediment showed a methane production (Figure 2), whereas no methane was detected from the inflow suspended solids incubations. Thus the collected sediment is well seeded with methanogens - gas production starts immediately after incubation - and inflow suspended solids are not. So fresh sediment solids and inflow suspended solids would not be identical. This is also indicated by the VS/DS ratio of both solids (Table 6).

TABLE 6 Comparison of VS/DS ratio of fresh sediment and inflow suspended solids

	1st Pond	4th Pond
Fresh sediment	59%	55%
Inflow suspended solids	77%	62%

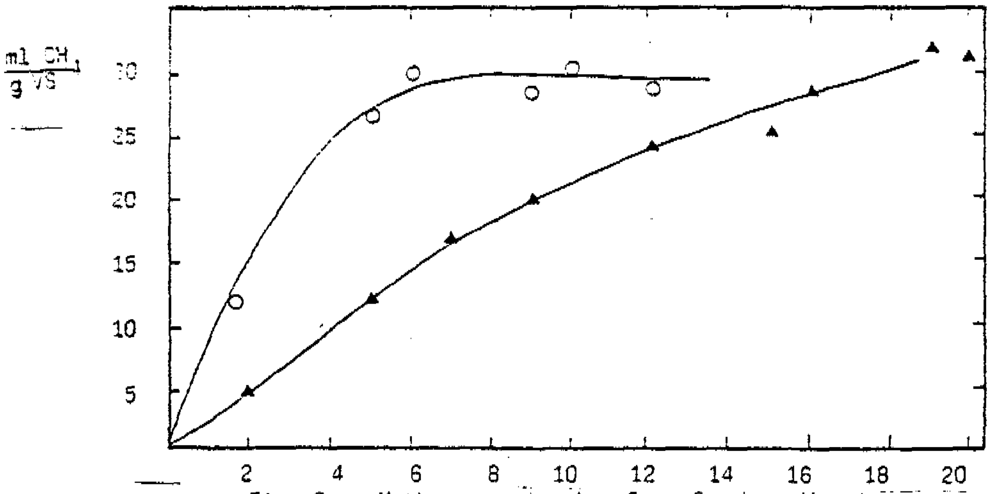

Fig. 2. Methane production from fresh sediments in the laboratory (▲ first pond, o fourth pond).

Proposed mechanism of sediment collection

According to the described observations the following mechanism of sediment collection can be proposed. The gas production in the pond bottom creates a continous motion of resuspension and resedimentation of particles. When these particles reach the sludge collector, their rebound is interrupted by the screen the collector forms against rising gas bubbles. As a consequence a net transport of sediment particles takes place resulting in an overestimation of the real sedimentation.

Mechanism verification by winter observations

The gas production is considerably lower in winter and as a consequence the resuspension of settled solids should be less than in summer. In order to check this sediment and gas collectors have been placed in both ponds during the cold season.
Collected data are represented in Table 7 and Table 8.

TABLE 7 Deposition rates in winter as g $DS/(m^2.\ day)$

Period	1st Pond	4th Pond
5/1-20/2 (ice cover)	5,3	1,5
20/2-27/2	14,3	6,1
27/2-9/3	29,3	6,9
Summer mean	52	19

The measured deposition rates are low compared to the summer data, especially during the period with ice cover. From the inflow and outflow DS-concentrations of the first pond a net deposition rate of 4,6 g $DS/(m^2.\ day)$ could be calculated for this period. Inflow and outflow DS-concentrations were equal in the fourth pond.

TABLE 8 Methane production rates in winter as ml $CH_4/(m^2.\ day)$

Period	1st Pond		4th Pond	
	Bottom	Sediment ollector	Bottom	Sediment ollector
20/2-27/2	166	96	14	1
27/2-9/3	348	80	42	5
Summer mean	2 720	2 700 *	723	520 *

* one value only.

The gas production rates are much lower than in summer indeed, but, more interestingly, there is now an important difference between the gas production rates from the pond bottom and those from the sediment collector. This supports the hypothesis on the collection mechanism like illustrated in Figure 3. In summer the vigorous gas production removes much sediment to the collector where digestion continues, rapidly approaching the gas production on the bottom. In winter the low gas production deplaces only little sediment to the collector (especially in the fourth pond where gas production is very low) and as a consequence the gas production in the collector remains far below the bottom gas production.

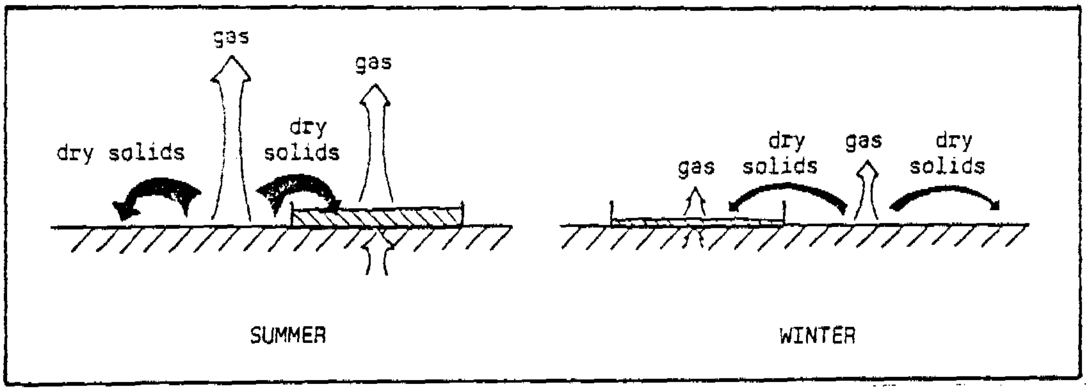

Fig. 3. Schematic representation of the mechanism of solids collection on a pond bottom.

CONCLUSION

The attempt to establish a short term sedimentation/digestion balance on pond bottoms by the in situ measurements of deposition and gas production rates failed. No problems were met with gas collection. However deposition rates were largely overestimated due to a net transport to the collectors of bottom sediment resuspended by gas bubbling. This net transport could not be quantified.

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A SIMULATION OF BENTHAL STABILIZATION

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ABSTRACT

An existing benthal model has been modified and calibrated for the description of the physical and biological processes in consolidating organic deposits of both municipal and papermill origin. Temperature sensitivities, phosphorus reactions, and flexibility as to solids identification were added to the model. Model predictions of organic stabilization, direct oxygen uptake, and soluble feedback of nitrogen, phosphorus, and chemical oxygen demand have been compared to data collected from both laboratory and field operations. Phosphorus distribution was controlled by advection and biological uptake in all cases. Nitrogen feedback varied in a complex manner and was difficult to simulate.

KEYWORDS

Benthal feedback, deposit simulation, benthal stabilization, nitrogen, phosphorus, papermill wastes, cellulose, wastewater lagoon, nutrients.

INTRODUCTION

The benthal stabilization of wastes in an aerated stabilization basin (ASB) can exert a major effect on the effluent water quality. An effective model of a wastewater stabilization pond must include an adequate description of the loading to and the feedback from the benthal must include an adequate description of the loading to and the feedback from the benthal zone. A major portion of the influent oxygen demand can be stabilized in the benthal deposit, and a major fraction of the nutrient demand in the overwater can be provided through benthal feedback.

Previous research has found that increased nitrogen feedback was correlated to a high nitrogen content of the benthal solids (Chiaro and Burke, 1980), increasing deposit depth (Fair et al., 1941), and overlying dissolved oxygen (DO) below 1.5 mg/l (Fillos and Molof, 1972). Increased phosphorus feedback was correlated to increasing temperature (Holdren and Armstrong, 1980), high sediment oxygen demand (Chiaro and Burke, 1980), overlying DO below 1.5 mg/l (Fillos and Molof, 1972), anoxic conditions (Kamp-Nielsen, 1974, 1975), solubility parameters (Avnimelech et al., 1983), and was independent of pH (Kamp-Nielsen, 1974, 1975), the overlying phosphorus level (Theis and McCabe, 1978) and interstitial phosphorus concentration of the deep deposit layers (van Raaphorst and Brinkman, 1984). In active deposits the majority of phosphorus has been in biomass, and a significant amount may reside as precipitates of calcium, aluminum, and iron (Stumm and Stumm-Zollinger, 1972). Reported bacterial phosphorus content has ranged from 1.5 to 3.8% by weight (Grady and Lim, 1980). Once solubilized, the conversion of organic phosphorus to orthophosphate has been rapid, relative to other phosphorus reactions (Houng and Gloyna, 1984; Ferrara and Harleman, 1980; Fritz et al., 1979).

Several lagoon models have incorporated some benthal feedback component (Houng and Gloyna, 1984; Ferrara and Harleman, 1980; Fritz et al., 1979; Marais, 1970). Bryant (1983) developed and validated a benthal model specifically to simulate the physical processes of consolidation, diffusion, and gas production. Biological degradation processes of solubilization, oxidation, nitrification, denitrification, fermentation, and methanogenesis were modeled as well. The model calculated the carbon, nitrogen, and particulates within layers of the deposit for both batch and continual loading rates (Bryant and Rich, 1984). Over the last three years the baseline benthal model has been altered by the addition of phosphorus and cellulose reactions, temperature sensitivity coefficients, and a modified model of oxygen diffusion. The resulting benthal model predicts cumulative feedback components, time-dependent interstitial profiles, and time-dependent solids fractions for deposits of any specified temperature, loading rate, and cellulose/biomass ratio. This paper reports the specifics of the model modifications and the results of validation comparisons.

MODEL DEVELOPMENT

A simple flowchart for the simulation is shown in Figure 1. A first-order dynamic consolidation equation and an exponential steady-state porosity relationship were evaluated in each time increment to define the change in thickness of discrete benthal layers and the resulting transfer of interstitial fluid in the deposit. A set of general one-dimensional transport equations for eleven soluble components was solved by the State Variable analog

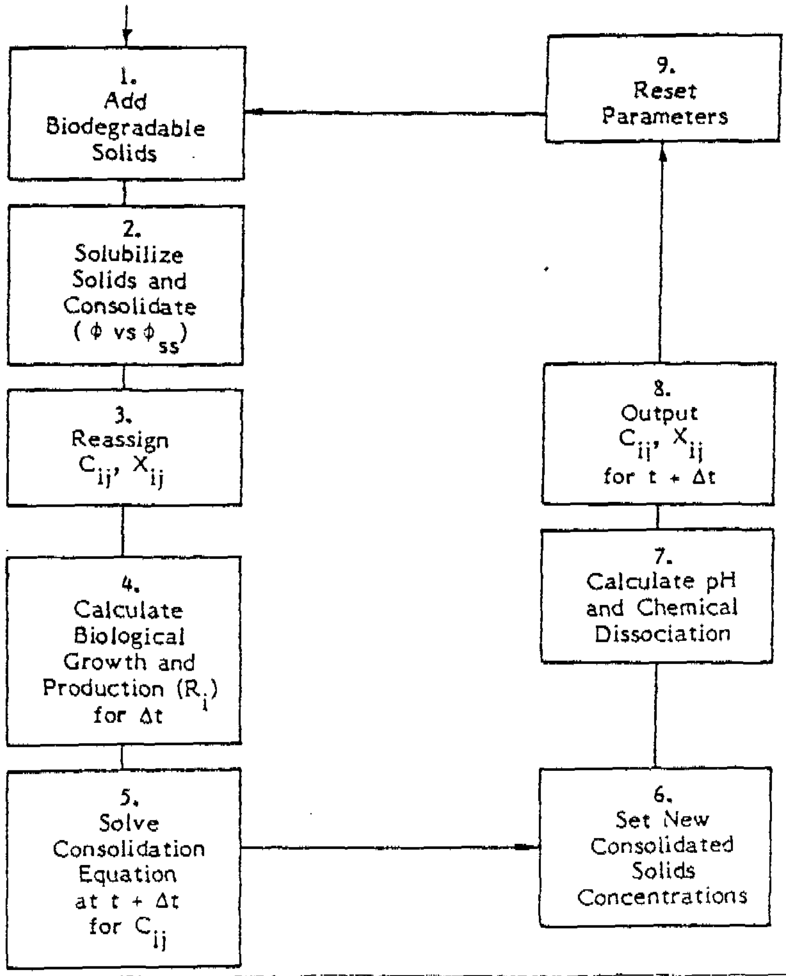

Figure 1. Numerical Solution Flowchart.

for the discrete benthal layers in each time increment. Eight categories of solids were also inventoried by the simulation. Model output consisted of predicted depth profiles of interstitial components as a function of time. The details of these basic elements of the benthal model have been reported earlier (Bryant and Rich, 1984; Bryant, 1983).

The model of phosphorus transformations, based upon reports of previous research, included solubilization and mineralization of organic phosphorus as well as biological uptake and precipitation of orthophosphate. Because of the low clay content of ASB deposits, adsorption was not included in the model. The rate of solubilization was assumed to be equal to that of biomass, and mineralization was assumed to be sufficiently rapid that organic phosphorus could be ignored as an intermediate (Houng and Gloyna, 1984; Ferrara and Harleman, 1980; Fritz et al.,1979). Biological uptake of orthophosphate was based on a constant stoichiometry of $C_5H_7O_2NP_{0.143}$. Phosphorus precipitation was assumed to be controlled by iron and calcium only. As an initial approximation, the relationship presented by Stumm and Morgan (1970) was programmed as a four-stage piecewise model. The model was modified next to include cellulose degradation at a rate comparable to that of biomass (O'Rourke, 1968). Also, temperature sensitivities were incorporated for all reaction rates, based upon existing research results. Because some components of papermill wastewater have been found resistant to degradation or likely to degrade to resistant products (Levenberger et al., 1985), the anaerobic fermentation rate constant was reduced to 1.0 d^{-1} for papermill simulations. The list of reactions rate constants and temperature sensitivities in the benthal model are listed in the Table 1.

TABLE 1. Rate Constants and Temperature Coefficients of Biological Reactions in the Benthal Model

REACTION	$k(d^{-1})$	INITIAL θ	MODIFIED θ <20°	MODIFIED θ >20°
AEROBIC:				
Solubilization	0.08	1.050	1.150	1.050
Organic oxidation	0.39	1.047		
Acetate oxidation	0.39	1.047		
Propionate oxidation	0.39	1.047		
Nitrification	44.00	1.103		
Cellulose oxidation	0.08	1.047		
ANAEROBIC:				
Solubilization	0.02	1.030	1.400	1.100
Fermentation	1.00	1.020	1.020	0.800
Lipid fermentation	4.00	1.400		
HAc reduction	4.00	1.400		1.400
CO_2 reduction	11.00	1.050		
HPr fermentation	4.00	1.400		
Denitrification	40.00	1.150		
Cellulose fermentation	0.10	1.000		

The model of oxygen transport into the deposit was changed to the form proposed by Klapwijk and Snodgrass (1983)

$$ZOX^2 = 2\ (D)(C_{ov})/(DC1/DT) \qquad (1)$$

where

ZOX = depth of oxypause from deposit interface, cm,
D = oxygen diffusion coefficient, cm^2/day,
C_{ov} = DO of overlying, mmol/l,
$DC1$ = potential DO demand in each time increment, mmol/l,
DT = time increment, days.

Aerobic and anaerobic reactions were assigned proportionally within the benthal layer containing the calculated oxypause depth. Despite the incorporation of Equation (1), the benthal model continued to provide very low predictions of direct oxygen uptake and efforts continue to resolve this problem.

Nutrient limitation of the listed reactions was incorporated into the model. All demands upon a given reactant in each time increment were calculated and compared to the existing concentration plus simultaneous production. If the reactant supply was insufficient, all reactions involving the reactant were reduced by a proportionality constant. All reactants were evaluated by an iterative approach for each time increment. This logic provided two desirable aspects: the avoidance of an artificial sequence of reaction priorities for demand upon a reactant pool, and the ability to sustain reactions with an apparent zero concentration of any reactant.

LABORATORY EXPERIMENTS

Consolidation rates and porosity profiles for the three deposit types were calibrated by short-term batch settling tests. The benthal model predictions were then compared with the measured interstitial profiles from a set of ten benthal deposits developed in the laboratory. The experimental design varied the sludge type, temperature, and initial deposit volume, as indicated by the deposit descriptions in Table 2. The papermill deposits were composites of lagoon influent wastes mixed with a biomass component from a Kraft mill activated sludge plant. The ratio of biomass to nonbiomass concentrations was determined from prior analysis of benthal samples from the two Kraft mill ASB systems. To develop benthal profiles for model verification, interstitial samples were taken twice in the first week, once per week for four weeks, and once biweekly thereafter. All interstitial samples were analyzed for pH and soluble organic carbon (TOC). Ammonia was measured for profiles of both Mill A deposits, one Mill B deposit, and one deposit of municipal waste activated sludge (WAS). Orthophosphates were determined only for the Mill A profiles. Initial and final solids properties were measured to support mass balances for carbon, nitrogen, and phosphorus. A detailed description of the soluble feedback from these deposits has been reported separately (Bryant and Knutsen, 1986).

RESULTS

The calibrated benthal model was used to simulate the ten experimental conditions. The simulation results were compared to laboratory results with regard to overall deposit consolidation, time-dependent interstitial profiles of pH, TOC, and ammonia, and cumulative deposit feedback components. Both the areas of model success and failure were instructive in explaining benthal mechanism interactions. A comparison of simulated and measured values for deposit consolidation and interstitial concentrations, summarized in Tables 2 and 3, was rather successful, considering the range of experimental variables evaluated. The consolidation results matched well for all deposits. A short-term batch settling test was used to calibrate the coefficients of the first-order consolidation equation in the model (Bryant and Rich, 1984), and the results indicate that this simple test provided sufficient data for the benthal simulation. Temperature sensitivities were modified as shown in Table 1 for simulations above or below 20°C. In particular, solubilization reactions were apparently much more sensitive to low temperature than high temperature. Even with these changes, simulation results were less effective at the temperature extremes.

Benthal phosphorus profiles were measured only for the Mill A deposits at 20°, but simulated values closely mated these profiles at all times. However, that simulation match, as well as the results in Tables 2 and 3, could only be achieved after deactivating the precipitation model, discussed above. Thus, advection and biological uptake were apparently the mechanisms controlling the interstitial phosphorus concentrations in these rapidly consolidating, organic deposits.

Measured feedback components have been discussed in detail separately (Bryant and Knutsen, 1986). It should be noted, however, that the C:N:P ratio of benthal feedback varied significantly as a function of the experimental variables, and that the pattern of change was not amenable to simple explanation. Simulated benthal feedback of carbon, nitrogen, and phosphorus was compared to the measured values from the experimental deposits, as shown in Table 4. Consistently low estimates of direct oxygen uptake were resolved in later work by increasing the effective dispersion coefficient at the benthal interface. The 20°C papermill deposit simulations were the most successful predictions of benthal feedback, and complex changes in the nutrient feedback with temperature is the focus of further study.

TABLE 2. Cumulative Measured (and Simulated) Benthal Consolidation Rates and pH Concentrations

DEPOSIT Source-Temp.-Vol.	CONSOLIDATION RATE (cm/day)	INTERSTITIAL pH Day 24	Day 53	Day 94
Mill A-20°-9.75 L	0.119,0.091	8.7-10.8	6.8-8.4	5.9-6.4
	(0.131)	(8.9- 9.8)	(6.4-9.5)	(6.2-7.8)
Mill B-20°-9.75 L	0.040,0.085	8.6-8.6	5.7-7.9	6.1-7.2
	(0.099)	(7.0-7.3)	(5.8-6.3)	(5.7-7.2)
Munic.-20°-9.75 L	0.080,0.085	7.5-8.5	7.1-8.5	7.4-7.6
	(0.110)	(6.3-7.0)	(6.1-6.7)	(6.3-6.7)
Mill A-26°-16.25 L	0.159	7.6-10.3	7.7-8.1	6.1-6.3
	(0.242)	(7.4- 9.8)	(6.4-7.8)	(6.1-9.4)
Mill A-26°-3.25 L	0.063	7.6-8.6	--	5.5-7.5
	(0.044)	(7.2-9.8)	--	(7.4-7.9)
Mill A-15°-16.25 L	0.108	7.8- 8.2	6.8-8.4	6.4- 6.9
	(0.145)	(9.5-10.1)	(9.6-9.8)	(9.6-10.0)
Mill A-15°-3.25 L	0.051	7.8-8.6	8.3-8.4	8.5-8.9
	(0.025)	(7.7-9.7)	(9.7-9.9)	(9.3-9.9)

TABLE 3. Cumulative Measured (and Simulated) Benthal TOC and Ammonia Concentrations

DEPOSIT	INTERSTITIAL TOC (mgC/l) Day 24	Day 53	Day 94	INTERSTITIAL AMMONIA (mgN/l) Day 24	Day 53	Day 94
A-20°	25-210	957-1420	830-2050	0.0-8.8	0.0	0.0
	(146)	(1130)	(363)	(5.8)	(0.4)	(0.8)
B-20°	119-1630	753-2076	43-2340	0.0-8.4	0.0	0.0
	(501)	(1188)	(1609)	(0.7)	(0.2)	(0.4)
M-20°	41-124	5-93	14-95	16-103	0-255	53-227
	(124)	(61)	(6)	(63)	(99)	(139)
A-26°-16	346-415	1690-2060	1860-2420			
	(619)	(1190)	(1280)			
A-26°-3	96-1060	--	3-313			
	(961)	--	(1530)			
A-15°-16	500-891	1110-1360	706-1780			
	(77)	(297)	(524)			
A-15°-3	79-105	427-523	718-886			
	(581)	(1030)	(1436)			

SUMMARY

Benthal feedback of soluble carbon and nutrients varied significantly with temperature, sludge type, and deposit volume. Such changes can be extremely important to the effluent water quality of an ASB and to the performance of intermediate and later cells within the ASB system. The measured benthal feedback changes were sufficiently complex to require a relatively detailed model for simulation. A benthal model which included first-order consolidation and biological equations adequately addressed many of the observed trends at moderate temperature. Phosphorus uptake was well predicted under all conditions by biological uptake alone. However, nitrogen feedback trends and some carbon feedback trends at 15° and 26°C were not well simulated by the model. The steady-state predictions of the model for continual deposition conditions have been used with the DYLAMO model to very accurately simulate the performance of operating Kraft mill ASBs.

TABLE 4. Cumulative Measured and Simulated Benthal Oxygen Demand and Feedback Measurements for 112 Days

Deposit	D	F_c	F_n	M	P
Source-Temp. Vol.	gO_2/m^2	gO_2/m^2	gO_2/m^2	gO_2/m^2	mgP/m^2
Mill A - 20° - 9.75L	239.7	824.29	52.800	0.055	608.8
Mill A - 20° - 9.75L	207.2	1290.00	NM	1.093	609.8
Simulation	1.4	427.00	0.690	0.073	1610.0
Munic. - 20° - 9.75L	347.6	686.10	377.175	17.080	10858.0
Munic. - 20° - 9.75L	317.4	747.40	NM	18.830	16418.0
Simulation	9.7	16.20	62.500	0.020	1560.0
Mill B - 20° - 9.75L	251.0	800.50	15.488	5.397	214.0
Mill B - 20° - 9.75L	241.0	729.10	NM	4.748	732.9
Simulation	1.7	855.00	0.260	0.018	1050.0
Mill A - 26° - 16.25L	392.2	1262.00	12.475	2.259	1498.0
Simulation	2.0	1105.00	1.100	0.081	2090.0
Mill A - 26° - 3.25L	304.0	973.40	NM	1.379	315.9
Simulation	2.8	1294.00	1.820	0.018	4820.0
Mill A - 15° - 16.25L	63.5	704.40	0.027	0	1774.0
Simulation	0.9	152.00	0.240	0.012	277.0
Mill A - 15° - 3.25L	214.2	537.80	NM	0	122.0
Simulation	0.8	436.00	0.440	0.012	301.0

D = direct benthal oxygen uptake,
F_c = soluble carbonaceous feedback, as COD,
F_n = soluble nitrogeneous feedback, as ammonia nitrogen,
M = methane production as oxygen demand bypassed,
P = soluble phosphorous feedback, as orthophosphate,

The benthal model is being used currently to simulate the benthal feedback input to the DYLAMO simulation model of ASBs (Sackellares et al., 1986). Successful simulation of an operating Kraft mill ASB wastewater has been obtained and results reported. Typical predictions of the steady-state feedback rates are shown in Table 5. Such predictions can be generated as a coupled input for the modeling of processes in any overlying water column.

Table 5. Estimated ASB Steady-state Feedback Rates for Continual Deposition at Mill A.

CONDITION	D	F_c	F_n	P
	$gO_2/m^2/d$	$gO_2/m^2/d$	$gO_2/m^2/d$	$mgP/m^2/d$
Baseline	0.09	3.36	2.57	0.71
Low (1%) cellulose	0.15	9.00	4.79	1.21
High overwater DO	0.21	2.93	1.71	0.43
High interface mixing	1.00	3.30	1.90	0.54
Half VSS loading	0.19	2.90	2.60	0.50

Note: All simulations were for VSS loading of 126 $g/m^2/day$ and 20°C.

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EFFECTS OF MATURATION ON THE CHARACTERISTICS OF WASTEWATER STABILIZATION POND SLUDGES

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ABSTRACT

The physical-chemical characteristics (pH, eH, S^{2-}, TOC, NH_4^+, particulate Kjeldahl nitrogen (PKN), total phosphorus (TP) and PO_4^{3-}) of the sludge, accumulated in the first of a series of wastewater stabilization ponds, have been studied on two vertical profiles in order to characterize the effects of the maturation of the sludge. The effects on the concentration of faecal indicator organisms and salmonella have also been determined.
The maturation leads to an increased density of the sludge and to the removal of about 80% of the TOC, 70% of PN and of 75% of TP initially present in the fresh sludge.
The concentrations of sulfate reducing bacteria do not change and those of the total coliforms, the faecal coliforms and the faecal streptococci are only moderately modified, as the maximum decrease is 4 log. Salmonellas only are found in the superficial sediment layer, which might indicate their rapid inactivation.

KEYWORDS

Wastewater stabilization ponds, sludge, maturation, TOC, nitrogen, phosphorus, faecal indicator organisms, Salmonella.

INTRODUCTION

The sludge accumulated in the wastewater stabilization ponds contribute to the global treatment process by the anaerobic digestion of sedimented organic matter and by the trapping of undegradable solids. The digestion of the organic matter takes place rapidly as far as the easily hydrolysable fraction is concerned (Bryant et al., 1984). The digestion however of the slowly hydrolysable solids still goes on in the deeper layers untill the sludge is removed by desludging.

Little data about maturation phenomena in wastewater stabilization pond sludge is available in literature. However the knowledge of these phenomena, which will affect the overlaying water quality (Fillos et al., 1972) and of the final characteristics of the sludge seems interesting.

The aim of this work was to study the physical-chemical and microbiological evolution of the sludge accumulated in the first pond of a wastewater stabilization pond. It was supposed this evolution could be apprehended by the study of the vertical profiles.

MATERIAL AND METHODS

Location of the study
The sludge studied is from the first pond of the wastewater stabilization ponds of La

Chapelle Thouarault (France) which have been described earlier (Carré et al., 1986).

Sampling

The samples has been taken in summer in two points situated in the zones where the sludge accumulation was most important : near the inlet and near the outlet of the pond. Considering the respective positions of these points, the difference between the sludges would be maximal.

The settling solids have been collected with traps like described elsewhere (Iwema, 1987). Five samples have been taken in each point.

Vertical profiles have been constituted according to the method described by Parker et al., (1968). Six subsamples have been taken on each profile.

The samples have been stored at 4°C before analysis which were realized within 12 hours. A fraction of the samples has been centrifuged immediately after arrival at the laboratory in order to collect the interstitial phase (10 000 g, 30 mn).

Physical-chemical analysis

The measurements of pH (according to standard NF T 90-008) and of eH (platinum electrode) have been realized in situ. Sulfide has been determined by indirect iodometric titration according to Roger (1986). Dry solids (DS) were determined by drying at 105°C (NF U 44-171). TOC was determined after combustion with an analyser type Carmograph. Dissolved ammonia nitrogen (DNH_4) has been determined in the interstitial phase after distillation (NF T 90-015) and particulate Kjeldahl nitrogen (PKN) according to standard NF T 90-010. Dissolved phosphorus (PO_4^{3-}) has been determined in the interstitial phase according to standard NF T 90-023 and total phosphorus (TP) after mineralisation of the samples with potassium persulfate.

Bacteriological analysis

The determinations of bacterial populations have been realized with the liquid media method using 3 tubes per dilution. Total coliforms have been isolated in lactose broth with bromocresol purple (30°C, 48h) and faecal coliforms have been determined on brilliant green lactose bile broth (44°C, 48h) ; from the positive lactose broth tubes, faecal streptococci have been isolated on azide dextrose broth (37°C, 48h) and confirmed on ethyl violet azide broth (37°C, 48h). Sulfate reducing bacterias have been isolated, after inactivation of vegetative forms (80°C, 5 mn), by incorporation in sulfite agar (46°C, 24h).
Salmonella have been isolated on modified Rappaport R 10 (44°C, 24h), confirmed by inoculation on Hektoen enteric agar (37°C, 24h) and identified with Api 20 E strips. The results are given in MPN per liter sludge (MPN/l).

RESULTS AND DISCUSSION

Physical-chemical characteristics

The results are given in Table 1 and Figure 1.

TABLE 1 Physical-chemical characteristics of sludge

Near inlet					Near outlet				
depth (cm)	DS %	eH mV	S^{2-} mg/gDS	pH	depth (cm)	DS %	eH mV	S^{2-} mg/gDS	pH
trap	4,7	-232	3,2	6,35	trap	3,2	-216	4,1	6,70
0-5	8,1	-230	3,3	6,42	0-3	3,9	-245	5,8	6,70
5-15	12,3	-230	3,3	6,55	3-8	6,1	-280	5,9	6,72
15-25	18,7	-235	5,0	6,55	8-20	10,1	-295	6,5	6,75
25-35	22,6	-250	5,3	6,62	20-35	12,1	-270	7,8	6,75
35-45	23,0	-280	5,9	6,62	35-50	20,7	-300	6,8	6,82
45-55	37,7	-290	5,5	6,70	50-60	30,1	-285	5,4	6,82

Dry solids. The dry solids content is low in the sludge collected in the traps, but somewhat higher near the inlet.
Dry solids content increases with depth and thus with the age of the sludge. After ten years the dry solids content has increased about tenfold. This increase is more marked near the inlet of the pond.
The nature and the dimensions of the the particles which settle in each point can explain the differences which exist between the profiles near the inlet and near the outlet. Indeed,

mineral lithogenic solids (clay, silt) carried with the sewage and organic anthropogenic solids with larger dimensions settle preferentially near the inlet, whereas fine organic particles, among which many algae - more than near the inlet as a result of the direction of the prevailing winds - settle near the outlet.

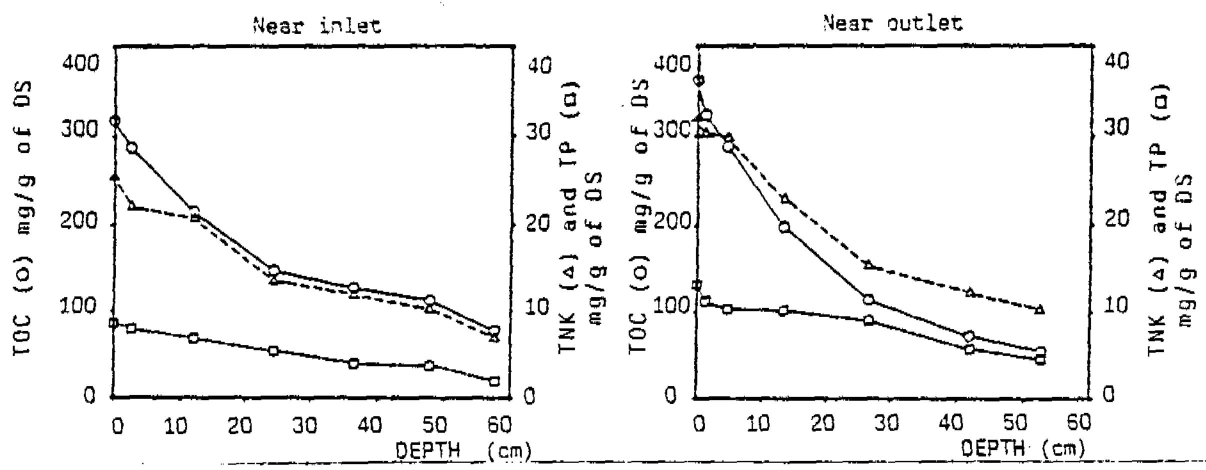

Fig. 1. Total organic carbon, total nitrogen and total phosphorus in core-samples

Total organic carbon. Total organic carbon content of the young sludge is very high at both points, and highest near the outlet. The profiles show an important decrease in TOC content with depth and thus with time, confirming the observations of Somiya et al., (1984). This decrease is more important near the outlet.

The deepest and thus the oldest sludges, which have residual TOC contents comparable to those of organic matter rich limnic and oceanic sediments (Henrichs et al., 1987) have lost 80% to 85% of their total carbon in 10 years.
The differences between the two profiles can be explained by the larger fraction of mineral lithogenic solids carried with the raw sewage and settled near the inlet and by more rapid decomposition of the organic solids near the outlet as a result of (1) the more important porosity of the sludge in that point, offering a larger surface for bacterial activity and (2) the algae ability to mineralize rapidly (Foree et al., 1970 and Dessery et al., 1984).

Oxydation-reduction potential. Near the inlet and near the outlet low eH values are observed in the sludges including the fresh sediment collected in the traps. A decrease of the eH with depth can be observed in both profiles. The intensification of the anaerobic conditions can result from the accumulation of reduced compounds.
In this kind of ponds where highly organic sediments accumulate, like found here and like also noticed by Gloyna (1971), the lower part of the water column which is in contact with the sediments, is not oxygenated. This prevents the formation of an oxydized layer in the water-sediment interface, layer which could limit the exchange between both compartments.

Sulfide. The sulfide concentrations in the sludges are important, even in the sludge collected in the traps and they increase with depth of both points. The sulfide concentrations are somewhat lower near the inlet, where the supply of mineral lithogenic solid is more important.
The values of the sulfide concentrations reflect the sulfidization of the environment linked to the prevailing redox conditions, -200 to -300 mV, the most favorable to sulfate reduction (Poduska et al., 1981).

pH. The measured pH values, slightly acid especially near the inlet like Vuillot et al., (1982) showed earlier, show a tendency to increase with depth and from the point near the inlet to the point near the outlet.
The pH increase with depth may be a result of the predominancy of methanogenesis accompagnied by the degradation of the initially produced volatile fatty acids. The higher pH of the sludge near the outlet can be explained by the nature of the compounds which accumulate there, that is antropogenous particles resuspended in the pond elsewehere and thus partially degraded and algae which decompose rapidly.

Particulate Kjeldahl nitrogen and dissolved ammonia nitrogen. The analysis (Figure 1) show the existence of high Kjeldahl nitrogen concentrations in the sludge-slightly higher near the outlet where the mineral lithogenic solids are less abundant - and which result from the organic nitrogen supplied with the raw sewage and from the sedimented algae.

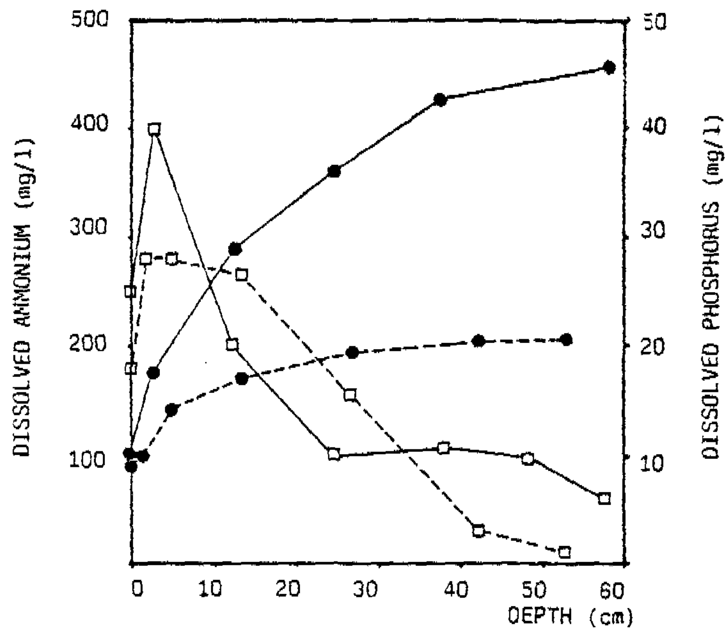

Fig. 2. Concentrations of dissolved ammonium (●) and dissolved phosphorus (□) in core-samples : — near pond inlet ;--- near pond outlet

The particulate Kjeldhal nitrogen concentrations decrease with depth, whereas the dissolved ammonia concentrations in the interstitialphase increase and become very important especially near the inlet (Figure 2). Like noticed by several authors (Murray et al., 1978, Rosenfeld, 1979), this evolution reflects the mineralization of the organic nitrogen which rate increases at long term (Dessery, 1984).The redox potentials in the sludge facilitate, on the other hand the solubilization of the ammonia (Delaune et al., 1981).

The difference between the particulate Kjeldahl nitrogen concentrations of the superficial sludge (the young sludge) and the deeper sludge (the old sludge) corresponds to the regeneration of about 70% of the initially present nitrogen.

The lower ammonia concentrations observed in the interstitial phase of the sludge near the outlet can result from the more rapid mineralization of the particulate nitrogen - due to the dimensions and the nature of the particles accumulated in this point - and from the easier transfer of the ammonia to the overlaying water - due to the higher porosity of the sediment in this point.

Total phosphorus and dissolved phosphorus. The results of the analysis show the total phosphorus concentration to be higher near the outlet (Figure 1). At both points one observes a decrease of the total phosphorus concentration with depth and the difference between the concentrations of the superficial and the deep sediments corresponds to the regeneration of 75% of the originally present phosphorus.

The phosphorus dissolved in the interstitial phase is especially abundant in the superficial layers (Figure 2) and its concentration decreases with depth, more sharply near the outlet than near the inlet. The important release of phosphorus from the sludge of a facultative pond has already been noticed by Gloyna (1984) and results mainly from the mineralization of the organic phosphorus which makes up two thirds of the total phosphorus in the studied sludge (Soulemane, 1985) and does not or hardly results from the reduction of inorganic phosphates (ferric phosphate) according to Singer (1972). Considering the anaerobic conditions in the entire sludge column which are favorable to solubilization of phosphorus, the high dissolved phosphorus concentration observed in the upper levels, shows that the release of the phosphorus mainly occurs in the young sludge and thus rapidly after the accumulation in the sediments - unlike the nitrogen - like recorded by several authors (Stumm et al., 1972 and Somiya, 1984).

<u>Microbiological characteristics of the sludge</u>. Figure 3 presents the levels of contamination of the sludge in the vertical profiles near the inlet and the outlet of the pond.

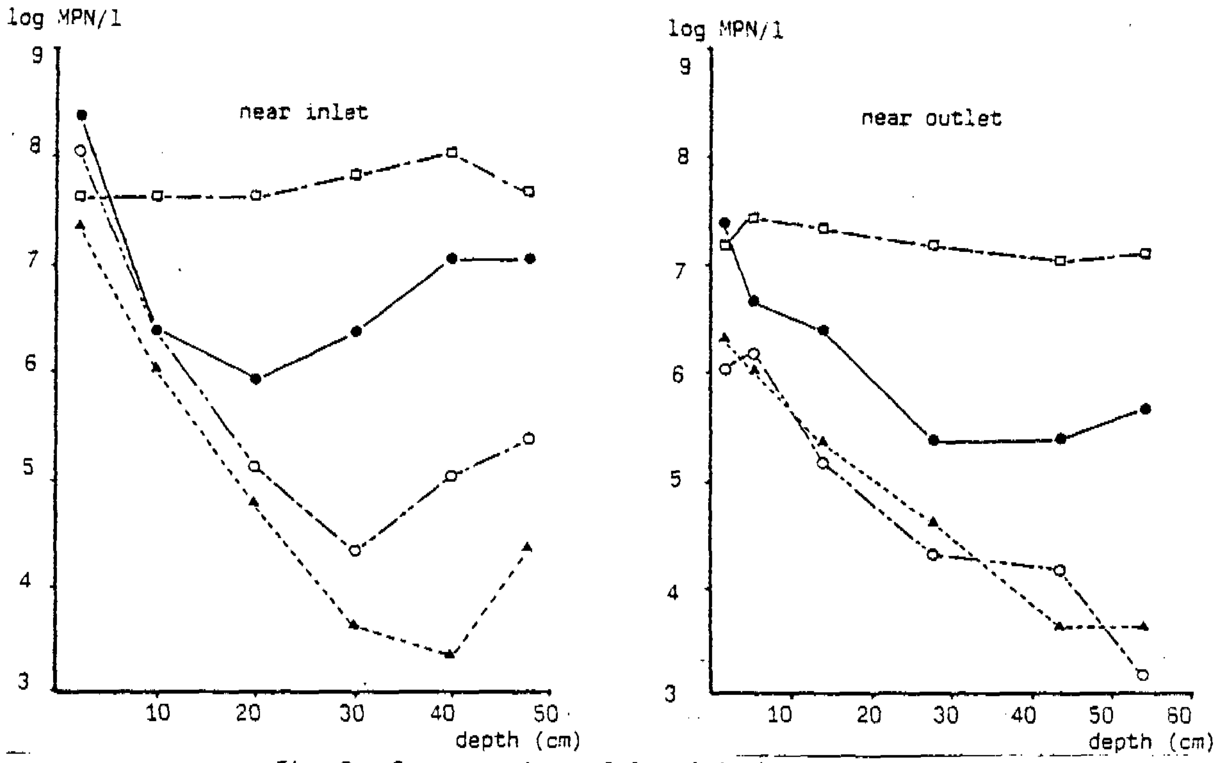

Fig. 3. Concentrations of faecal indicator organisms observed in the vertical profiles of the sludge near the inlet and near the outlet of the pond. ● Total coliforms (TC), ○ faecal coliforms (FC), ▲ faecal streptococci (FS), □ spores of sulfate reducing clostridia (SRC).

Near the inlet the superficial layer appears to be heavily contaminated with faecal indicator organisms (about 10^8 MPN/liter sludge). The concentrations of TC, of FC and of FS decrease with depth, reaching a minimum between 20 cm and 40 cm : 2.10^5 MPN/l TC at 20 cm, 2.10^4 MPN/l FC at 30 cm and 2.10^3 MPN/l FS at 40 cm. At lower levels the concentrations increase, reaching in the lowest layers 10^7 MPN/l TC, 2.10^5 MPN/l FC and 2.10^4 MPN/l FS.
With respect to the sulfate reducing bacteria, an other image is observed, since the concentration appears to be constant over the whole depth.

Near the outlet of the pond, the superficial layer appears to be less contaminated than near the inlet : about $10.^6$ MPN/l FC and FS and 10^7 MPN/l TC and sulfate reducing bacteria. The depth gradient shows a decrease for TC, FC and FS concentrations reaching in the lowest layers respectively 5.10^5 MPN/l TC, 2.10^3 MPN/l FC and 5.10^3 MPN/l FS. Like near the inlet, concentrations of sulfate reducing bacteria do not vary with depth near the outlet.

Whereas the profiles observed near the outlet are not surprising - a steady inactivation with time of the strict faecal indicators, a moderate inactivation of the total coliforms, and a constant level of the very resistant sulfate reducing bacteria - the increase of the concentration in the deeper layers near the inlet asks for some explanations. This increase could result from the sampling technic used. Indeed the way the core samples were taken could allow a contamination of the lower layer sludge by the sludge from the upper levels which are more contamined. However, in that case, the core samples taken near the outlet should also show a recontamination which is not observed. An other explanation might be the difference between the dimensions of the particles which settle near the inlet and near the outlet. The larger particles settle near the inlet,whereas the smaller ones, among which many algae, settle near the outlet. The technic used for bacteria counting now - the MPN technic - does not allow the correct evaluation of the exact number of bacteria adsorbed on the particles, since several bacteria on one particle are counted as one bacteria only. The maturation process results in a fragmentation of larger particles into smaller ones which all carry adsorbed bacteria which will be counted now. The sludge accumulated near the inlet, formed of large particles, could thus liberate, during its maturation much more important quantities of bacteria than those liberated by the smaller particles settled near the outlet of the pond. This could explain the observed differences between both profiles.

With respect to the Salmonella, only the samples collected with the traps and so corresponding to the youngest sediments were contaminated with these micro-organisms, however to a limited extend : 3,6 10^{2} MPN/l and 3,0 10^{2} MPN/l near respectively the inlet and the outlet. Moreover the concentrations of faecal indicators in these samples correspond fairly well with the values observed in the upper layers of the core-samples.

CONCLUSION

The study of the physical-chemical characteristics of two columns of sludge accumulated in the first of a series of wastewater stabilization ponds, shows that the 10 years storage results in an important increase of the density and in a an enhanced degradation of their organic matter.
The organic matter degradation is accompagnied by the regeneration of about 80% of TOC, 70% of PKN and 75% of TP. Unlike the phosphorus which is rapidly mineralized and which is in dissolved state only significantly present in the young sludge, the ammonia, dissolved in the interstitial phase, has a concentration which increases with depth and thus with the age of the sludge. The differences observed between the two profiles can be explained by the difference in the nature - more mineral near the inlet - and the dimensions of the settling particles - smaller near the outlet.

The bacteriological analysis of the superficial sludges shows that the contamination is more important - about one log for all the parameters studied - near the inlet than near the outlet, except for the spores of the sulfate reducing clostridia.

The maturation of the sludge results in with depth decreasing concentrations - except for the spores of the sulfate reducing clostridia - for the entire profile near the outlet but for only the upper 40 cm of the profile near the inlet ; below 40 cm there is an increase in concentrations of this point. This increase with depth and so with age may be caused by the more important fragmentation of the particulate matter on which the bacteria are adsorbed.

The Salmonella concentration appears at random and low and no concentration gradient could be established.

The effect of the maturation of the sludge on the concentrations of the faecal indicator organisms other than the spores of sulfate reducing clostridia appears limited. Indeed, the observed decreases do not exceed 4 log at most.

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THE IDENTIFICATION OF BENTHIC FEED-BACK IN FACULTATIVE PONDS

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ABSTRACT

An identification study of the feed-back of soluble organic matter from the benthic sludge in facultative ponds is described, based on field data reported for the New Mexico pond system in the USA. The study employed recursive estimation techniques to aid the identification of appropriate model structures.Model development progressed from the analysis of a simple non-reactive system to one incorporating two sub-systems, namely the planktonic region and the benthic region. It was found that the inclusion of a temperature-related feed-back term produced good model results while the temperature-corrected decay rate constant remained constant over time. The magnitude of the feed-back term would appear to be equal to the incoming load in the hottest months of the year. The implications for performance evaluation and design are discussed.

KEYWORDS

Waste stabilisation ponds; wastewater treatment; process modelling.

INTRODUCTION

The knowledge of the treatment mechanisms which occur in facultative stabilisation ponds is reasonably advanced. However, the incorporation of this knowledge into design methodology has been lacking. Most design methods are based on a maximum allowable BOD loading in the coldest month of the year. This approach may be inappropriate when viewed against the various objectives of pond design. For example the critical factor in the coldest month may be the removal of pathogens, LUMBERS and ANDOH (1985). The BOD loading has more relevance to the maintenance of an aerobic layer at the surface and hence facultative conditions. The aerobic layer in facultative ponds has two functions:

- the achievement of a better quality effluent through the predation by higher organisms and aerobic oxidation.

- the elimination of odour nuisance by the gas stripping which occurs through the aerobic layer.

With regard to the prevention of odour the hottest months of the are likely to be of most importance when several factors mitigate to reduce to dissolved oxygen concentration in the upper region. These factors are:

- the increased rate of biological activity.

- the lower dissolved oxygen saturation concentration.

* the increased activity in the benthic sludge and the feed-back of soluble organics to the upper aerobic zone.

* the likely reduction in wind speeds and the associated surface aeration.

As one of the main concerns regarding the use of ponds is the fear of a potential odour nuisance the objective of this study was to identify from field data the influence of the feed-back from the benthic layer so that this could be incorporated as necessary in design.

METHODOLOGY

The approach to model development aws based on the progressive introduction of model complexity as justified by the reduction in errors between the model predictions and the field measurements. The two principles by which the suitability of model structures were assessed were:

1. the recursive estimates of the parameter values should be stationary, showing on trend or systematic variation.

2. the residual errors between the model output and the observations should be stochastic in nature and have no serial structure.

Other criteria used were the Root Mean Squared Error and the Coefficient of Determination.

The data used in the study were reported by LARSEN (1974). A time series plot of the input and output soluble or filtered COD is given in Figure 1.

The final model structure adopted comprised a simple CSTR for the aerobic region and a source/sink term for the benthic region. The model equations are given in ANDOH (1986) together with a detailed description of the model development.

MODELLING RESULTS

The results of the initial model which assumed a simple non-reactive system with no decay mechanisms are shown in Figure 2 for soluble COD. It can be seen that clearly there is a significant degree of treatment occurring. The analysis of the errors and their auto-correlation indicate both bias and serial structure as would be expected, Figure 3.

A simple first order model with no benthic feed-back was then evaluated and the decay rate constant estimated recursively. The tragectory of the estimates of the decay rate constant is shown in Figure 4 where it can be seen that a seasonal element in the model would appear to be missing. The simulation resluts for this model are given in Figure 5 illustrating poor model performance. The error analysis in Figure 6 indicates again that there is both bias and structure in the errors.

It was postulated that the seasonal component missing from the model was a feed-back term from the benthos. This term was then estimated as a parameter and the resulting tragectory displayed a strong dependence on temperature. A source/sink term for benthic activity was then added to the model and the resulting simulation shown in Figure 7 where a much better fit to the data can be seen. The error plots are given in Figure 8 where the errors are more random and also show no serial structure. It was concluded that the addition of a simple source/sink term linked to a CSTR model gave adequate predictions and indicated the importance of the inclusion of such a term.

IMPLICATIONS FOR EVALUATION AND DESIGN

It has been inferred from an unbiased analysis of field data that the feed-back of soluble organics from the benthic sludge in a facultative pond is an important factor regarding the overall loading to the aerobic layer and the maintenance of facultative conditions. Formal experimental verification of this hypothesis is now required so that performance assessment can consider the effect of seasonal variations in benthic feed-back.

Regarding the design of facultative ponds the work has several implications:

* the inclusion of benthic feed-back into design may be important where odour elimination is essential.

* the advantage of anaerobic ponds prior to facultative ponds can be assessed in relation to year round facultative conditions.

* there is a potential for the application of dynamic models to aid the consideration of different seasonal variations in temperature.

* the practice of designing ponds for the allowable BOD loading in the coldest month does not consider the different objectives of pathogen removal and odour control.

CONCLUSION

The use of recursive parameter estimation techniques in model identification has been described and the importance of the benthic feed-back in facultative ponds inferred from field data. The practice of designing ponds for the coldest month is not compatible with other objectives. In the coldest month pathogen removal is likely to be the limiting factor and is unrelated to BOD loading. In the warmest months odour control may be of most concern and the additional loading from the benthic sludge may be critical for the maintenance of aerobic conditions in the surface region.

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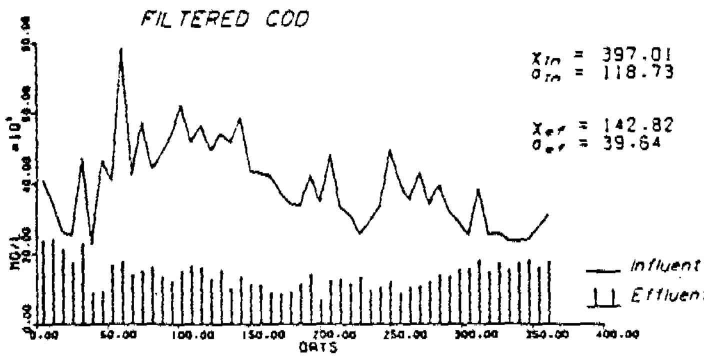

Fig 1. 7-days Moving Average of influent & Effluent Filtered COD (Pond 1)

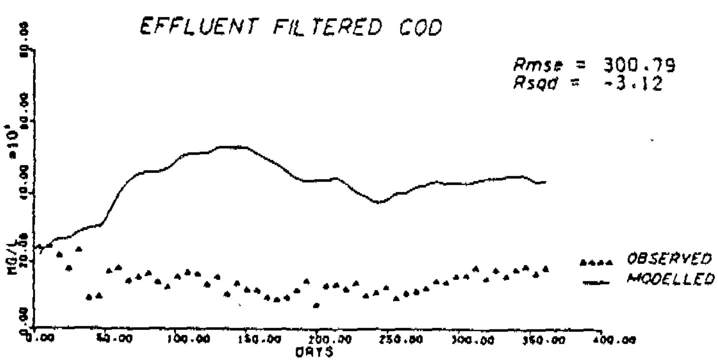

Fig 2. Simulation Plot for Pond 1 Effluent Filtered COD (Zero Rates Model)

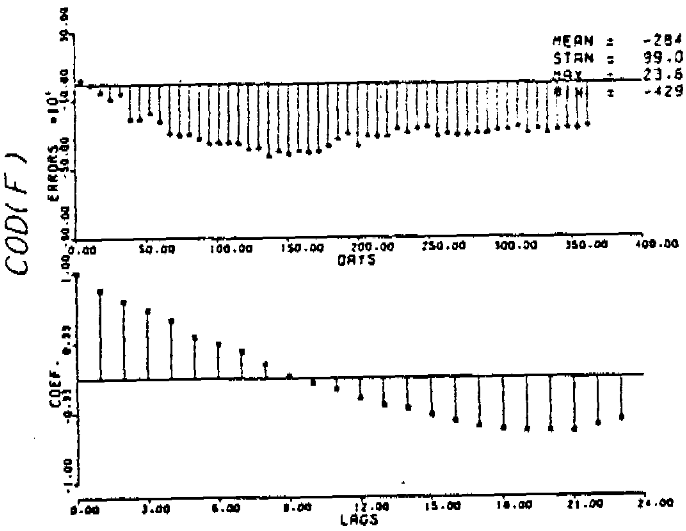

Fig 3. Errors Plot for Zero Rates Model Simulation of Pond 1 Effluent Filtered COD

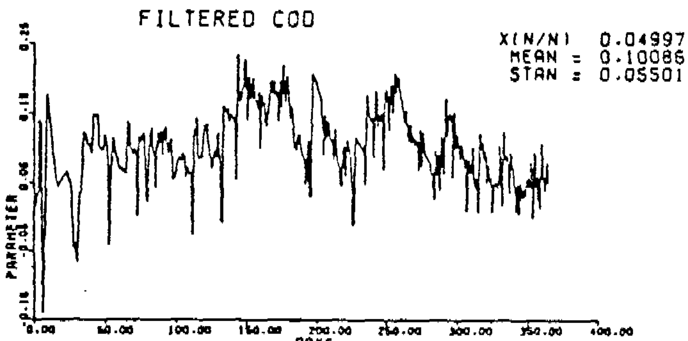

Fig 4. Trajectory of EKF estimate of Pond 1 Filtered COD First Order Decay Rate

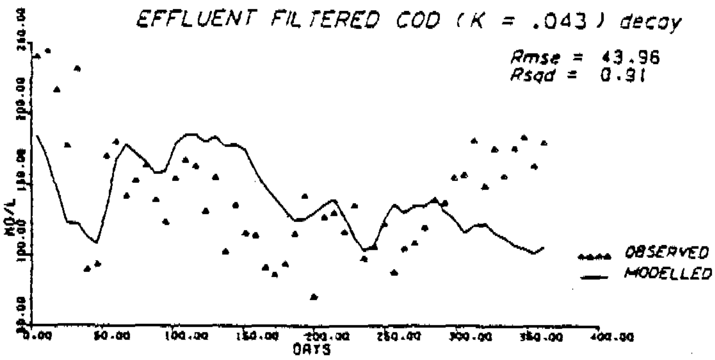

Fig 5. Simulation Plot for Pond 1 Effluent Filtered COD (First Order Decay Rate Model)

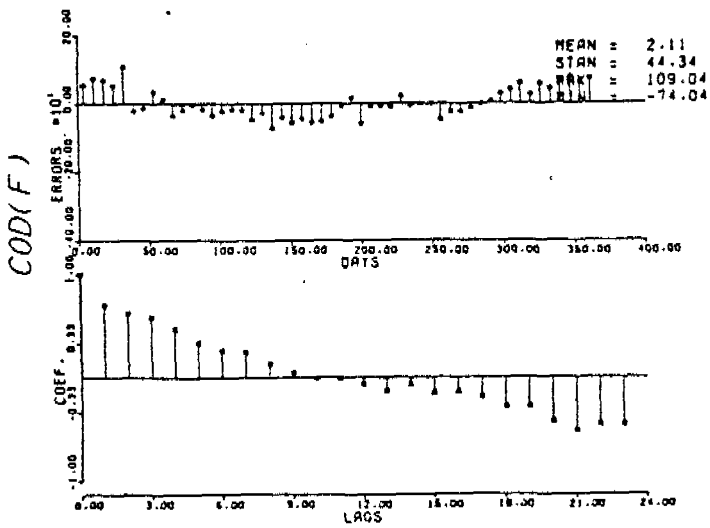

Fig 6. Errors Plot for First Order Decay Rate Model Simulation of Pond 1 Effluent Filtered COD

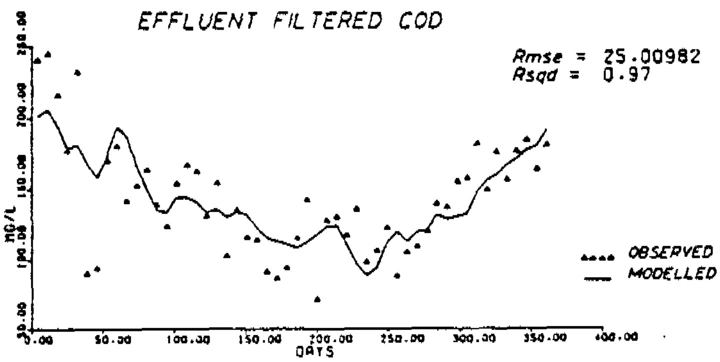

Fig 7. Simulation Plot for Pond 1 Effluent Filtered COD (Carbonaceous Organics Working Model 3)

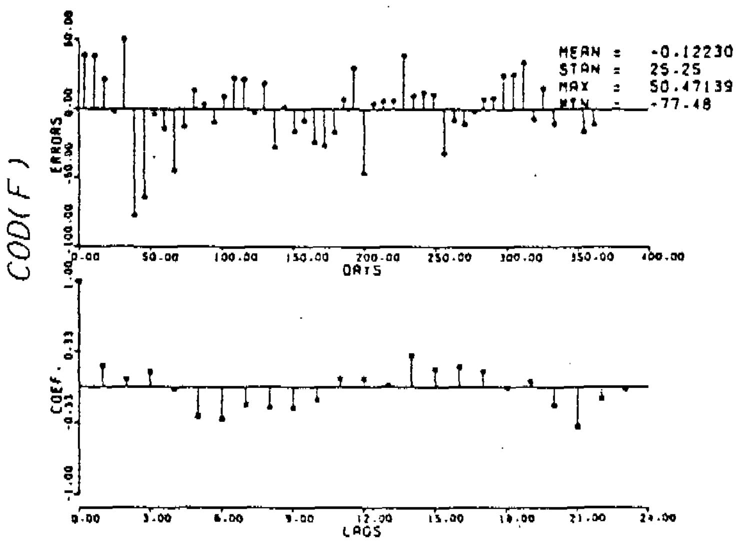

Fig 8. Errors Plot for Carbonaceous Organics Working Model 3 fit to Pond 1 Effluent Filtered COD

THEME 5

POND DESIGN

DESIGN EQUATIONS FOR BOD REMOVAL IN FACULTATIVE PONDS

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ABSTRACT

Facultative pond performance data collected for the US Environmental Protection Agency (USEPA) at four locations throughout the USA and data collected by others were used to evaluate the most frequently used design equations and to develop non-linear design equations. Empirical models were evaluated as well as the classical plug flow and complete mix models. The first order plug flow model gave the best fit of all the rational models. The empirical non-linear models did not fit the data, nor did the other empirical models with the exception being the areal loading and removal model. Attempts to verify the models developed with the USEPA data using data collected by others was not successful with the exception of the areal loading and removal model.

KEYWORDS

Facultative ponds, biological wastewater treatment, models, BOD_5 removal, wastewater, design equations, biochemical oxygen demand

INTRODUCTION

Stabilization ponds have been employed for treatment of wastewater for over 3000 years. The first recorded construction of a pond system in the United States was at San Antonio, TX, in 1901. Today, almost 7000 stabilization ponds are utilized in the United States and even larger numbers throughout the world for treatment of wastewaters (USEPA, 1980). Ponds are used to treat a variety of wastewaters from domestic wastewater to complex industrial wastes, and they function under a wide range of weather conditions, from tropical to arctic. Ponds can be used alone or in combination with other wastewater treatment processes. As understanding of pond operating mechanisms has increased, different types of ponds have been developed for application to specific situations; however, the work described herein is limited to facultative ponds.

PONDS EVALUATED

Facultative pond performance data collected for the USEPA at four locations throughout the USA were used to evaluate the most frequently used design equations and to develop non-linear design equations (USEPA, 1977 a,b,c,d). The procedures used with the EPA data were also applied to a set of data collected by Neel et al. (1961). A brief description of the design, loading and performance of the systems is presented in Table 1.

TABLE 1 Design and Actual Loading Rates and Detention Times for Selected Facultative Ponds

Location	Cells in Series	Organic Loading Rate Design	Actual Total System	Actual First Cell	Theoretical Hydraulic Detention Time Design	Actual	Monthly Final Effluent Exceeded 30 mg/L BOD_5
		kg BOD_5/ha/d			days		
Peterborough, NH	3	20	15	36	57	107	Oct.,Feb.,Mar.,Apr.
Kilmichael, MS	3	67[a]	15	23	79	214	Nov., July
Eudora, KS	3	38	17	43	47	231	Mar., Apr., Aug.
Corinne, UT	7	36	12	30	180	70	None
Experimental System	1	--	--	22 to 112		17 to 87	Not Applicable

[a]First cell.

DATA ANALYSIS

The analyses of the data were limited to the performance obtained in the primary cell because BOD_5 concentrations in the primary cell effluent appear to represent performance of the systems far better than in the following cells. Algae succession, changes in nutrient concentration, and the buffering capacity of the total system appear to exert more influence on the cells following the primary cell. The commonly used design methods are discussed individually in the following sections.

Theoretically, most of the models evaluated should have a line of best fit that has an intercept of zero or unity, but an analysis of the data infrequently yields such an ideal relationship. Therefore, all of the attempts to fit the data to a model were evaluated with the least squares technique with an intercept and with the line of best fit forced through an intercept of zero. When there are large differences, the equations describing the lines of best fit for both cases are presented on each figure along with the corresponding correlation coefficients. In general, if both correlation coefficients are significant (5 percent level) and approximately equal, it can be assumed that the intercept is approximately zero, and the model describe the data to an acceptable degree.

EQUATIONS APPLIED

All of the frequently used design equations were evaluated including rational and empirical models as well as non-linear empirical models developed for this study. A summary of the equations evaluated is presented in Table 2.

RESULTS AND DISCUSSION

Rational Equations

Kinetic models based upon plug flow and complete mix hydrualics or combinations of flow regimes and first order reaction rates with and without Michaelis-Menten enzyme kinetic relationships have been proposed by many authors to describe the performance of wastewater stabilization ponds (Equations 1-10) (Middlebrooks et al., 1982). These models are frequently modified to reflect the influence of temperature by incorporating Equation 22 into the basic equation.

Equations 1-10 were evaluated using the USEPA (1977 a,b,c,d) and Neel et al. (1961) data. The influence of temperature on the calculated reaction rates was evaluated. The reaction rates calculated with the ten equations were essentially independent of the pond wastewater temperature. The logical explanation for the lack of influence by the water temperature is that the pond systems are so large that the temperature effect is masked by other factors. There is no doubt that temperature influences biological activity as shown by the decline in performance during the winter months, but for the USEPA (1977 a,b,c,d) and Neel et al. (1961) systems the influence on reaction rates was overshadowed by other parameters that may include dispersion, detention time, light, species of organisms, etc.

TABLE 2 Design Equations Applied to Facultative Pond Data

Rational Equations

Equation No.

1. $S_o - S = kt$

2. $\ln [S_o/S] = kt$

3. $[1/S - 1/S_o] = kt$

4. $t = [K_s/u] \ln [S_o/S] + [S_o - S]/u$

5. $t = [K_s/u] [1/S - 1/S_o] + [1/u] \ln [S_o/S]$

6. $t = [K_s/2u] [1/S^2 - 1/S_o^2] + [1/u] [1/S - 1/S_o]$

7. $t = [(S_o/S_1) - 1]/k$

8. $S = S_o [4a\, e^{(1/2D)}]/(1+a)^2\, e^{a/2D} - (1-a)^2\, e^{-a/2D}$

9. $\dfrac{S_o - S_1}{X_1} = \left(\dfrac{k_d}{y}\right) t + \dfrac{1}{y}$

10. $\dfrac{t}{1 + tk_d} = \left(\dfrac{k_s}{u}\right) \dfrac{1}{S_1} + \dfrac{1}{u}$

Empirical Equations

11. $L = 10.37 + 0.725\, L_o$

12. $t = 3.5 \times 10^{-5}\, S_{ou}\, [\theta^{(35-T)}]\, ff'$

13. $t = 3.5 \times 10^{-5}\, S_{ou}\, (1.099)^{\frac{\text{LIGHT } (35-5)}{250}}\, ff'$

Non-Linear Equations

Equation No.

14. $S/S_o = k\, t^a\, S_o^b\, L^c\, T^d$

15. $S/S_o = \dfrac{1}{1 + (a+bT)\, t\, e^{(C+dT)\,(pH-6.6)}}$

16. $S/S_o = k\, t^a\, S_o^b\, L^c\, T^d\, (pH)^e$

17. $S/S_o = k\, t^a\, S_o^b\, L^c\, T^d\, (pH)^e + Z$

18. $\ln (S/S_o) = k(t-a)\, (T-b)\, (pH-c)\, L$

19. $\ln (S/S_o) = -k\, t\, e^{aT} - b\, (pH) - cL$

20. $\ln (S/S_o) = -k\, t\, e^{aT} - b(pH) - cL$

21. $\ln (S/S_o) = -k\, t\, T\, (pH)\, L$

22. $(S_o - S)/t = k\, t^a$

23. $(S_o - S)/t = k\, t^a\, S_o^b\, L^c\, T^d$

Temperature Influence

24. $k_2/k_1 = \theta^{T_2 - T_1}$

where,

t = Hydraulic residence time, days
S_o = Influent BOD_5 concentration, mg/L
S = Effluent BOD_5 concentration, mg/L
k = Reaction rate constant, units vary
u = Maximum reaction rate for Michaelis-Menten type kinetics, units vary
K_s = Substrate concentration at 0.5 u
a = $\sqrt{1 + 4\,k\,t\,D}$
D = Dimensionless dispersion number
e = Base of natural logarithms, 2.7183
k_d = Decay rate, day^{-1}
Y = Yield constant, mass of SS or VSS formed/mass of BOD_5 removed
L = Areal BOD_5 removal, kg/ha-d
L_o = Areal BOD_5 loading, kg/ha-d
S_{ou} = Ultimate influent BOD or COD, mg/L
θ = Temperature coefficient, dimensionless
T = Pond water temperature, °C
f = Algal toxicity factor, dimensionless
f' = Sulfide oxygen demand factor, dimensionless
L = Light intensity, langleys
pH = pH value, units
a,b,c,d,e = Reaction orders for various reactants
Z = constant
k_2, k_1, T_2, T_1, = Reaction rate constants and temperature

After observing the lack of influence by the water temperature, the two data sets were fitted to Equations 1-10 to determine if the systems could be defined by these relationships. The best fit of the USEPA data was obtained with the simple plug flow model (Equation 2), and this relationship is shown in Figure 1. The fit of the data is less than ideal but is statistically highly significant (1 percent level). Further attempts to incorporate various types of temperature and light intensity relationships into Equations 1-10 to improve the relationships were unsuccessful.

Using the Neel el al. (1961) data to verify the plug flow relationship shown in Figure 1, the relationship between measured and predicted efluent BOD_5 concentrations shown in Figure 2 was obtained. If the plug flow model developed with the USEPA (1972 a,b,c,d) data defined the performance of the Neel et al. (1961) systems, all of the data points would lie on the 45° line shown on Figure 2. Obviously the model does not predict the performance of the Neel et al. ponds. All attempts to verify the EPA models with the Neel et al. (1961) data were unsuccessful.

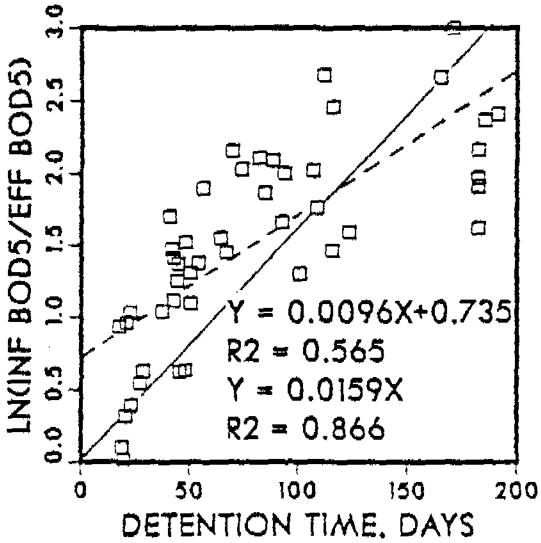

Fig. 1. Plug flow model evaluation using USEPA (1977 a,b,c,d) data

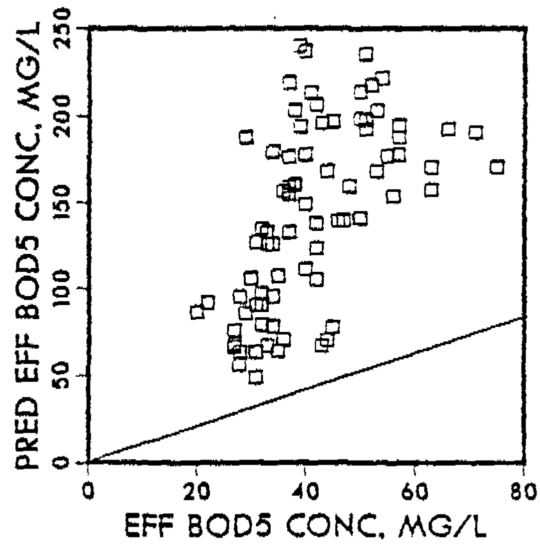

Fig. 2. Verification of plug flow model using Neel et al. (1961) data

Attempts to develop models using the Neel et al. (1961) data with Equations 1-10 resulted in poorer results than those obtained with the USEPA data. For example, using a non-linear data fitting technique with Equation 8, the parameters converged rapidly giving reasonable values for the constants, but with the Neel et al. (1961) data it was not possible to get the parameters to converge. This is the opposite of what was expected because the Neel et al. (1961) data were collected at the same location and under the same climatic conditions.

Empirial Equations

In a survey of the first cell of facultative ponds in tropical and temperate zones, McGarry and Pescod (1970) found that areal BOD_5 removal (L, kg/ha/d) may be estimated through knowledge of areal BOD_5 loading (L_o, kg/ha-d) using Equation 11. The regression equation had a correlation coefficient of 0.995 and a 95 percent confidence interval of $\pm$ 33 kg/ha/d removal. The equation was reported to be valid for any loading between 34 and 560 kg BOD_5/ha/d. McGarry and Pescod (1970) also found that, under normal operating ranges, hydraulic detention time and pond depth have little influence on percentage or areal BOD_5 removal. With such a large 95 percent confidence interval, it is impractical to apply the equation to pond systems loaded at rates between 34 and 112 kg/ha/d or less as was the situation with the majority of the months of operation for the EPA and Neel facultative pond systems.

Relationships between organic removal and organic loading for the lower rates observed at the four EPA and five Neel facultative pond systems were developed using BOD_5, soluble biochemical oxygen demand ($SBOD_5$), chemical oxygen demand (COD), and soluble chemical oxygen demand (SCOD). Statistically significant relationships were observed for all four organic carbon estimating analyses with the EPA data, but the best relationships were observed when the organic removals were calculated using the influent BOD_5 and the effluent BOD_5 and $SBOD_5$ (Figures 3 and 4). $SBOD_5$ data were not available from the Neel et al. (1961) study, but an excellent relationship was obtained with the BOD_5 data (Figure 5). There is excellent agreement between the relationships obtained with the EPA and the Neel data. Either can be used to accurately predict the performance of the other. The incorporation of temperature and/or light correction factors caused a deterioration in the relationship for both sets of data. The BOD_5 and $SBOD_5$ relationship is shown in Figure 4. The 95 percent confidence intervals for the EPA and Neel relationships are smaller than the value reported by McGarry and Pescod (1980) (EPA BOD_5 vs $EBOD_5$ C.I. = $\pm$ 2.0 kg/ha-d; EPA BOD_5 vs $ESBOD_5$ C.I. = $\pm$ 0.4 kg/ha-d; Neel BOD_5 vs $EBOD_5$ C.I. = $\pm$ 0.37 kg/ha-d).

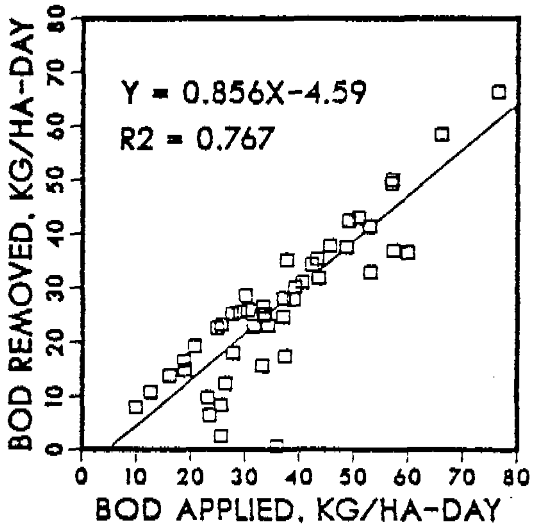

Fig. 3. Removal rate versus loading rate using USEPA (1977a,b,c,d) influent and effluent BOD_5 data

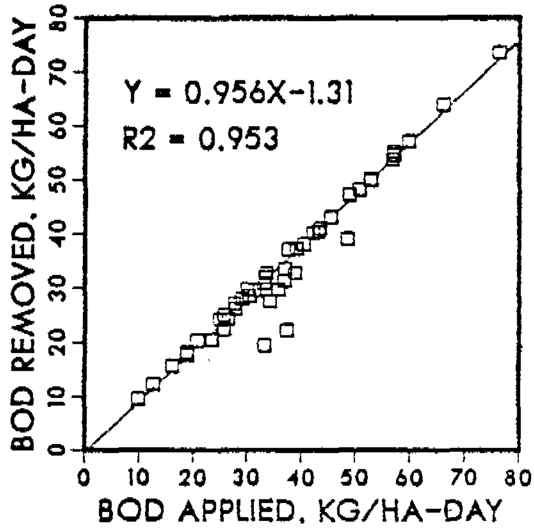

Fig. 4. Removal rate versus loading rate using USEPA (1977 a,b,c,d) influent BOD_5 and and effluent $SBOD_5$ data

Although excellent relationships were obtained, there is reason to question their validity because the relationships were developed from a correlation of a number with a number obtained by subtracting a relatively small value from the first number and multiplying both by the flow rate divided by the surface area of the pond, i.e. Load = influent BOD (flow rate)/area and Removal = (influent BOD - effluent BOD) (flow rate)/area. A correlation of influent BOD versus (influent BOD - effluent BOD) for the Neel et al. (1961) data yields an excellent coefficient of determination (r^2 = 0.896). Converting the influent and effluent values to loading and removal rates yields a coefficient of determination of 0.982.

Gloyna (1976) proposed Equation 12 for the design of facultative wastewater stabilization ponds. The BOD_5 removal efficiency can be expected to be 80 to 90 percent based on unfiltered influent samples and filtered effluent samples. A pond depth of 1.5 m is suggested for systems with significant seasonal variations in temperature and major fluctuations in daily flow. Surface area design using the Gloyna equation should always be based on a 1-m depth. The additional 0.5 m of depth is provided to store sludge. According to Gloyna (1976), the algal toxicity factor (f) can be assumed to be equal to 1.0 for domestic wastes and many industrial wastes. The sulfide oxygen demand (f') is also equal to 1.0 for $SO_4^{=}$ ion concentration of less than 500 mg/L. Gloyna (1976) also suggests using the average temperature of the pond water in the critical or coldest month. In this equation, sunlight is not considered to be critical in pond design but may be incorporated into the Gloyna equation by multiplying the pond volume by the ratio of sunlight in the particular area to the average found in the Southwest.

Although ultimate BOD data were not available, COD, SCOD, BOD_5, and $SBOD_5$ data were used. Use of the Gloyna equation with the EPA and Neel data failed to produce any good relationships. The relationships obtained with the COD, SCOD, BOD_5, AND $SBOD_5$ data were statistically significant, but the data points were scattered. The relationship shown in Figure 6 was the best fit obtained for the data, and resulted in Equation 13. The validity of Equation 13 is questionable because of the scattered data, but the relationship is statistically significant (1 percent).

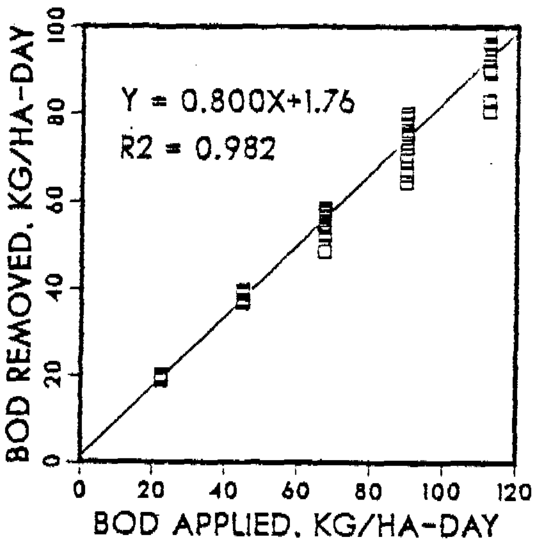

Fig. 5. Removal rate versus loading rate using Neel et al. (1961) data

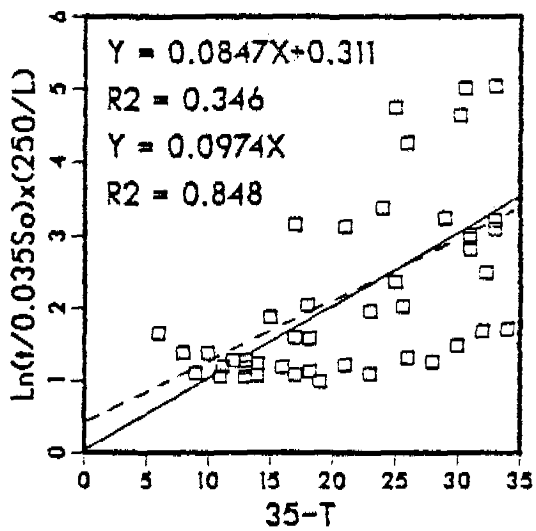

Fig. 6. Modified Gloyna equation relationship obtained with USEPA (1977a,b,c,d) data

Non-linear Equations

Equation 23 produced the best fit of the USEPA data of all the non-linear equations (Figure 7); however, when the equation was solved for the effluent concentration of BOD_5, there was considerable scatter of the results (Figure 8). An attempt to verify Equation 23 using the Neel data was unsuccessful.

None of the other non-linear equations, including many not shown in Table 2, produced a successful fit of either set of data. Temperature and light correction factors did little to improve the fit of the data to the various models.

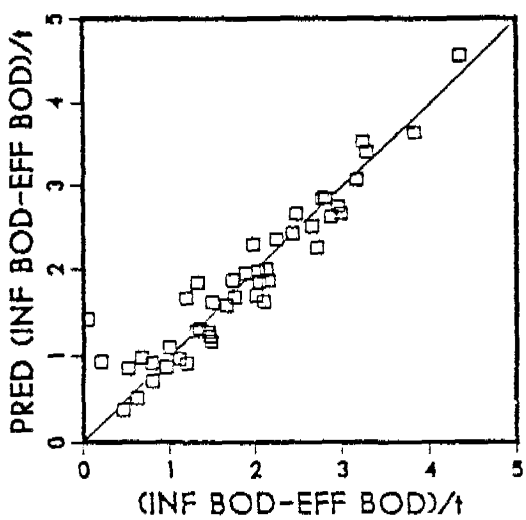

Fig. 7. Relationship obtained with Equation 23 using USEPA (1977a,b,c,d) data

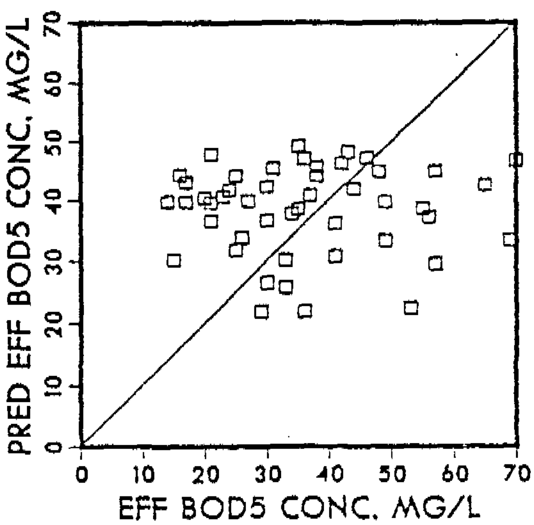

Fig. 8. Predicted effluent BOD_5 concentration versus the measured effluent BOD_5 concentration using Equation 23 to predict the effluent value

CONCLUSIONS

A simple plug flow hydraulic model with first order reaction rate produced the best fit of the USEPA data with the rational models. Attempts to verify the plug flow model based on the USEPA data with the Neel data were unsuccessful.

The fit of the data to the plug flow model was not improved by incorporating temperature and light intensity correction factors. The Neel data also were uninfluenced by temperature and light corrections. As reported by Neel et al. (1961), the reaction rates were affected most by the hydraulic detention time.

A plot of organic removal rate versus the organic loading rate produced the best fit of the data of all the models considered. Excellent relationships were obtained with both the USEPA and Neel data. Either equation obtained using total BOD_5 concentration in the influent and effluent could be used to accurately predict the performance of the other; however, the statistical validity of the relationship is questionable.

None of the non-linear equations produced a relationship capable of predicting the performance of the pond systems.

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INTERPRETATION OF LABORATORY - SCALE
WASTE STABILIZATION POND STUDIES

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ABSTRACT

The objective of this paper is to demonstrate the importance of choosing an appropriate mathematical model when analyzing the data from laboratory-scale studies of waste stabilization ponds. Two case studies are presented based on work by Thirumurthi and Nashashibi (1967) and Uhlmann et al (1983), both using semicontinuous methods of experimentation involving the addition of discrete volumes of feed at regular intervals. In both cases the authors have used mathematical models of continuous processes to analyse their results. This paper shows how semicontinuous models can be used in both studies, leading to significant differences in interpretation of the data; in the first case study this relates to the determination of rate constants and, in the second case study, to the determination of an appropriate model to describe hydraulic mixing. Each case study concludes with a discussion of the significance of the semicontinuous interpretation in the context of waste stabilization pond design.

KEYWORDS

Waste stabilization ponds; reaction engineering; hydraulic mixing; parameter estimation; systems modelling

INTRODUCTION

Chemical reaction engineering principles have been used in design and performance studies of waste stabilization ponds since the 1960's. (Marais, 1966; Thirumurthi and Nashashibi, 1967; Thirumurthi, 1969; Mara, 1976; Ferra and Harleman, 1980). Even so, a review of performance data from large-scale waste stabilization ponds by Finney and Middlebrooks (1980) revealed serious discrepancies between expected and observed removal efficiencies of BOD and faecal colliforms. The principal reason, in their view, lies in inadequate descriptions of the hydraulic residence-time distribution in a waste stabilization pond but problems arising from reaction rate data would seem to be equally important (Wood, 1986). The physical and chemical processes taking place in a waste stabilization pond are complex and simplifying assumptions are necessary in order to develop mathematical models for design purposes. It is useful to discuss briefly the assumptions in relation to BOD removal.

Steady-state organic and hydraulic loadings. It is reasonable to accept this as a basis for large-scale design even though in practice inevitable variations in effluent quantity and quality are going to occur. The design is therefore based on temporal averages of volume rate and pollutant concentration, and provided fluctuations about the averages are not large this seems adequate. Only Ferra and Harleman (1980) appear to have attempted a dynamic analysis of waste stabilization ponds, in a useful but preliminary study. In laboratory-scale experiments it is possible to maintain constant loadings and therefore these studies can be more precisely analyzed by a steady-state mathematical model.

First-order BOD removal as a rate limiting step. Previous researchers, without exception, appear to have accepted this as a reasonable assumption, implying that the rates of other processes (oxygen mass transfer, bacterial and algal growth, etc.) are an order of magnitude greater than the rate of reaction of the soluble organics in the wastewater. In the circumstances, given the complexity of the system, this is perhaps not unreasonable as a starting point. The term 'first-order' for the controlling reaction is, however, open to question in view of the apparent dependence of the experimentally determined rate constant on organic loading as well as temperature, the only variable normally found to influence a genuine first-order rate constant.

Simple models of hydraulic mixing. Hydraulic mixing in large-scale waste stabilization ponds is not easy to describe with any certainty; few residence-time measurements are available because of the difficulty in carrying out accurate tracer studies, principally due to maintaining steady flow conditions, and the large analytical and sampling errors associated with the inevitable massive dilution of the tracer. In sequence simplified mixing models are used, such as plug-flow, plug-flow with diffusion, single stirred tank, or multiple stirred tanks in series. In the laboratory, mixing is more readily controlled and can be made to conform with complete mixing or plug-flow, mentioned above, although as shown later in this paper non-ideal mixing can occur.

Scope of this Paper

The contents of this paper are solely concerned with laboratory-scale experiments and are presented as two case studies. The first refers to the determination of rate constants using data determined by Uhlmann et al (1983), emphasizing the need for an appropriate mathematical model and addressing the question of whether the rate-constants are truly first-order. The second uses data determined by Thirumurthi and Nashashibi (1967) and demonstrates a methodology for describing the non-ideal mixing in their experiments, and the subsequent use of the non-ideal mixing model to determine rate constants.

Perhaps the most important question of all is not faced in this paper; the extent to which data obtained in a laboratory-scale system can ever be representative of conditions in a large-scale waste stabilization pond. The assumption is made, along with other researchers, that any rate and mixing parameters, so obtained, are meaningful. It is perhaps a question that deserves more attention.

CASE STUDY I : RATE CONSTANT DETERMINATION

This example demonstrates the importance of using an appropriate mathematical model to estimate rate constants from data obtained in a laboratory-scale experiment. Uhlmann et al (1983), in a well-planned study, used rectangular glass vessels (16 l) which were fed with discrete amounts of a synthetic waste every 24 hrs. Equal volumes of liquids were displaced from the tanks at each addition, thereby maintaining constant reaction volume, and the vessel contents were then stirred. After sufficient time, a steady level of BOD was achieved in the displaced liquid. The experiment was therefore semicontinuous, as a consequence of the discrete addition-displacement process; yet a continuous plug-flow model was used to determine the rate constants from the data. Wood (1986) pointed out this anomaly and, using a semicontinuous model, found marked differences between the rate constants obtained from the two models for comparable data sets. The outcome was not unexpected because the two model equations are quite different -

$$\frac{\text{Plug-Flow}}{\text{(Continuous)}} \qquad \frac{S_e}{S_i} = \exp(-k\bar{t}) \tag{1a}$$

$$\frac{\text{Discrete Addition}}{\text{(Semicontinuous)}} \qquad \frac{S_e}{S_i} = \frac{f}{\exp(kf\bar{t}) + f - 1} \tag{1b}$$

S_i, S_e = BOD in the feed and displaced fluid, respectively
k = first-order rate constant
$\bar{t}$ = mean residence-time
f = ratio of feed volume to reaction volume in discrete addition model

Details of the derivation of the semicontinuous model are given by Wood (1986). The experiment was carried out at five temperatures (4, 10, 20, 30 and 40° C), for four mean hydraulic residence times (5, 10, 20 and 40 days), and four BOD loadings (1, 5, 25 and 125 g BOD/m³.d); and for each combination of these variables two vessels were illuminated for 16 hours per day and two were kept in the dark. Values for S_i and S_e were determined as COD (on filtered samples) and as BOD_5 (on unfiltered samples). Arithmetic means of the rate constant were calculated from the four experiments carried out for each combination of variables. As the authors explain, this averaging procedure was necessary because neither BOD_5 or COD alone is necessarily representative of the substrate quality; moreover, uncertainties in sampling and analysis, and differences between experimental units, are compensated. Figure 1 shows data for 20° C which has been analyzed using both of the models described above.

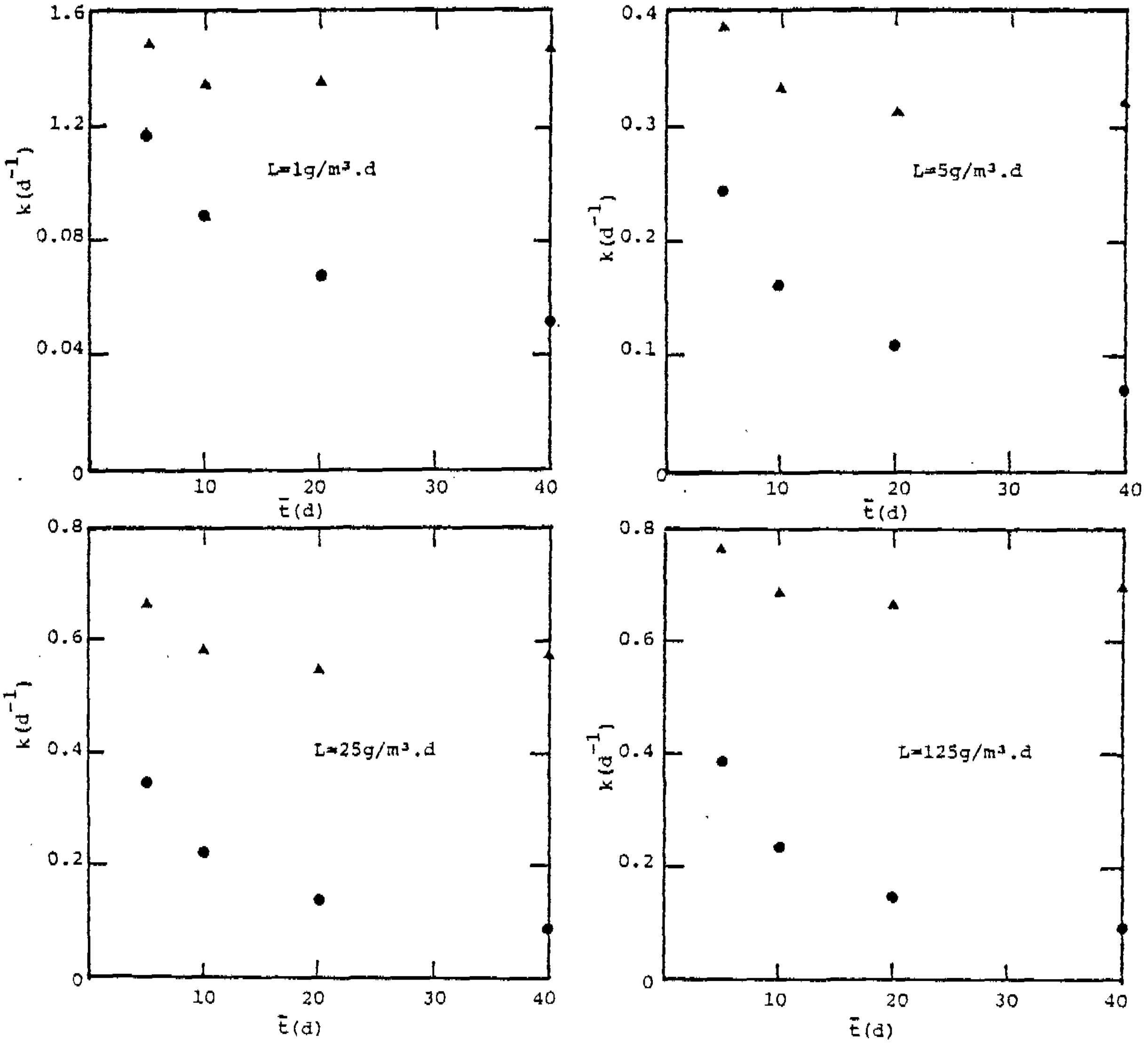

Fig.1 Comparison of rate constants $k(d^{-1})$ from continuous model (●) and semicontinuous model (▲): 20°C data.

Interpretation of Rate Constants

For convenience, in the following discussion the term SC will be used to refer to the semicontinuous model and PF to the continuous plug-flow model. The results for experiments at 20° C given in Figure 1 show similar trends to those at other temperatures. (Space limitations preclude a comprehensive coverage but Wood (1986) gives numerical values of SC and PF rate constants for all experimental combinations.) Two main conclusions can be drawn from the results:

(a) the PF rate constants are significantly different from the SC rate constants
(b) the PF rate constants are dependent on BOD loading, mean residence-time and temperature whereas the SC rate constants appear to depend only on BOD loading and temperature.

The relationship between the SC rate constant and BOD volume loading at a given temperature appears to tend towards an assymptotic value, suggesting a relationship of the form -

$$k = \frac{k^{*}.L}{L + K} \tag{2a}$$

where k = SC rate constant (d^{-1})

k^{*} = limiting value of SC rate constant (d^{-1})

L = BOD value loading ($g/m^3.d$)

K = a parameter ($g/m^3.d$)

k^{*} and K are both temperature dependent, as follows -

$$k^{*} = 0.011 + 0.0312\ T \tag{2b}$$

$$K^{-1} = 0.194 + 2.24\ T^{-1} \tag{2c}$$

where T = Temperature (°C)

The relationship between the PF rate constant and the experimental variables is more complex, as follows -

$$K = \frac{\bar{t}^{\,B}}{A} \tag{3}$$

where $A = (0.327 + 10.277/T + 1/(L(0.25 + 0.476/T))$

$B = -(1.39 + 1.304/T + (0.061 + 0.05T)/L)^{-1}$

$\bar{t}$ = mean residence time (d)

L, T as defined above

Reaction Order

As mentioned earlier, the rate limiting process in a waste stabilization pond is assumed to be a first-order removal of BOD. The use of the term 'first-order' must be questioned because of the dependence of the rate constant on BOD loading as well as temperature, as confirmed in studies by Thirumurthi (1969, 1974). In a genuine first-order reaction the rate constant only depends on temperature, and is independent of factors such as reactant loading or mean residence time. (An exception arises in the case of a 'pseudo' first-order reaction for a second-order system with a large excess of one of the reactants.) The kinetics of the waste stabilization system would therefore appear to be non-linear.

The present author has examined several simple n-th order kinetic models and a Monod model in an attempt to resolve this problem but the results are not conclusive. A significant difficulty lies in the accuracy of the available input/output data based on measurements of BOD_5; as is well-known these measurements are far from precise and certainly do not achieve the level of accuracy to enable a clear discrimination to be made between various potential kinetic models. Until this difficulty is overcome, the first-order rate limiting step of BOD removal must be adopted as the basis for design and performance studies of large-scale waste stabilization ponds. Therefore it is most important that the best estimates of the first-order rate constant be obtained from the available data, with particular emphasis being placed on the use of an appropriate model for this purpose. As this case study shows, significant errors in the rate constant can occur from the use of an inappropriate model - no matter how carefully the experimentation is planned and carried out. Based on the study by Uhlmann et al (1983), equations (2a), (2b) and (2c) are recommended, in place of equation (3), for design calculations.

CASE STUDY II : MIXING IN A SEMICONTINUOUS SYSTEM

This case study describes a methodology for determining the mixing in a semicontinuous system from tracer measurements. Thirumurthi and Nashashibi (1967) used reaction volumes of 23.6 l in a study which involved a once-daily addition of synthetic waste with a corresponding displacement of effluent from the reaction vessel but, unlike the previous study by Uhlman et al (1983), no deliberate stirring of the reaction volume took place following each addition of waste. The only stirring of the system was self-induced by the act of pouring fresh waste into the reactor. Tracer experiments using sodium chloride were therefore carried out to characterize the mixing by a suitable mixing diffusion and used the solution of Wehner and Wilhelm (1956) to estimate mixing parameters from the tracer data. Here, again, a model only appropriate in the context of a continuous flow system has been used for a semicontinuous system involving the discrete addition of fresh feed. Existing mixing models based on continuous residence-time theory are well covered in the literature (see, for example, Levenspiel (1984), for an up-to-date overview) but little has been published of direct relevance to the discrete addition system; the methodology proposed here is meant to be a preliminary examination of this problem.

Description of Proposed Methodology

For convenience a discrete volume of liquid is referred to as a pulse. Suppose that a suitable tracer is dissolved in one of the inlet pulses and let the time of its injection be designated as the origin for time measurement. All previous pulses and all subsequent pulses are free from tracer. The sequence of effluent pulses is then monitored for tracer content and the results expressed as a fraction of the original tracer injected. In this way, the residence-time history of injected material can be ascertained, as follows:-

Let v = the volume of a pulse
c_i = tracer concentration in the injected pulse
c_k = tracer concentration of the k-th effluent pulse
t_p = interval between pulses
f_k = fraction of inlet pulse with residence-time $k\ t_p$

Then $$f_k = c_k / c_i \qquad (4)$$

This data, if required, can then be converted to a cumulative residence-time distribution by simple summation.

Interpretation of Residence-Time Data

Following the practice used for continuous flow systems, it is useful to consider the limiting cases of complete-mixing and zero-mixing, and then use them to formulate descriptions for intermediate mixing.

(a) Complete-mixing. If the entering pulse of fluid first displaces an equal volume pulse of the existing fluid in the system and then completely mixes with the residual fluid in the system, this process can be described as follows:-

$$v.\ u_{k-1} + (V - v).\ c_{k-1} = V.\ c_k \qquad (5a)$$

where v = pulse volume
V = system volume
u = tracer concentration, entering pulse
c = tracer concentration, displaced pulse
k = the k-th pulse after tracer injection

This equation can be simplified into the form of a transfer function by the use of the backward shift operator z^{-1} which has the effect of replacing the c_{k-1} term by $z^{-1}.c_k$; the resulting expression is:-

$$\frac{c_k}{u_{k-1}} = \frac{f}{1-(1-f)z^{-1}} \qquad (5b)$$

where f = the ratio of pulse to system volume

(b) Zero-mixing. If the entering pulse simply displaces an equal volume of existing system fluid and subsequently retains its identity, without mixing with its surroundings, the process is described as follows:-

$$c_k = u_{k-\tau} \tag{6}$$

where τ is the delay-time between pulse entry and exit.

The pulses are assumed to leave in the same order as they enter the system: in consequence the delay-time (i.e residence-time) for each pulse is identically equal to τ. This mixing limit is analogous to the 'plug-flow' condition in continuous flow systems.

(c) Partial mixing. Intermediate degrees of mixing of the inlet pulse with its surroundings can be characterized by regarding the total flow system as a structure of sub-regions, each of which behaves as either a zero-mixed or completely-mixed entity. Once the structure is formulated it is not a difficult matter to combine the transfer function models of each of the sub-regions into an overall transfer model for the whole system. This results in an input-output transfer function model with the general functional relationship:-

$$\frac{c_k}{u_{k-\tau}} = \frac{B(z^{-1})}{A(z^{-1})} \tag{7}$$

where $A(z^{-1})$ and $B(z^{-1})$ are polynomials in the transfer function operator z^{-1}.

Estimation of Transfer Function Model Parameters from Experimental Data

The practical problem of interest is the reverse of the process described above. From input/output measurements of tracer concentration, 'best' estimates are made of the polynomial coefficients in the transfer function model, Equation (7). The overall transfer function model is then decomposed into sub-elements, each of which can be identified with one of the ideal mixing models, Equations (5b) or (6). The method of estimation in this paper has been developed by Young (1984) and is based on instrumental variables; details cannot be given here but can be obtained from his book.

The experimental tracer data were obtained by Nashashibi and Thirumurthi (1967) from a control pond of the same dimensions as their waste stabilization ponds. The pond was first filled with distilled water and the discrete addition process was begun with the injection of a pulse containing sodium chloride; thereafter a similar volume of distilled water was added daily and the sequence of pulses of displaced effluent were monitored for sodium chloride. The results of one experiment are shown in Figure 2. (A notable feature of this data is that the experiment was terminated after 20 days when appreciable amounts of tracer were still being found in the effluent pulses.)

The measured sequence of input/output tracer data was analysed with a computer package, 'MICRO-CAPTAIN', which incorporates a recursive version of the estimation algorithm developed by Young (1984). The user first nominates the orders of the polynomials $A(z^{-1})$ and $B(z^{-1})$ in the transfer function model, Equation (5), and the time delay parameter τ. The package then accepts the input/output tracer data in sequence and progressively estimates the coefficients in the polynomials $A(z^{-1})$ and $B(z^{-1})$ and their standard errors; it also provides a coefficient of determination which is a normalised measure of how well the model explains the data, and an error variance norm which is a sensitive indicator of overparameterization. The user can therefore screen several transfer function models and choose the one giving the best compromise between the coefficient of determination and the number of parameters specified for estimation.

The transfer function model best describing the experimental results from Thirumurthi's study proved to be a second-order polynomial in $A(z^{-1})$, a third-order polynomial in $B(z^{-1})$ and a time delay of 2 days.

The coefficients in the polynomials are given below:-

$$A(z^{-1}) = 1 + a_1 z^{-1} + a_2 z^{-2} \tag{7a}$$

where $a_1 = 1.054$; $a_2 = 0.104$

$$B(z^{-1}) = b_0 + b_1 z^{-1} + b_2 z^{-2} + b_3 z^{-3} \tag{7b}$$

where $b_0 = 0.0038$; $b_1 = 0.0134$; $b_2 = 0.0208$; $b_3 = 0.0121$

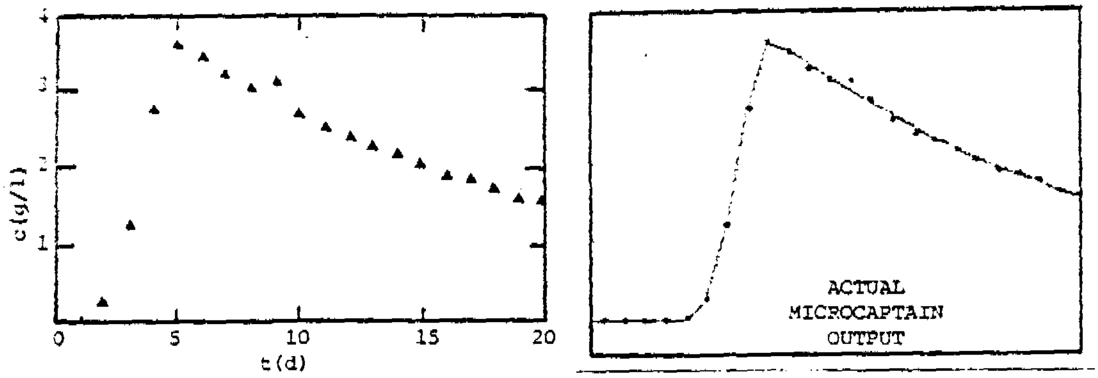

Fig.2 Experimental tracer concentration data and the corresponding transfer function model predictions.

Interpretation and Use of the Transfer Function Model

The coefficient of determination for the model is 0.999, as reflected in Fig.2 by the close agreement between prediction and measurement. The second-order polynominal $A(z^{-1})$ can be factorised and the transfer model rearranged -

$$c_k = \frac{0.056}{z^{-1} - 0.944} \cdot \frac{0.890}{z^{-1} - 0.110} \cdot U \tag{7c}$$

where $U = 0.076\, u_{k-2} + 0.268\, u_{k-3} + 0.415\, u_{k-4} + 0.242\, u_{k-5}$

This is equivalent to two completely-mixed regions in series with subsequent division of the fluid into fractions with delay times of 2, 3, 4 and 5 days. With the residence-time thus characterised, by combining equations (4) and (7c), the model can be used to estimate rate constants from input/output data from a pond with non-ideal mixing - such as used by Thirumurthi and Nashashibi (1967).

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WASTE STABILIZATION POND PREDICTION MODEL

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ABSTRACT

This paper presents a prediction model for calculating the expected effluent performance of a facultative waste stabilization pond. The model is intended to improve on current design approaches through more appropriate recognition of physical and climatic factors which affect the prediction of pond performance. The importance of hydraulic routing through a pond is stressed along with wind mixing, thermal stratification, pond geometry, and other basic design parameters. The final form of the model is a working equation based on analyses of the various pond processes; because of the complexity of the final analytical model, it must be applied using computerized methods.

KEYWORDS

Facultative ponds; prediction model; effluent performance; design factors; physical and environmental effects; computer application.

INTRODUCTION

Three techniques are commonly used in the design of a waste stabilization pond. One technique is based on organic loading; a second involves the use of empirical relationships based on actual pond performance data; and a third involves the use of a rational design model based on the assumption of first order biochemical oxygen demand (BOD5) removal rates. However, many of these relationships have serious limitations in predicting pond performance. Therefore, the analytical development which follows in this paper is intended to improve on these current design approaches.

HYDRAULICS OF FLOW THROUGH POND

The hydraulic behavior in ponds is usually simulated in one of two ways; the pond is assumed to be either a plug flow reactor or a completely-mixed reactor. The equations which generally are used to describe removal efficiency in plug flow and completely-mixed reactors are listed below:

Plug flow: $C_e/C_o = e^{-Kt}$ (1)

C_e = effluent BOD5 concentration (kg/m^3)
C_o = influent BOD5 concentration (kg/m^3)
K = first order reaction constant (d^{-1})
t = mean residence time in reactor (d)

Completely-mixed: $C_e/C_o = 1/(1 + Kt)$ (2)

The accuracy of these equations may vary substantially with actual pond conditions and therefore their application is limited.

The hydraulic flow pattern shown in Figure 1, as previously used by Ferrara and Harleman (1981), is used as the basis for hydraulic flow routing through the pond. However, instead of representing the active zone as one reactor, it is used here as two separate reactors, the first being a plug flow reactor and the second being a completely mixed reactor. In addition, the two-dimensional concept in the active zone is expanded to include the depth dimension as shown in Figures 2 and 3. The proposed model, then, consists of three separate reactors; a plug flow reactor and a completely mixed reactor in the active zone and a completely mixed reactor in the return zone. Figure 2 represents the case where the pond inflow is directed toward the bottom of the pond, which occurs in warmer periods when thermal stratification is most pronounced, whereas Figure 3 represents the case where the pond inflow is directed toward the top of the pond as in colder periods.

For purposes of modeling, pond parameters are identified and estimated as follows:

$\underline{a} = L'/L$

$\underline{a}$ = plug flow parameter

L' = length of plug flow reactor in the active zone (m)

L = total pond length (m)

$\underline{b} = W'/W$

$\underline{b}$ = active zone width parameter

W' = width of active zone (m)

W = total pond width (m)

$\underline{g} = D'/D$

$\underline{g}$ = active zone depth parameter

D' = depth of active zone (m)

D = total pond depth (m)

$\underline{p} = Q_R/Q_o$

$\underline{p}$, is used to estimate the flows in the active and return zones.

APPLICATION OF HYDRAULICS TO BOD5 REMOVAL PREDICTION

Two separate flow patterns will usually exist in a pond at different times of the year as shown in Figures 2 and 3. Thus separate equations must be generated to predict BOD5 removal.

In the overflow case, the inflow to the pond and the return flow from reactor 3 both enter reactor 1 and travel through in plug flow regime. The equation which describes BOD5 removal in this reactor is

$$C_{e1} = C_{i1}\,(e^{-K_1 t_1}) \tag{3}$$

C_{e1} = effluent BOD5 concentration of reactor 1 (kg/m^3)

C_{i1} = influent BOD5 concentration of reactor 1 (kg/m^3)

K_1 = first order reaction constant of reactor 1 (d^{-1})

t_1 = residence time in reactor 1 (d)

To utilize this equation, the values of C_{i1}, K_1, and t_1 must be evaluated.

The concentration of BOD5 in the reactor influent is calculated by combining the mass of non-settleable BOD5 contributed by the recycle flow and the non-settleable BOD5 contributed by the pond inflow, and dividing that sum by the active zone flow rate. This is expressed in equation form as

$$C_{i1} = (C_o)(1-i_s) + (C_{e3})(\underline{p}) \;\; / \;\; (1+\underline{p}) \tag{4}$$

C_o = pond inflow BOD5 concentration (kg/m^3)

i_s = fraction of BOD5 in pond inflow which is assumed to settle and deposit on the bottom of the pond

C_{e3} = effluent BOD5 concentration in reactor 3 (kg/m^3)

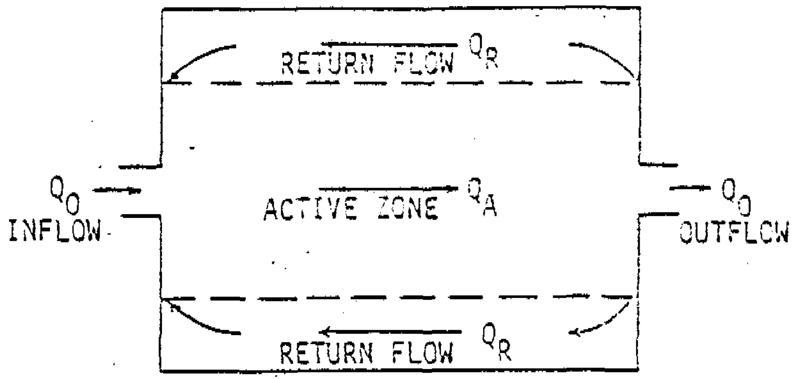

FIG. 1 - HYDRAULIC CONCEPT OF POND FLOW - PLAN VIEW

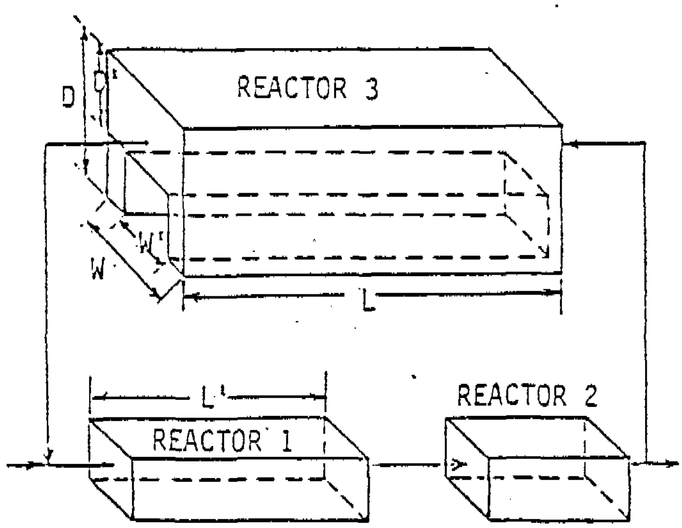

FIG. 2 - HYDRAULIC PATTERN FOR POND BOTTOM FLOW CONDITION

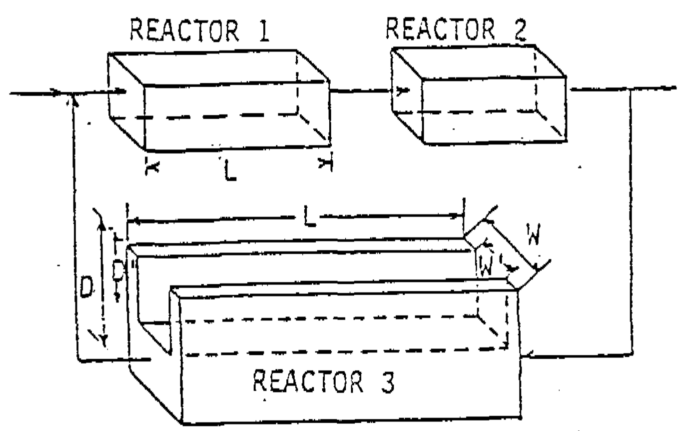

FIG. 3 - HYDRAULIC PATTERN FOR POND TOP FLOW CONDITION

Two assumptions are made regarding the settleable BOD5: the first is that all of the settleable BOD5 in the pond inflow settles out before the flow enters reactor 1, and the second is that the BOD5 found in the recycle flow is entirely non-settleable.

The value of t_1 will depend upon the four pond parameters which define the size of each reactor and the flow through it. The appropriate equation is

$$t_1 = t(\underline{a})(\underline{b})(\underline{g})/(1+\underline{p}) \qquad (5)$$

t = mean pond residence time (d)

The first order reaction constant K has been shown to vary tremendously in different studies. Marais (1966) found that the data best fit the equation

$$K_T = K_{35}\ (1.085)^{T-35} \qquad (6)$$

T = pond liquid temperature (^oC)

K_T = first order reaction constant at temperature T (d^{-1})

K_{35} = first order reaction constant at 35^oC (= 1.2 d^{-1})

This equation will be used to determine the value of K in each of the three reactors.

After the flow passes through reactor 1, it enters reactor 2. The equation which estimates BOD5 removal in this reactor is

$$C_{e2} = C_{e1}/(1+K_2t_2) \qquad (7)$$

C_{e2} = effluent BOD5 concentration of reactor 2 (kg/m^3)

K_2 = first order reaction constant in reactor 2 (d^{-1})

t_2 = residence time in reactor 2 (d)

The value of K_2 is determined using equation (6) and, if the liquid temperature in reactor 2 is equal to the liquid temperature in reactor 1, then K_2 will be assumed equal to K_1. It should be noted that this will probably not be true; it is likely that K_1 will be larger than K_2 since the BOD5 which could easily be metabolized was removed in reactor 1, leaving the less reactive BOD5 for reactors 2 and 3. The value of t_2 is determined by the equation

$$t_2 = t(1-\underline{a})(\underline{b})(\underline{g})/(1+\underline{p}) \qquad (8)$$

The effluent from reactor 2 is the pond effluent, as well as the influent to reactor 3. The effluent BOD5 concentration C_{e2}, however, will not be equivalent to the total effluent BOD5 concentration, because the effluent will contain algae which exerts an oxygen demand. In this paper, then, the term 'total effluent BOD5 concentration' will consider the BOD5 contribution from algae, while the term 'effluent BOD5 concentration' will not include the effect of algae.

Reactor 3 is the most complex of the three reactors when considering the overflow case. BOD5 loading comes from the influent from reactor 2, and by those products of anaerobic fermentation which do not leave the pond as a gas. Because this reactor is the only one which has the pond bottom as one of its boundaries, it is assumed that all of the anaerobic end products which do not leave the pond as a gas enter this reactor only.

The equation which describes the activity in this reactor is

$$C_{e3} = C_{e2} + (s_p)(K_s)(S)/(Q_R)\ /(1+K_3t_3) \qquad (9)$$

s_p = fraction of anaerobic fermentation products which do not leave the pond as a gas

K_s = first order anaerobic reaction constant (d^{-1})

S = mass of sludge accumulated at the bottom of the pond (kg)

K_3 = aerobic first order reaction constant in reactor 3 (d^{-1})

t_3 = residence time in reactor 3 (d)

The value of K_3 will be estimated using equation (6), and t_3 will be determined using the equation

$$t_3 = t(1 - (\underline{b})(\underline{q}))/\underline{Q} \qquad (10)$$

In using equation (6) to determine K_3, the temperature of the liquid in that reactor will have to be estimated. Because the volume of reactor 3 includes the lower level of the pond, where cooler water resides, the liquid temperature in reactor 3 will be assumed lower than the liquid temperatures in reactors 1 and 2.

The discussion of liquid temperatures will be temporarily set aside in order to first discuss the unknown quantities found in equation (9). The first term to be discussed is the mass of sludge accumulation, S. The rate of change of sludge mass at the bottom of the pond can be described by the equation

$$dS/dT = (i_s)(C_o)(Q_o) - (K_s)(S) \qquad (11)$$

T = time

Expressed in finite difference form, the equation becomes

$$\Delta S/\Delta T = (i_s)(C_o)(Q_o) - (K_s)(\bar{S}) \qquad (12)$$

or

$$(S_2 - S_1)/\Delta T = (i_s)(C_o)(Q_o) - (K_s)(S_1 + S_2)/2 \qquad (13)$$

S_2 = sludge mass at time T_2 (kg)

S_1 = sludge mass at time T_1 (kg)

$\Delta T = T_2 - T_1$

Anticipating that the sludge mass S_1 is known at time T_1, and the objective is to determine the value of S_2 at time T_2, equation (13) is solved for S_2, yielding

$$S_2 = \frac{(C_o)(Q_o)(i_s)(\Delta T) + (S_1) \quad 1 - (K_s)(\Delta T)/2}{(1 + (K_s)(\Delta T)/2)} \qquad (14)$$

Assuming that C_o, Q_o, i_s, S_1, and ΔT are either known or established, the only unknown variable is K_s.

The variable K_s has been found to be highly sensitive to temperature. Below 17 ^oC, fermentation in the sludge is negligible, while at 23 ^oC or more, fermentation may become so intense that sludge may be propelled to the pond's surface according to Mara (1974). It is possible that the pond could become anaerobic if fermentation is intense enough.

Marais has generated an expression from a pond in South Africa that will be used to calculate K_s in this model.

$$dV_g/dT = (C_v)(s_g)(K_s)(S) \qquad (15)$$

V_g = volume of gas liberated (ft^3)

C_v = volume of gas liberated per unit mass BOD5 destroyed (ft^3/lb)

s_g = fraction of anaerobic end products which leave the pond as a gas (= $1 - s_D$)

The equation used by Marais to estimate the variable K_s is

$$K_{s(T_s)} = 0.002\ (1.35)\ T_s^{-20} \qquad (16)$$

T_s = average sludge temperature (^oC)

To use this equation, however, the temperature of the sludge layer must be estimated. Under very cold conditions, the sludge temperature may be 4 ^oC. the approximate temperature at which water achieves its maximum density. Under summer conditions, the sludge layer will remain cooler than the liquid at the top of the pond.

Based upon data taken from South African ponds, Marais (1970) developed a relationship between maximum daily pond surface temperature and average sludge layer temperature as follows:

$$(T_s - 4) = 0.777\,(T_T - 4) \tag{17}$$

T_T = maximum daily temperature at pond water surface (^oC)

If equations (3), (4), (7), and (9) are combined, the following equation results:

$$C_{e2} = \frac{(C_o)(1-i_s)(1+K_3t_3) + (s_p)(K_s)(S)/(Q_o)}{(1+\underline{p})(1+K_2t_2)(1+K_3t_3)(e^{K_1t_1}) - \underline{p}} \tag{18}$$

This is the equation which will be applied to predict effluent BOD5 concentrations when the flow is expected to stay toward the top of the pond.

As stated earlier, equation (18) will not be applicable when the flow travels toward the bottom of the pond. When flow travels toward the bottom of the pond in the warmer months, the orientation of the active zone is altered in the model. In this case, all three reactors have the pond bottom as one of its boundaries; therefore, anaerobic fermentation products are expected to enter each reactor in proportion to the bottom area which each reactor occupies.

The addition of anaerobic end products to the plug flow reactor complicates the equation which describes the activity within the reactor. To simulate this situation, the reactor is broken down into a large number of small plug flow reactors, with an equal amount of anaerobic end products being added at the beginning of each small reactor. The following equation reflects the alteration of the basic plug flow equation to include the addition of anaerobic end products:

$$C_e = C_i + \frac{(s_p)(K_s)(S)(\underline{a})(\underline{b})}{(Q_o)(1+\underline{p})(N)} \left(e^{-K_1t_1/N}\right) \tag{19}$$

N = the number of small plug flow reactors into which reactor 1 is divided

The effluent from the Nth reactor, which is equivalent to the effluent from the large reactor, can be evaluated by the equation

$$C_{e1} = C_{i1}\left(e^{-K_1t_1(N-1)/N}\right) + \frac{(s_p)(K_s)(S)(\underline{a})(\underline{b})}{(Q_o)(1+\underline{p})(N)} \quad \Sigma\left(e^{-K_1t_1/N}\right)^i \tag{20}$$

The summation in equation (20) is equivalent to

$$\Sigma\left(e^{-K_1t_1/N}\right)^i = \frac{(N)\left(e^{-K_1t_1/N} - e^{-K_1t_1}\right)}{K_1t_1} \tag{21}$$

Combining equations (20) and (21) yields the equation

$$C_{e1} = C_{i1}\left(e^{-K_1t_1(N-1)/N}\right) + \frac{(s_p)(K_s)(S)(\underline{a})(\underline{b})\left(e^{-K_1t_1/N} - e^{-K_1t_1}\right)}{(Q_o)(1+\underline{p})(K_1t_1)} \tag{22}$$

Previously developed equations can be used to calculate the variables in equation (22) except for t_1, which is calculated differently in the underflow and overflow cases. In the underflow case, the equation is

$$t_1 = t(\underline{a})(\underline{b})(1-\underline{g})/(1+\underline{p}) \tag{23}$$

In reactor 2, BOD5 is contributed by anaerobic end products and by the effluent from reactor 1. The completely-mixed equation which represents the activity in the reactor is

$$C_{e2} = \frac{C_{e1} + \dfrac{(s_p)(K_s)(S)(1-\underline{a})(\underline{b})}{(Q_o)(1+\underline{p})}}{1+K_2t_2} \tag{24}$$

The value of t_2 is expressed by the equation

$$t_2 = t(1-\underline{a}(\underline{b})(1-\underline{g})/1+\underline{p}) \tag{25}$$

Since reactor 3 is also assumed to be a completely-mixed reactor, the equation describing the activity in the reactor will be similar to equation (25). It is

$$C_{e3} = \frac{C_{e2} + \dfrac{(s_p)(K_s)(S)(1-\underline{b})}{(Q_o)(\underline{p})}}{1+K_3t_3} \tag{26}$$

The residence time t_3 is calculated using the equation

$$t_3 = t(1 - (b)(1-g))/p \tag{27}$$

If equations (4), (22), (24), and (26) are combined, the resulting equation and final form of the prediction model is:

$$C_{e2} = \frac{A + (B)(D + E + F)}{(1+K_2t_2)(1+p)(1+K_3t_3) - (e^{-K_1t_1(N-1)/N})(p)}$$

$$A = (C_o)(1-i_s)(e^{-K_1t_1(N-1)/N})(1+K_3t_3)$$

$$B = (s_p)(K_s)(S)/Q_o$$

$$D = (e^{-K_1t_1(N-1)/N})(1-b)$$

$$E = (a)(b)(e^{-K_1t_1/N} - e^{-K_1t_1})(1+K_3t_3)/(K_1t_1)$$

$$F = (1+K_3t_3)(1-a)(b)$$

CONCLUSION

The above prediction model has been developed to stress the importance of hydraulic routing and physical effects along with the usual basic design parameters. Use of the model requires a computerized approach and therefore a computer program has been developed and tested using actual pond data in an academic thesis by Wagner (1983). Initial model testing has indicated that the model and program have applicability where properly applied.

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AN ALTERNATIVE APPROACH TO THE DESIGN OF WASTE STABILIZATION PONDS

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ABSTRACT

Waste stabilization ponds are frequently used for reducing the bacterial concentration of wastewaters. An examination of their performance suggests that they are inefficient in removal due to a combination of mixing and the presence of organic compounds which reduce die-off rates.

An alternative approach to design is suggested which emphasises organic removal by methane production followed by light-induced die-off in an algal free aerobic pond.

KEYWORDS

Mixing; Bacterial Die-off; Algal Growth; Pond Design

OBJECTIVES OF DESIGN

Wastewater treatment plants are designed to achieve a number of different objectives, commonly one or all of the following:

a. Reduction of BOD
b. Reduction of Suspended Solids
c. Reduction of Nutrients
d. Reduction of Pathogens

It is difficult to judge the efficiency of waste stabilization ponds in reducing the oxygen demand since a large proportion of the organic matter is converted into algal cells. Simulations of the impact of a stabilization pond effluent on a stream has shown that the oxygen demand may exceed that of the untreated waste (Botero, 1979). From the viewpoint of reducing oxygen demand it appears that the conversion to methane in anaerobic ponds is the main contribution; the conversion to algal cells in facultative ponds is of doubtful benefit.

Similarly the growth of algae in facultative ponds can contribute significant amounts of Suspended Solids to the effluent so that the overall change in SS is small.

The efficiency of waste stabilization ponds in removing nutrients is also poor since much of the nitrogen and phosphorus leaves the pond system in the form of algae. If the nitrogen and phosphorus content of the algae are included in the effluent then removal efficiencies are much lower than conventional treatment.

It is in the removal of pathogens that stabilization ponds appear to be pre-eminent. Removals of Faecal Coliforms in conventional treatment are usually in the range 90 - 98% (Carrington, 1980) whereas removals in pond systems are often 99.9 - 99.999% (Mara, 1977). For this reason many pond systems are designed on the basis of pathogen removal. However it is doubtful if existing pond designs are efficient in bacterial removal. Comparison of bacterial removal and retention times as shown in Table 1 suggest that compared with other treatment processes ponds are remarkably ineffective.

TABLE 1 Comparison of Removal Rates in Different Wastewater Treatment Processes

Process	Retention Time	% Removal	Removal Rate
Primary Sedimentation + Activated Sludge	4 hours + 6 hours + 2 hours = 12 hours	95	20% per hour
Primary Sedimentation + Percolating Filter	4 hours + 1 hour + 2 hours = 7 hours	95	25% per hour
Waste Stabilization Pond	30 days	99.999	20% per day

The remainder of this paper considers the reasons for the poor removal rates and suggests an alternative approach to design which may be more effective.

MECHANICS OF BACTERIAL REMOVAL

In considering the efficiency of ponds in reducing the bacterial concentration it is useful to examine the mechanisms that may be involved. These are as follows:

a) Light-induced mortality - in some aquatic environments such as the sea, it is apparent that light is responsible for rapid die-off of bacteria (Gameson and Gould, 1985). But light penetration in stabilization ponds is limited to the top 10-15 cm and as the highest bacterial concentrations rarely occur in the surface layer it is unlikely that light is a major cause of death in ponds. This is confirmed by typical values for die-off rates in ponds (T_{90} values of 20 - 30 hours for E.coli) which approximate to dark values in freshwater and seawater experiments.

b) pH - induced mortality - a more recent explanation of bacterial death in ponds (Smallman, 1986) suggests that it is due to algal photosynthesis causing periods (6-12 hours per day) of high pH. Results from his dialysis experiments showed significantly higher mortalities at pH levels above 9. He also showed that pH rose to 9 - 10.3 during periods of intense photosynthetic activity.

c) Starvation-induced mortality - experiments on bacterial die-off in fresh and marine waters (Gameson, 1985) have shown that T_{90} values of 1 - 2 days occurred in the presence of low levels (< 20 mg/l) of organic matter in the absence of light, predators or other sources of mortality. When the concentration of organic matter was increased the T_{90} values increased to 2 - 3 days. Similar results from Le Moyne, 1972 indicate that at BOD levels above 20 - 30 mg/l growth of coliforms can occur at appropriate temperatures.

It would therefore appear to be important to maintain low organic concentrations (BOD $\leqslant$ 20mg/l) if starvation is to cause a rapid die-off.

d) Sedimentation - the primary stages in any pond system, especially where they are anaerobic, act in a similar manner to a primary sedimentation tank and remove around 50% of the incoming bacteria. It is doubtful whether this mechanism is significant in subsequent stages.

Fig. 1 presents a comparison of die-off rates in ponds and other freshwater environments. This shows that die-off in ponds is comparatively slow and this appears to be due to the presence of organic matter which is coming from the algae. Because of the tendency of algal cells to release organic compounds into the water it is impossible to obtain the low BOD concentrations required (< 20 mg/l) for rapid die-off by starvation.

In the absence of UV and with only modest elevation of pH level it appears that the death rate in stabilization ponds is due mainly to starvation and is limited by the organic concentration. To achieve higher death rates would need some or all of the following

i. Lower turbidities
ii. Higher pH
iii. Lower BOD

EFFECTS OF MIXING

Due to the low dilution rates the advective contribution to mixing in most ponds is negligible. It is wind action that is chiefly responsible for mixing. This is generally sufficient to ensure that ponds function as partly or completely mixed reactors and there is, therefore, a distribution of retention times. Fig. 1 shows the exit-age distribution from a gulp injection of tracer and shows that significant proportions of the flow have retention times of only 1, 2 or 3 days in a pond where the media retention is 5 days. Superimposed on the tracer curve is the corresponding concentration of bacteria using an initial concentration of 10^9/l and a die-off rate of 2.0 per day. These results can be superimposed to calculate the overall bacterial concentration in the effluent and also to show the contribution that each day's flow makes to the effluent quality. The results of the calculation are presented in Table 2.

Day	Bacterial Concentration at end of Day	Average Concentration during Day	Proportion of Effluent leaving Pond	Bacterial Contribution in Effluent x 10^8
0	1×10^9	-	-	
1	1.3×10^8	6×10^8	.188	1.13
2	1.8×10^7	7×10^7	.200	.14
3	2.5×10^6	8×10^6	.167	.013
4	3.3×10^5	1×10^6	.125	.0015
5	4.5×10^4	1.5×10^5	.094	.0001
6	6.1×10^3	2×10^4	.073	.000015
7	8.3×10^2	2.5×10^3	.056	.0000016
8	110	3×10^2	.038	.0000002
9	15	60	.029	.00000002
10	2	8	.017	.000000002
11	0.1	1	.007	.0000000002
Totals	-	-	.974	1.3

The results show that short-circuiting has an important effect in controlling the bacterial quality of the effluent from ponds. If the proportion of effluent leaving the pond after 1 day could be halved then the bacterial concentration in the effluent would be reduced by 40%.

If the pond could be made to approximate to a plug flow regime without changing the retention times, then the bacterial removal would be improved from less than 90% to almost 99.99%.

The importance of mixing has previously been mentioned by Marais, 1966 but its significance has not been sufficiently stressed in manuals on pond design (Arthur, 1984). Also the greater use of anaerobic ponds presents more opportunity to design systems which are less influenced by wind-induced mixing.

The effects of the mixing pattern are not limited to their direct action on bacterial quality. Completely mixed ponds dilute the organic concentration of the influent immediately which tends to limit or prevent any bacterial re-growth but is also ensures a minimum level which is generally sufficiently high to prevent rapid starvation.

The tendency to complete mix also influences the pattern of algal growth in that the spread of retention times allows the maintenance of algae even in ponds with mean retention periods as low as 3 - 5 days.

ALTERNATIVE APPROACH TO DESIGN

In approaching the design of waste stabilization ponds it is useful to summarise the salient points that have emerged from the above discussion.

a. Light plays little part in increasing the death rate directly or indirectly through raising the pH.
b. Die-off rates in ponds are generally lower than those obtained in fresh or saline waters in the dark. Comparable rates are obtained only when the waters contain organic matter above

BOD levels of 20 mg/l.
c. The presence of algae appears to be undesirable for a number of reasons. Excretion of organic metabolites prevents the BOD from falling to starvation levels thereby encouraging bacterial survival. Also algae add significantly to the suspended solids and the oxygen demand of the effluent.
d. The most successful mechanism of BOD removal is conversion to methane in anaerobic ponds. Conversion to algal cells is not effective in improving the quality of pond effluents.

The first part of a pond system should therefore be a two-stage anaerobic reactor, preferably plug flow which would remove the majority of the BOD as methane.

The use of an anaerobic stage has many advantages. Much greater depths can be used thus reducing the land area required. Also deeper ponds are less susceptible to wind-induced mixing. In this way it is possible to obtain BOD removals that are sufficient to ensure starvation conditions in the subsequent facultative stage. It would be necessary to use a two-stage anaerobic pond system to do this but this need not be a disadvantage since the overall retention time and land area will still be much less than conventional designs.

The overall aim of the anaerobic stage should be to reduce the BOD to less than 40 mg/l. This would enable the second facultative stage to operate at a retention time of 3 - 4 days whilst still providing the low BOD concentration to cause starvation. Assuming a decay rate of 0.2 per day and a retention time of 4 days the BOD of the effluent would be

$$40 \times e^{-1.2} = 15 \text{ mg/l}$$

This short retention time would prevent the development of a significant algal population.

This second stage would be designed to be aerobic. Since algal photosynthesis would not contribute to the oxygen budget the design would be based upon the BOD loading not exceeding the surface reaeration capacity. Obviously wind velocities and frequencies will largely determine reaeration rates but the values of K_2 (reaeration coefficient) in quiescent conditions vary between 0.2 and 0.9 per day and suggest that the allowable depth for aerobic conditions would be at least 1 metre. (Padden and Gloyna, 1971).

The relative shallowness of this facultative stage would have the following secondary benefits:

a. Increased rate of bacterial mortality due to light. Because of the low algal levels light penetration would give significantly enhanced die-off.
b. Maximise mixing due to wind action which would reduce any tendency to short-circuiting.

CONCLUSIONS

It seems unlikely that there is an optimum design of waste stabilization pond to suit all conditions and it therefore appears worthwhile to approach pond design in a variety of ways.

This alternative approach is an attempt to approach pond design by emphasising the importance of mixing and organic content in controlling the bacterial die-off. The importance of methane production in BOD removal is emphasised. The undesirability of algae is also stressed both in encouraging the survival of bacteria and in creating an oxygen demand in the pond effluent.

A simple comparison of the alternative design shows that it would require a smaller land area (60 - 70%) than the conventional design.

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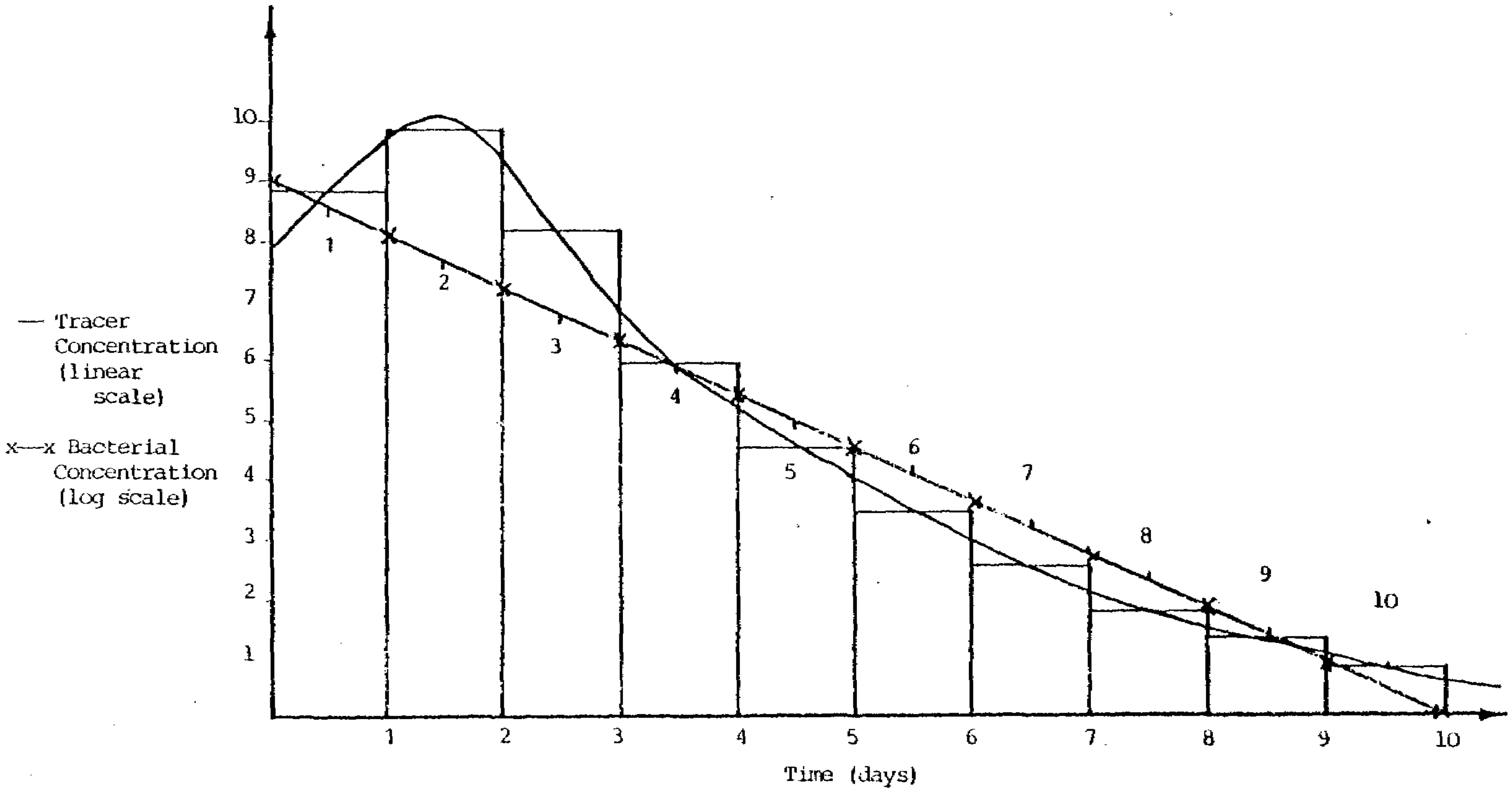

Fig. 1 Exit-age distribution and bacterial die-off in a partly mixed stabilization pond

ATTACHED-GROWTH WASTE STABILIZATION POND TREATMENT EVALUATION

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ABSTRACT

This study investigated the feasibility of improving waste stabilization ponds (WSP) performance through the addition of attached-growth media in the pond water. An artificial media consisting of fine strings of polyvinylidene chloride was empolyed as an attached-growth media in the laboratory-scale and pilot-scale experiments. Better removal of organic, nutrient and suspended solids were obtained in the attached-growth waste stabilization ponds (AGWSP). The AGWSP were found to be rather stable against increased hydraulic loadings. However, the fecal coliform die-offs in the AGWSP units were not significantly different from those of the control units without attached-growth media.

KEYWORDS

Waste stabilization ponds; attached media; film thickness; organic removal; fecal coliform die-offs; kinetic models.

INTRODUCTION

Conventional waste stabilization ponds (WSP) are a common method of wastewater treatment for small communities and several types of industry. However, some drawbacks of WSP, especially in facultative and maturation ponds, include the presence of low microorganism density in the pond water. Therefore, WSP are normally designed and operated at relatively low organic loadings, resulting in a large land area requirement for pond construction and the sensitivity of pond performance to increased or shock loadings. This study was undertaken to investigate the feasibility of improving WSP performance through the addition of attached-growth media in the pond water. The WSP incorporating attached-growth media are called "attached-growth waste stabilization ponds" (AGWSP). The attached-growth media served as habitat for the growth of attached-growth bacteria and algae.

METHODS

An artificial media consisting of fine strings of polyvinylidene chloride was employed in the experiment. The media modulus of each string is 0.09 mm and the rope woven's diameter is 2 cm. The specific surface area and specific volume of the media are 7.33 m^2/m and 0.3 L/m, respectively (Seo, 1986). The experiments were conducted using 4 laboratory-scale (working dimensions = 0.40 x 0.20 x 0.15 m : length x width x depth, located indoor, temperatures = 22 ∿ 25°C) and 1 pilot-scale (working dimensions = 4.0 x 2.0 x 1.0 m : length x width x depth, located outdoor under ambient condition, temperatures = 25 ∿ 33°C) AGWSP units. Control laboratory-scale and pilot-scale units without attached-growth media were run in parallel for data comparison. The densities of attached-growth in the laboratory-scale AGWSP units were varied at 5, 10, 20 and 40% of the reactor volume, and organic loadings (OL) applied to these reactors were varied at 30, 50, 100 and 200 kg COD/ha-day. The hydraulic retention time (θ) of the laboratory-scale units was controlled at 5 days, while the pilot-scale units were operated at θ of 5 and 10 days. Because of the limited quantity of media available, the pilot-scale AGWSP unit was installed with only 5% of the attached-growth media, but the OL varied at 100 and 200 kg COD/ha-day. A synthetic wastewater (glucose as main carbon source) having COD concentrations adjustable from 100 to 600 mg/L was utilized as the influent feed to avoid fluctuation of the influent characteristics to the laboratory-scale units. Film thickness of the attached media was determined microscopically by placing a string under a microscope equipped with a micrometer: the measurements were done three times and the average reported for each data point. Dissolved oxygen (DO) and pH were measured by DO meter (TOA Co.) and pH meter (Beckman Co.), respectively. Other analyses were determined using the procedures outlined in Standard Methods (APHA, 1981): chemical oxygen demand (COD), dichromate reflux method; suspended solids (SS), filtration method; ammonia nitrogen (N), preliminary distillation followed by acidimetric method; organic nitrogen, kjeldahl method; nitrite nitrogen, diazotization method; nitrate nitrogen, copper-cadmium reduction method; phosphorus (P), stannous chloride method; fecal coliform, most probable number (MPN) method; chlorophyll a, trichromatic method; heterotrophic bacteria density, standard plate count method.

All pond units were firstly acclimated by being fed with a mixture of a facultative pond effluent (10%) and the synthetic wastewater (90%). Then, the laboratory-scale and pilot-scale AGWSP units were continuously fed with the synthetic wastewater and campus wastewater, respectively. The acclimation period lasted approximately 30 days. A steady state condition was considered to exist when the filtered COD concentrations of the AGWSP effluents became relatively constant for a period of 7 days. All experimental data reported were obtained from the AGWSP units operating under steady state conditions. The campus wastewater having a filtered COD concentration of 80-100 mg/L was used as feed to the pilot-scale units. Figure 1 shows the arrangement of a laboratory-scale AGWSP unit.

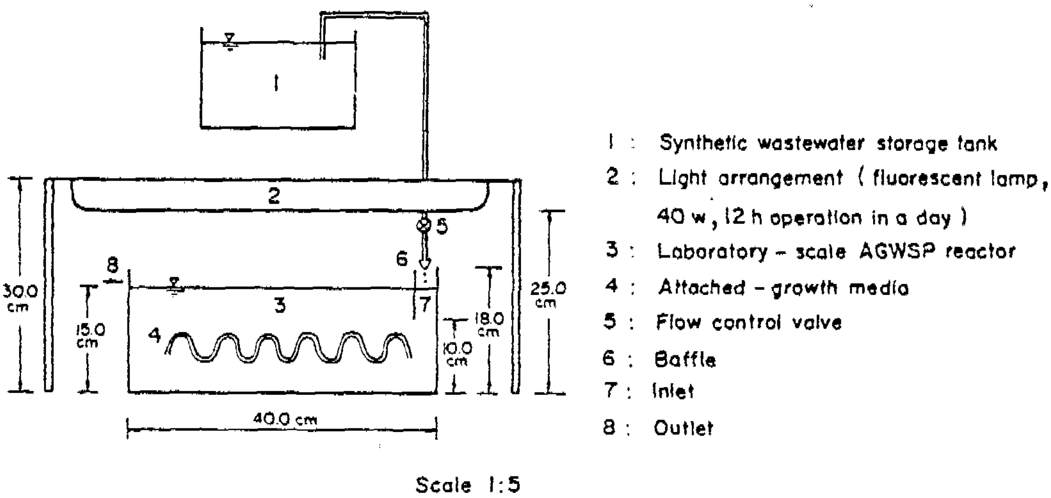

Fig.1 . Schematic profile of laboratory - scale AGWSP unit

RESULTS AND DISCUSSION

Biomass Development

Biofilm thickness. Thickness of biofilm growth on the attached-growth media was found to linearly increase with time during the initial stage of operation (up to 30 days) due to the higher attachment and growth rate of biomass. The biofilm thickness later became constant, being about 60-65 μm. Organic loadings seemed to have some effects on the biofilm thickness. For the laboratory-scale AGWSP units operating at organic loadings of 50 and 30 kg COD/ha-day, the corresponding biofilm thickness were found to be 61 and 58 um, respectively. The biofilm in the pilot-scale AGWSP unit operating at 100 kg COD/ha-day had thickness ranging from 60-80 µm, a little higher from that of the laboratory-scale unit. The SS of the campus wastewater being fed to the pilot-scale AGWSP units might be attached to the attached-growth media, attributing to an increase in biofilm thickness.

Chlorophyll a and heterotrophic bacteria. The effects of varying the percentage of attached-growth media in the laboratory-scale AGWSP units with respect to biomass composition are shown in Table 1. It can be seen that the attached-growth heterotrophic bacteria number increased with increasing percentage of media, while the opposite occurred with dispersed-growth heterotrophic bacteria. A similar trend was found with the chlorophyll a data also. These phenomena are understandable because increasing the media volume provided more surface area for the growth of attached bacteria and algae, but at the same time it interfered with the light penetration and algae-bacterial symbiotic reactions in the pond water. Hence, the decreases in the number of dispersed-growth heterotrophic bacteria and algae (or chlorophyll a). However, as shown in Table 1, the overall densities of heterotrophic bacteria and chlorophyll a were found to increase with increasing the percentages of media volume, indicating the beneficial effects of the attached media in yielding more biomass growth for waste stabilization.

Table 1 DO, Heterotrophic Bacteria and Chlorophyll a Concentrations in Laboratory-Scale AGWSP Units*

Media (%)	DO (mg/L)	Biomass (mg/L)			Heterotrophic Bacteria (No./mL)			Chlorophyll a (mg/L)		
		Attached-Growth	Dispersed-Growth	Total	Attached-Growth	Dispersed-Growth	Total	Attached-Growth	Dispersed-Growth	Total
0	1.36	-	176.1	176.1	-	7.2×10^6	7.2×10^6	-	2.37	2.37
5	1.30	48.0	168.3	216.3	4.3×10^6	7.4×10^6	12.3×10^6	0.28	2.32	2.60
10	1.19	92.8	177.3	270.1	7.3×10^6	6.3×10^6	14.1×10^6	0.60	2.52	3.12
20	0.44	191.6	104.0	295.6	11.4×10^6	6.9×10^6	18.3×10^6	1.07	2.19	3.26
40	0.51	278.2	99.0	377.2	15.4×10^6	5.8×10^6	21.2×10^6	1.71	2.04	3.75

*measure at 10:00 a.m. with an OL of 100 kg COD/ha.day

Based on the morphological examination, the dominant bacterial species of both attached and dispersed types is the gram positive, small rod having average length of 1-2 µm. Chlorella was observed to be the dominant algal species of both the attached and dispersed types in the AGWSP units. The DO concentrations in the AGWSP units were observed to decrease with increasing percentage of attached-growth media (Table 1), because higher bacterial biomass required more oxygen to oxidize the organic matter. The pH in all units of AGWSP were satisfactory, ranging from 8.0-8.4.

Performance of AGWSP

COD removal. The data of COD removal at various organic loadings and media volumes are shown in Table 2. As can be expected, there is an optimum condition in the AGWSP system that where maximum COD removal can be achieved. It appears from the data in the AGWSP units having a 10% media volume was most effective in COD removal. Although the AGWSP units with 20 and 40% media volumes had more biomass (Table 1), the attached media protected most part of the pond volume from light penetration. Therefore, less algal photosynthesis occurred, leading to less oxygen production, and consequently insufficient oxygen supply to the bacteria or biomass responsible for organic matter oxidation. It seems clear from Table 2 that the 40% attached media volume in the AGWSP units did not bring much benefit in terms of COD removal, while the 20% of attached media volume produced COD removal results more or less similar to those of the optimum condition, i.e. the 10% attached media volume. Table 2 shows that the pilot-scale AGWSP units had slightly higher percentages of COD removal than those of the laboratory-scale units when being operated at the same conditions. This results were probably due to the higher temperature, wind aeration and the presence of natural sunlight available to the pilot-scale AGWSP units which could enhance more biological activities in the pond water. The higher percentage of COD removal in the pilot-scale AGWSP units with 5% attached media than those of the control units was due to higher biomass present in the pond water, similar to the results of the laboratory-scale units.

TABLE 2 COD Removal at Various Organic Loadings and Media Volumes

Media (%)	* COD Removal (%)			
	OL 30 kg COD/ha-day	OL 50 kg COD/ha-day	OL 100 kg COD/ha-day	OL 200 kg COD/ha-day
0	76.2	72.8	74.4 (80.8)	66.8 (68.1)
5	80.0	77.0	79.2 (85.6)	72.2 (74.1)
10	80.1	77.7	82.0	74.1
20	80.5	77.0	78.6	70.2
40	78.1	76.8	76.1	67.4

* (   ) = pilot-scale data; θ = 5 days, except the pilot-scale data obtained from an OL of 100 kg COD/ha.day (θ = 10 days)

Nutrient and SS removal. The removal of total N (from 40 to 62%) and total P (from 40 to 59%) were found to increase with increasing the percentage of attached media volume. According to Oswald and Gotaas (1955), most of the total N that were removed would be converted into biomass production, as shown in Table 1. Since the pond water pH were between 8.0-8.4, some of the total N could be lost through ammonia volatilization (Sawyer and McCarty, 1978), and a small portion through nitrification and denitrification. Phosphorus would be partly incorporated into the biomass growth, and some settled down to the bottom of the pond under the slightly alkaline condition of pond water. The effluent SS concentrations of the laboratory-scale AGWSP units were found to decrease with increasing the percentage of media volume in the pond water and decreasing organic loading. The range of SS concentration was 90 - 180 mg/L. The effluent SS concentrations of the pilot-scale AGWSP units were about 50 - 80 mg/L. These relatively low SS concentrations in the pilot-scale AGWSP units could be due to several reasons such as the pond depth, the influent SS characteristics and other environmental factors. However, all the control units were found to produce effluents having SS concentration about 10% higher than those of the AGWSP units.

Fecal coliform removal. During the investigation of fecal coliform removal, the raw campus sewage having fecal coliform density in the range of 2.2×10^7 - 4.8×10^7 MPN/100 mL was used as feed to the acclimated laboratory-scale AGWSP. The results shown in Table 3 indicated that there were about 2-3 log orders of fecal coliform removal in both the laboratory-scale and the pilot-scale units. The attached-growth media did not appear to yield significant benefits with respect to fecal coliform removal in the AGWSP system. Although a large number of fecal coliform cells would be clumped or attached to the media surface, the media volume also produced shading effects, preventing the direct exposure of the fecal coliform cells to the ultraviolet (UV) rays of the fluorescent lamp and sunlight in cases of the laboratory-scale and pilot-scale units, respectively. However, θ was found to have a direct effect on fecal coliform die-off in which the Run-2 data (θ = 10 days) consistently had fecal coliform densities lower than those of the Run-1 (θ = 5 days).

TABLE 3 Fecal Coliform Die-offs in AGWSP Units

Media (%)	Influent Fecal Coliform Range, MPN/100 mL	Effluent Fecal Coliform, MPN/100 mL			
		Lab-scale		Pilot-scale	
		*Run 1	**Run 2	*Run 1	**Run 2
0	2.2×10^7	1.7×10^5	1.1×10^4	4.2×10^5	1.5×10^4
5		2.1×10^5	1.5×10^4	8.2×10^5	7.1×10^4
10	to	5.6×10^5	4.9×10^4	-	-
20		7.9×10^5	8.1×10^4	-	-
40	4.8×10^7	3.2×10^6	4.3×10^5	-	-

*θ = 5 days; **θ = 10 days

Increased Hydraulic Loading Response of AGWSP

The laboratory-scale AGWSP units were tested for their responses to increased hydraulic loadings. At each influent COD concentration (i.e. 150, 300 and 600 mg/L), the hydraulic loadings were varied at 150, 250, 500 and 1,000 m^3/ha-day, corresponding to the θ values of 10, 6, 3 and 1.5 days, respectively. Figure 2(a) shows the effluent filtered COD concentrations of the AGWSP units subjected to increased hydraulic loadings, comparing with the control units without attached-growth media. The AGWSP units consistently produced the effluents having COD concentrations lower than those of the control. The degree of performance stability of the AGWSP units against increased hydraulic loadings was found to increase with increasing influent COD concentrations (or higher organic loadings). In case of the AGWSP unit receiving the influent COD concentration of 600 mg/L and operating at the θ value of 1.5 days, the effluent COD value was 265 mg/L which was 55 mg/L lower than that of the control unit. To interpret the data of Figure 2(a) into a kinetic term, a Monod equation was utilized as follows (Ball, 1983):

$$R = \frac{PS}{K_s + S} \qquad (1)$$

Where R = substrate removal per unit pond surface area, kg COD/ha-day; P = maximum substrate removal per unit pond surface area, kg COD/ha-day; K_s = pseudo-half velocity coefficient, mg/L; and S = effluent COD concentration, mg/L.

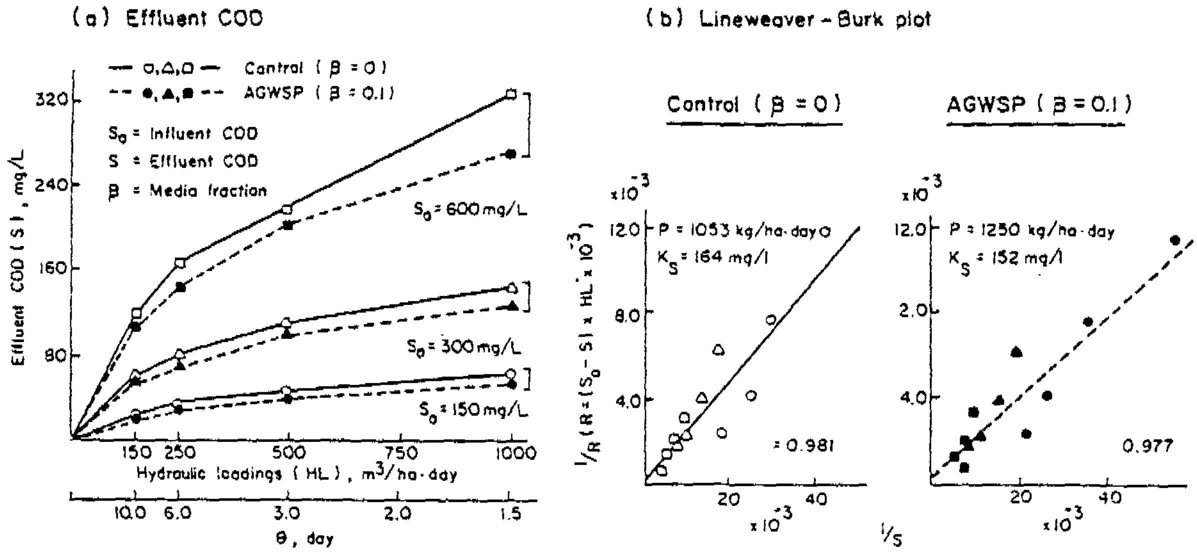

Fig. 2. Effect of increased hydraulic loadings on COD removal in the laboratory - scale AGWSP units

Rearranging Equation (1), the following (Lineweaver-Burk plot) can be written:

$$\frac{1}{R} = \left(\frac{K_s}{P}\right) \frac{1}{S} + \frac{1}{P} \qquad (2)$$

Using the data in Figure 2(a), the Lineweaver-Burk plots of the performance of both the AGWSP and control units were made (Figure 2(b)). It was found that the P value of the AGWSP unit (with 10% attached-growth media) was 1,250 kg COD/ha-day which is 197 kg COD/ha-day higher than that of the control unit. The higher P value of the AGWSP suggests the beneficial effects of the attached-growth media or the attached-growth microorganisms present in the pond water in the stabilization of organic matter and resistance against increased hydraulic loadings.

Kinetics of AGWSP Treatment

Because a relatively low θ normally encounters in a WSP, the flow conditions in WSP are neither completely-mix nor plug-flow, but rather partially-mixed flow (Polprasert and Bhattarai, 1985). A conceptual model of an AGWSP system is shown in Figure 3.

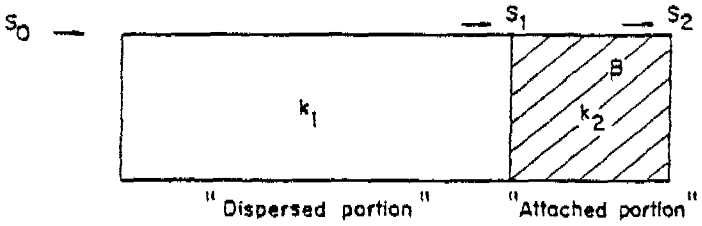

Fig. 3 . Conceptual model of AGWSP system

The shaded area in Figure 3 is assumed to represent the attached-growth portion of the AGWSP system, while the unshaded area represents the dispersed-growth portion of the system. According to Levenspiel (1972) and Wehner and Wilhelm (1956), Equation (3) can be used to show the organic removal occurring in the dispersed-growth portion of the AGWSP in Figure 3.

$$\frac{S_1}{S_o} = \frac{4a \exp(1/2d)}{(1 + a)^2 \exp(a/2d) - (1 - a)^2 \exp(-a/2d)} \tag{3}$$

where S_1 and S_o = effluent and influent COD concentrations of the dispersed-growth portion, mg/L; d = dispersion number, dimensionless; $a = \sqrt{1 + K_1 (1 - \beta)\theta d}$; k_1 = COD removal rate, day^{-1}; and β = fraction of attached-growth media volume in AGWSP. The COD removal in the attached-growth portion of the AGWSP system is assumed to follow a fixed-film model proposed by Eckenfelder (1966):

$$S_2/S_1 = \exp(-K_2 S_A^y Q_v^{-z}) \tag{4}$$

where S_2 and S_1 = effluent and influent COD concentrations of the attached portion, mg/L; k_2 = COD removal rate, day^{-1}; S_A = specific surface area of attached-growth media, m^2/m^3; $Q_v = Q/\beta V$, volumetric flow rate per unit volume, $m^3/m^3.day$; Q = flow rate, m^3/day; V=reactor volume, m^3; y and z = empirical constants. Since S_A is constant for a certain media, Equation (4) can be simplified as follows:

$$S_2/S_1 = \exp(-K_2' Q_v^{-z}) \tag{5}$$

where $K_2' = K_2 S_A^y$

Combining Equations (3) and (5) and incorporating the temperature factor, the overall COD removal ratio for an AGWSP system is obtained as follows:

$$\frac{S_2}{S_o} = \frac{4a \exp(1/2d) \exp(-K_2'' \ 1.08^{T-20} Q_v^{-z})}{(1 + a)^2 \exp(a/2d) - (1 - a)^2 \exp(-a/2d)} \tag{6}$$

where $a = \sqrt{1 + 4k_1' \ 1.08^{T-20} (1 - \beta)\theta d}$; T = temperature, ^oC; k_1' and k_2'' = COD removal rate at 20^oC, day^{-1}.

The experimental data obtained from laboratory-scale experiments were used for the multiple regression analysis to determine the constant values of k_1', k_2'' and z in Equation (6). The overall regression coefficient (γ) was 0.92. Substituting these constant values into Equation (6) the following equation is obtained:

$$\frac{S_2}{S_o} = \frac{4a \exp(1/2d) \exp(-(0.35) \ 1.08^{T-20} Q_v^{-0.44})}{(1 + a)^2 \exp(a/2d) - (1 - a)^2 \exp(-a/2d)} \tag{7}$$

where $a = \sqrt{1 + 4(0.20)1.08^{T-20}(1 - \beta)\theta d}$

The dispersion number (d) of AGWSP units with $\beta = 0.05$ and $\theta = 5$ days was reported to be 0.17, similar to that of the control unit (Kugaprasatham, 1987). Equation (7) was validated with other sets (22 data points) of the laboratory-scale and pilot-scale data. Most of the validated data were found to be within the 95% confidence limits of Equation (7) with a correlation coefficient of 0.94.

SUMMARY AND CONCLUSIONS

The experimental results obtained from this study showed that improved performance of WSP could be achieved through the installation of attached-growth media in the pond water. The media enhanced the growth of attached biomass, increasing the biomass volume in the pond water and consequently leading to better organic and nutrient removal. The attached-growth media also attached or adsorbed some of the dispersed-growth microorganisms and other SS particles, thus less SS concentrations present in the AGWSP effluents. The AGWSP units were found to have better resistance to increased hydraulic loadings than those of the control units. However, a distinct advantage with respect to fecal coliform removal could not be observed, probably because of the clumping of fecal coliform cells to the attached-growth media and some shading effects of the media itself. The installation of attached-growth media would involve an additional capital cost. The media would need to be cleansed periodically to remove the excess scum and other non-volatile solids accumulation into the media surface, hence another operational cost. The conclusions to be made from this experiment are as follows:

1. The attached-growth media volume of 10% of the pond volume was found to be optimum for the AGWSP units to achieve the maximum COD removal. However, the more the percentage of attached-growth media, the higher the overall biomass (dispersed plus attached) growth in the AGWSP system.

2. At the θ value of 5 days, organic loading of 100 kg COD/ha-day, and optimum media volume, the percent COD removal in the AGWSP units was found to be 82%, which was 8% higher than that occurring in the control unit without attached-growth media.

3. N and P removal in the AGWSP units were found to increase with increasing percentage of attached-growth media due to their incorporation into the cell biomass development.

4. Fecal coliform removal in the AGWSP units was found to be more or less comparable to those in the control units.

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THE HYDRAULIC PERFORMANCE OF WASTE STABILIZATION PONDS IN PORTUGAL

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ABSTRACT

The hydraulic regime in two Portuguese facultative waste stabilization ponds, a primary facultative pond receiving raw sewage (in Portimão) and a secondary pond receiving the effluent from an anaerobic pond (in Vidigueira) was studied by means of a fluorescent dye tracer technique. The resulting dispersion numbers, respectively for winter and summer, for each pond were as follows: 0.595 and 0.371 in Portimão, and 0.574 and 0.523 in Vidigueira. These results did not agree with those obtained from the available predictive equations in the recent literature. From the point of view of design it is concluded that for ponds geometrically similar to those investigated in this study the equation of Wehner-Wilhelm can be used with a dispersion number of 0.6.

INTRODUCTION

Waste stabilization ponds (WSP) are biochemical reactors whose main function is to remove dissolved and settleable pollutants, as well as excreted pathogens. The treatment efficiency of WSP depends on many interrelated factors, for example the nature of wastewater, the hydraulic and organic loading rates and environmental conditions such as temperature and solar radiation intensity. However, another very important factor which has been given little attention in the past by design engineers, is the hydraulic regime within WSP. The hydraulic transport processes within a WSP are controlled by the shape of the pond, the positioning of the inlet(s) and outlet(s), the presence of dead spaces and the degree of wind-induced mixing. These factors affect the dispersion and the average retention time within a pond, and consequently its removal efficiency of organic matter (BOD) and pathogens.

DESIGN MODELS FOR WSP

An important consequence of the complex interrelation of these different factors affecting WSP performance is the current absence of a comprehensive design model. At the present time design practice commonly follows one of the following two approaches:

(a) an empirical BOD loading approach based on the data collected from existing WSP systems;

(b) a rational approach which attempts to model pond performance using the kinetic theory of chemical reactors.

Both these approaches have advantages and disadvantages. The empirical approach is simpler but does not permit the extrapolation of pond performance data to all locations and environmental conditions; it is however possibly the safer method at the current state of knowledge. On the other hand the rational approach has not yet produced a model of general applicability, in spite of the fact that is the more promising approach. Since WSP are biochemical reactors a rational approach must combine basic hydrodynamic laws (equations of continuity and motion) and the methods of chemical reactor design in order to be able to describe the mass pollutant transport and removal in the assumed type of reactor (batch or continuous flow) under the assumed hydraulic regime (ideal or non-ideal flow, steady or unsteady flow).

The biochemical processes within WSP are continuous rather than batch, and the comparison of a WSP to a steady flow reactor is a realistic approach and one which has been assumed by many authors. The hydraulic flow regimes assumed by past workers to occur in WSP are complete mixing, plug flow (both describing ideal flow conditions) and dispersed flow (which describes non-ideal, or real, flow conditions). More recently [1] there has been an attempt to model WSP performance in a comprehensive way through a combined or finite stage model which represents the mixing processes in a pond by a network of combinations of completely mixed, shortcircuiting plug flow and dead flow stages. However, finite stage models require many input parameters for which values may not be readily available, and they also require fairly sophisticated computational analysis for their solution. They are not, therefore, generally suitable for design purposes at their current state of development. The dispersed flow model is therefore a better approach to describe the hydraulic regime in WSP, and one that has been supported by several authors [3,4,5]. In the simplified one-dimensional dispersed flow model, the diffusion is proportional to the concentration gradient (Fick's law), and the steady-state situation for first order removal is described by the Wehner-Wilhelm equation in which the dispersion number (δ) characterises the degree of non-ideality of flow within the pond ($\delta = 0$ for plug flow, and $= \infty$ for complete mixing).

DETERMINATION OF POND DISPERSION NUMBERS

The method for estimating δ in a pond is based on the stimulus-response technique, by injecting instantaneously a tracer in the pond influent. Turbulent dispersion and mixing will cause an expansion of the cloud of the slug of tracer travelling downstream according to the convective movement of the fluid and consequently the concentration of the tracer will decrease, as shown in Fig. 1.

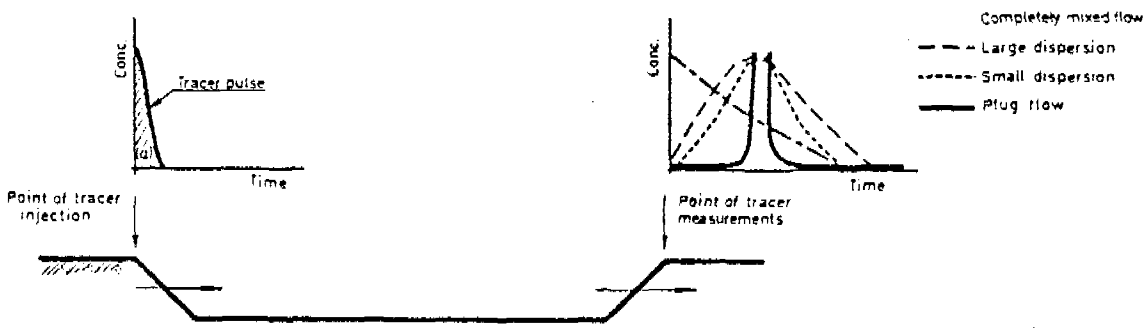

Fig. 1. The stimulus-response technique for determining dispersion numbers (the shape of curve (a) changes with the magnitude of dispersion)

Different amounts of dispersion will produce different spreading patterns in the tracer curve of non-ideal flow, as shown in Fig. 2. The boundary conditions, imposed by the flow conditions at the injection and measurement points of the tracer, influence the shape of the curve, since they influence longitudinal dispersion. Various analytical methods have been suggested for the determination of δ from the concentration curves. Levenspiel [6] developed a method of calculating the value of δ from the variance of the dimensionless time θ versus dimensionless concentration C_θ. The variance of this family of curves in the case of closed vessels (such as WSP) was found by Levenspiel to be:

$$\sigma_\theta^2 = 2\delta - 2\delta^2(1 - e^{1/\delta}) \qquad (1)$$

Equation 1 is the normally used by research workers to analyse their concentration-time data. However, Levenspiel clearly states that it is valid "if the continuous distribution function is measured only at a number of equidistant points". In other words, this means that equation 1 can only be used when the time interval between successive effluent samples is

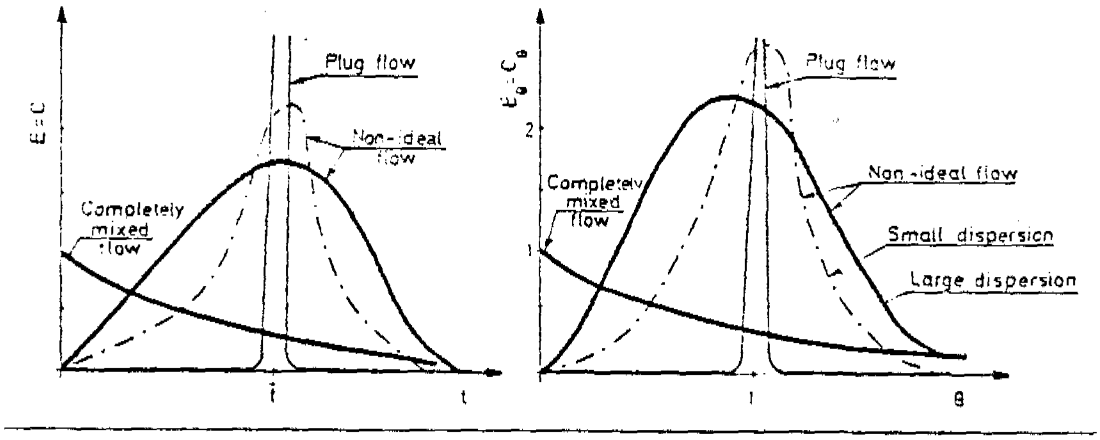

Fig. 2. Typical C - E curves as a function of time t and dimensionless time θ

essentially the same. (This was so in three of the experiments described herein, but in the winter experiment at Portimão it was not.)

When the time interval between successive samples is not constant, a modified version of equation 1 can be derived as follows, where an approximation considering non-equidistant values and normalized distribution is used:

$$\bar{t} = \int t.E(t).dt \approx \frac{\sum_{i=2}^{n} \frac{t_i+t_{i-1}}{2} \cdot \frac{c_i+c_{i-1}}{2} \cdot (t_i-t_{i-1})}{Q} \qquad (2)$$

$$Q = \int c dt \approx \sum_{i=2}^{n} \frac{c_i+c_{i-1}}{2} \cdot (t_i-t_{i-1}) \qquad (3)$$

$$\sigma_\theta^2 = \frac{\sum_{i=2}^{n} \left(\frac{t_i+t_{i-1}}{2}\right)^2 \cdot \frac{c_i+c_{i-1}}{2} \cdot (t_i-t_{i-1})}{Q(\bar{t})^2} - 1 \qquad (4)$$

Equation 4 for σ_θ^2 was used to obtain the values given in Table 3 for the "modified Levenspiel" model. Moreover it was found that, for the Portimão winter experiemnt, the Levenspiel equation (eq. 1) was not convergent at values of σ^2 above 0.922: the function began to oscillate widely above this value, and a solution to the equation was not possible with a value of σ^2 of 2.4, the value of that experiment. This demonstrated, in the case of non-equidistant sampling intervals, the greater appropriateness of equation (4).

TRACER SELECTION

Tracers suitable for use in aquatic environments include floats, chemical salts, radioisotopes, fluorescent and nonfluorescent dyes and microbes. The evaluation of the suitability of a tracer for water bodies [7] concerns some characteristics such as detectability, toxicity to man and aquatic organisms, the effect of water chemistry, photochemical and biological decay rates, adsorption losses on equipment and sediments, and cost. Fluorescent dyes present a major advantage as tracers for aquatic environments: they are detectable at very low concentrations (such as 0.01 μg/l) which are generally not toxic.

Waste stabilization ponds are water bodies with a high content of microorganisms, organic matter and SS and they are exposed to sunlight for long periods. Therefore the selected fluorescent tracer must present a low photochemical decay, a high resistance to adsorption on SS and organic matter, and a low biodegradability. Finally it should not be prohibitively expensive. A review of the available fluorescent dyes led to the conclusion that sulpho-rhodamine B was the most appropriate for use as a tracer in WSP.

EXPERIMENTAL METHODS

Experimental ponds

Two facultative ponds belonging to the WSP systems in Vidigueira and Portimão were investigated. Vidigueira is a rural town with a population of 6000, located at 38°19'N and 7°50'W and is 300 m above m.s.l. The Vidigueira WSP system comprises an anaerobic pond and a facultative pond which are shown schematically in Fig. 3; their dimensions are given in Table 1. Portimão

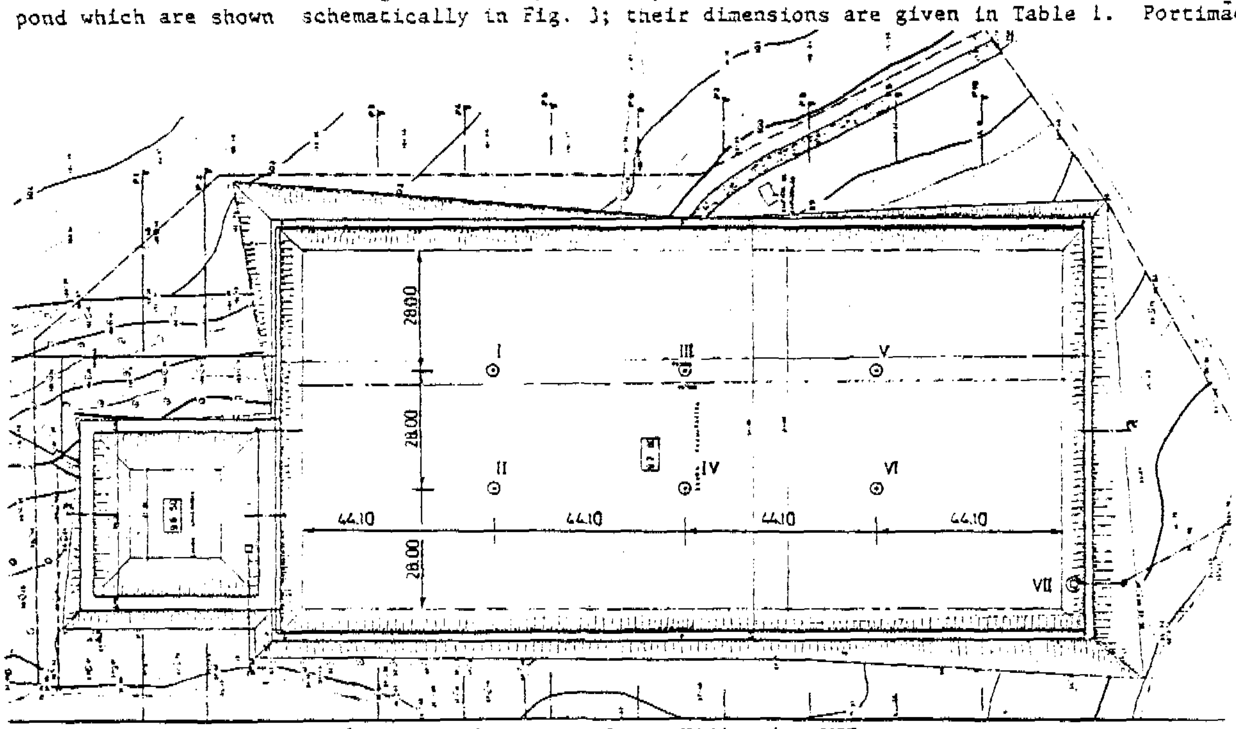

Fig. 3. General scheme of the Vidigueira WSP system

TABLE 1. Geometry of the Vidigueira WSP system

Geometry	Anaerobic Pond	Facultative Pond
Shape	Square	Rectangular
Top dimensions	38.5 m x 38.5 m	94.5 m x 186.5 m
Liquid surface	36.0 m x 36.0 m	92.0 m x 184.0 m
Middle depth	27.0 m x 27.0 m	88.3 m x 180.3 m
Bottom	21.0 m x 21.0 m	84.5 m x 176.5 m
Total depth	3.5 m	2.0 m
Liquid depth	3.0 m	1.5 m
Inside bank slope (v:h)	1:2	1:2.5
Volume	2,187 m^3	23,860 m^3
Mid-depth area	728 m^2	1,6 ha

is a tourist resort, located at 37°7'N, 8°32'W and 21 m above m.s.l., with a strong seasonal variation in population: 18,000 in summer and only 7,000 in winter. Its WSP system consists of a single primary facultative pond followed by a maturation pond (Fig. 4, Table 2).

Tracer studies

Both facultative ponds were studied for two different seasonal periods: 9 weeks in winter and 10 weeks in summer for the Vidigueira pond; and 6 weeks in both summer and winter for the Portimão pond. A slug of sulpho-rhodamine B dye (at sufficient concentration to give an overall concentration in the pond of 10 µg/l) was added to the influent flow, and grab samples were taken of the pond effluent and from various positions within the pond (Figs. 3 and 4). The timing of sampling for tracer analysis was altered after the first experimental

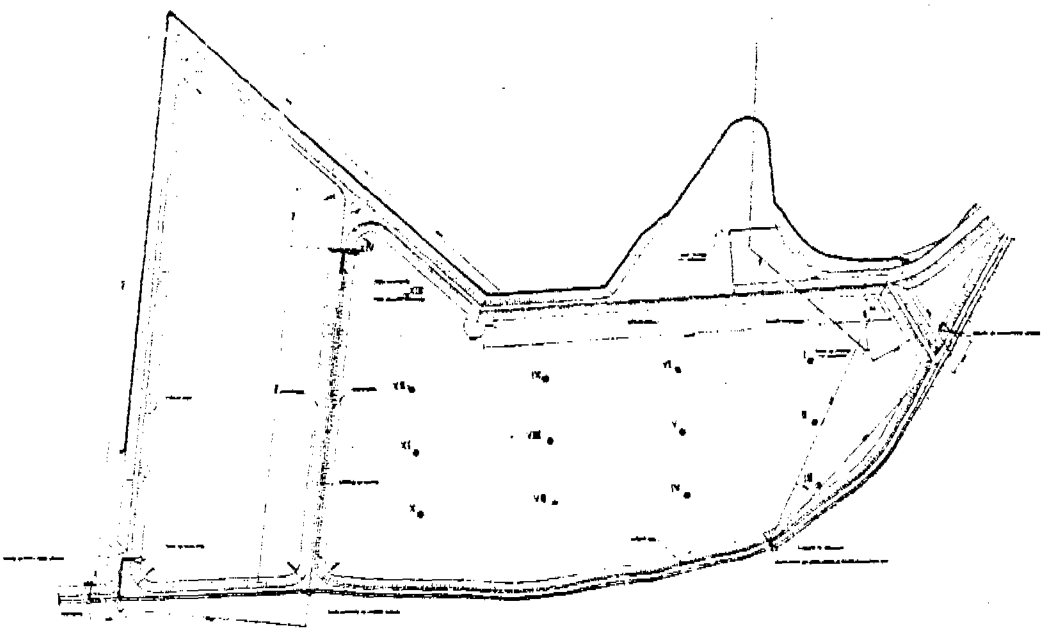

Fig. 4. General scheme of the Portimão WSP system

TABLE 2. Geometry of the Portimão WSP system

Geometry	Primary Facultative Pond	Maturation Pond
Shape	Approximately rectangular	Rectangular
Top dimensions	358.6 m x 123.3 m	312.2 m x 707.3 m
Liquid surface	353.8 m x 118.5 m	307.1 m x 102.2 m
Middle depth	350.2 m x 114.9 m	303.7 m x 98.8 m
Bottom	346.6 m x 111.3 m	300.2 m x 95.3 m
Total depth	2.0 m	2.0 m
Liquid dpeth	1.2 m	1.15 m
Inside bank slope (v:h)	1:3	1:3
Volume	48,285 m^3	34,506 m^3
Mid-depth area	4 ha	3 ha

in Vidigueira, where sampling started 24 hours after the tracer injection and proceeded daily. In the next two campaigns (Portimão/summer and Vidigueira/winter) samples were taken on the same day of tracer injection (3 hours after) and once per day thereafter. This procedure was again altered for the last campaign (Portimão/winter), when effluent samples for dye test were taken automatically every hour during the first 36 hours and then every two hours during the second 36 hours, after which samples were taken daily. After collection the effluent samples were kept in dark bottles prior to analysis in a Turner Designs model 10-005 fluorometer. The meter was zeroed with a sample of effluent taken prior to dye injection.

Physicochemical analysis

Procedures described in Standard Methods (15th ed.) were used to measure the values of the following parameters in both influent and effluent composite samples: pH, BOD_5, COD, suspended solids (volatile and non-volatile), dissolved solids (volatile and non-volatile), ammoniacal nitrogen and anionic detergents (MBAS). The composite samples were collected once per week during the first campaign (Vidigueira/summer) and twice per week in the other campaigns. Pond temperatures were obtained daily from maximum and minimum thermometers suspended at mid-depth in each pond.

Bacteriological analysis

The faecal coliform removal efficiency of the ponds was determined by collecting grab samples of the influent and the effluent once per week. Standard membrane filtration techniques were used (sodium lauryl sulphate broth at 44°C for 24 h).

Flow measurements

The influent raw sewage flow was measured daily in both WSP systems. A flow meter and recorder (Manning L-3000A) was available only during the second campaign in Portimão. During the other campaigns the liquid depth in the channel (1 m upstream of the Parshall flume) was measured and the flow determined using the standard flume flow equation.

EXPERIMENTAL RESULTS

Dispersion number and actual retention time

The values of the dispersion numbers and mean actual retention times calculated from the computerised analysis of the tracer experiments are given in Table 3, in which θ_d is the design retention time, θ the theoretical retention time (pond volume/mean daily influent flow rate), and $\bar{\theta}$ the mean actual retention time. The experimental dispersion numbers found for

TABLE 3. Results of retention times and dispersion numbers

Time Parameter (days)	Vidigueira Facultative Pond		Portimão Prim. Facultative Pond	
	Summer	Winter	Summer	Winter
θ_d	49.7*	74.6*	21.9**	76.6**
θ	78.9	27.1	13.5	12.4
$\bar{\theta}$	27.7	20.4	16.9	17.7
δ	0.523	0.574	0.371	0.595

* These values for the design retention time are not for winter and summer (no seasonal distinction was assumed by the designer), but for the initial and final (20 year) flows.

** These are the seasonal values for the first year.

the facultative pond at Vidigueira are quite similar in both the summer and winter experiments (the slightly higher values of δ in winter are probably due to greater wind-induced mixing). They indicate that the flow in the pond is strongly affected by dispersion and it is far from either plug flow or completely mixing. The long tails fo the retention time distribution curve [E(t)] shown in Figs. 5 and 6 suggest that there is a significant component of short-circuiting. This is mainly due to the wrong positioning of the inlet and outlet devices (Fig. 3) which causes most of the influent to flow rapidly to the outlet (and consequently the mean actual retention time is shorter than the theoretical detention time), while a small part of the influent stays in the pond for a long time.

Significantly different values of δ were found in the two experiments in Portimão. The higher value found in winter may be explained by the regular turnover of the pond contents, which was clearly visible to the naked eye (and which did not occur, at least to the same visible extent in summer). The transport of large sludge flocs from the pond bottom to the surface, which was as a result almost completely covered with scum, was observed for several days during the experiment. The resulting mixing was obviously much greater in winter than in summer. Such a phenomenon may also have occurred in Vidigueira (although it was not so clearly visible), but its effect (if any) would have been masked to a large extent by the very large difference in retention times that occurred there in summer and winter.

The dispersion number values found in these experiments do not agree with the values obtained from the predictive equation of Polprasert and Bhattarai [5], except in the case of Portimão pond in summer, or the equation presented by Arceivala [8] for unbaffled ponds with a width

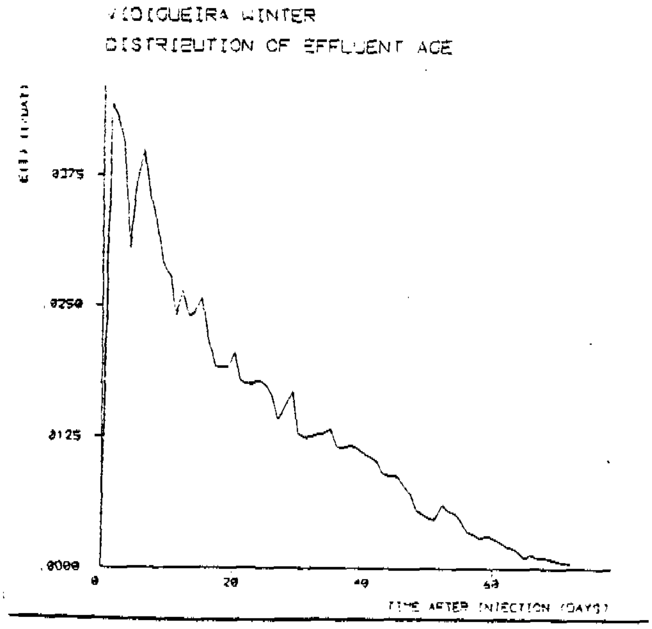

Fig. 5. Retention time distribution curve for the Vidigueira pond in winter

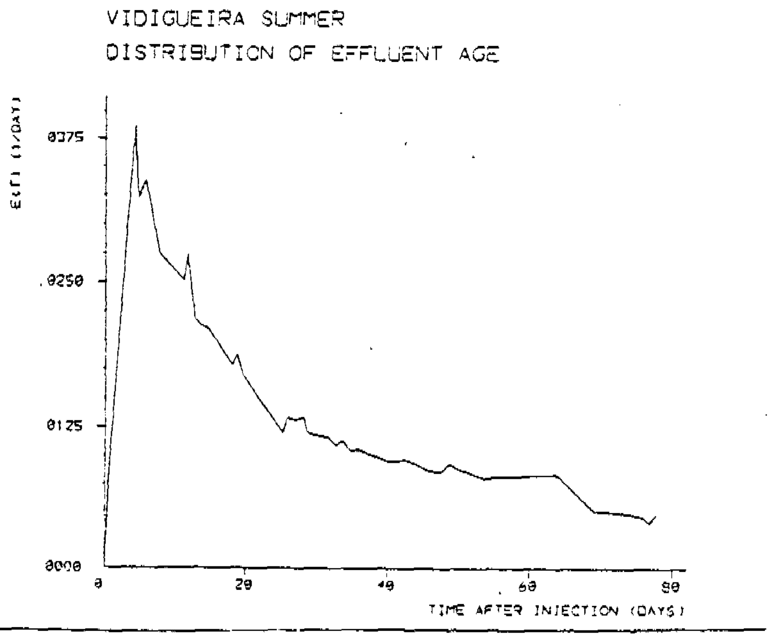

Fig. 6. Retention time distribution curve for the Vidigueira pond in summer

greater than 30 m, which both predict dispersion numbers based on the geometry of the pond, as shown in Table 4.

CONCLUSIONS

The design implications derived from the results of this study indicate that the dispersion numbers in the Vidigueira and Portimão ponds are around 0.6 (modified Levenspiel model) or less. From the Thirumurthi chart [3], this has some advantage in design in that this value of δ lies more or less in between zero (plug flow) and infinity (complete mixing). It appears therefore that facultative ponds whose geometry is similar to these ponds and which are to

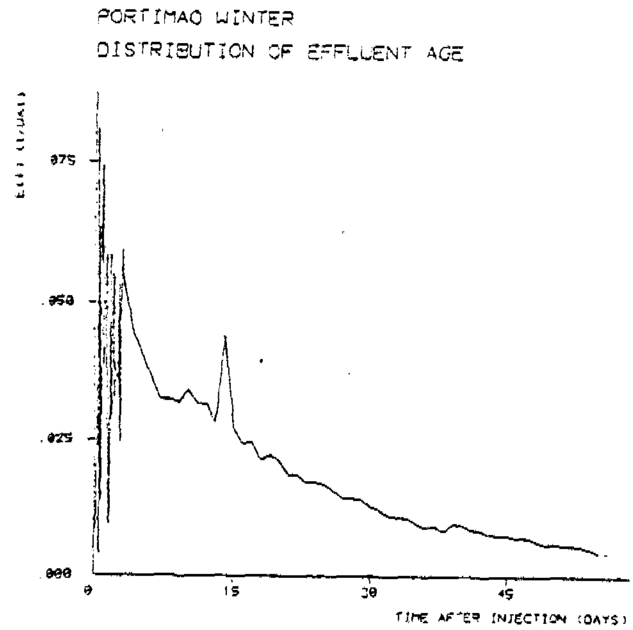

Fig. 7. Retention time distribution curves for the Portimão pond in winter

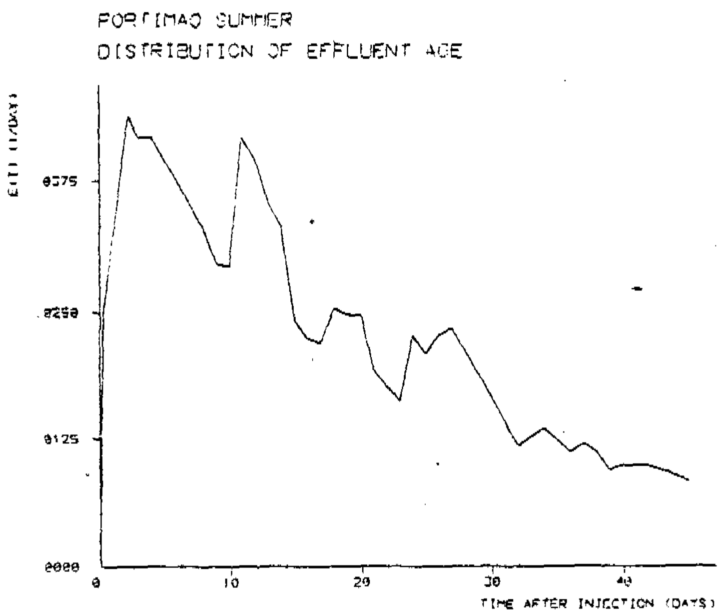

Fig. 8. Retention time distribution curves for the Portimao pond in summer

TABLE 4. Experimental and predicted values of the dispersion number

Experiment	DISPERSION NUMBER			
	Experimental		Predicted	
	Modified Levenspiel	Levenspiel	Polprasert & Bhattarai	Arceivala
Vidigueira				
Summer	0.523	0.595	2.129	87.6
Winter	0.574	0.651	1.042	29.6
Portimao				
Summer	0.371	0.316	0.332	5.0
Winter	0.595	> 10	0.358	4.6

operate under broadly similar climatic conditions would be best designed with a δ value of 0.6 in the Wehner-Wilhelm equation.

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THEME 6

HIGH-RATE ALGAL PONDS

STUDY OF THE PERFORMANCE OF A HIGH-RATE PHOTOSYNTETIC POND SYSTEM

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ABSTRACT

Colour changes and other marked disturbances were observed at a high-rate photosynthetic pond system at Alcochete, Portugal. Previous chemical and microbiological tests made it possible to attribute these occurrences to the proliferation of purple sulfur bacteria, following the probable production of sulfide inside the ponds by sulfate-reducing bacteria. Results from more recent tests and observations are presented, which confirm the earlier conclusions, in addition to revealing a number of inadequacies in the ponds chosen operating conditions, which are in all probability at the origin of the observed disturbances. Corrective actions planned include a more efficient mixing of pond contents, the strict prevention of contaminations with salty estuarine waters and the control of residence times and bottom sludge accumulation.

KEYWORDS

High-rate photosynthetic ponds; purple sulfur bacteria; sulfate-reducers; mixing; seawater contamination; sludge accumulation

INTRODUCTION

A three-pond high-rate photosynthetic system situated at Alcochete, a town in the outskirts of Lisbon, Portugal, has been designed to treat the sewage of 13 000 inhabitants. Each of the three parallel ponds has a closed-loop flow regime with a total area of 1 700 m^2 and water depth of 45 cm. Alcochete stands by the Tagus estuary and sewage is intermittently pumped from a lower point in the town to the treatment plant.

The flow movement in the ponds, with a design velocity of 3-5 cm/s (da Costa, et al., 1983), is hydraulically induced as the pre-screened and comminuted sewage enters through nozzles placed in a submerged distribution manifold. At night, recirculation pumps are operated and the treated liquid re-enters the ponds through the manifold, inducing a predicted flow velocity of 10 cm/s in these.

In normal operating conditions, ponds show an intense green colour, uniform or patched, and concentration of soluble BOD_5 at the outlet is under 40 mg/l. The level of algae concentration is then above 300 mg/l and ponds are almost entirely aerobic and odorless.

However, at several occasions over the past three years, colour changes have been observed in these ponds, often accompanied by intense odors, accumulation of algae at the surface of the liquid and invasions of insects. In an earlier study (Pinheiro and Novais, 1985), it was established that the pink colour appearing in the Alcochete ponds is due to the massive growth of pigmented sulfur bacteria of the Chromatiaceae family together with a drastic decline in the algae population. Both are the result of the appearance of large amounts of hydrogen sulfide in the ponds, for which sulfate-reducing bacteria, repeatedly identified in liquid samples taken at their outlet, are probably responsible. Adequate conditions for the proliferation of the latter groups of bacteria (anaerobiosis, organic nutrients, sulfate) were admitted to occur inside the ponds as a consequence of high organic loads, defficient aeration, particularly at their bottom, and contamination with salty, sulfate-rich estuarine waters.

It is the purpose of this paper to report on subsequent studies carried out during 1986, aiming at the positive identification of the operational defficiencies at the origin of the high-rate system's undesirable behaviour.

EXPERIMENTAL

Sampling was carried out weekly, in the morning. Grab samples were taken from the incoming sewage at a point just before it enters the distribution system, and from the effluent overflowing from ponds 1 and 2 into the discharge pipeline. Chemical analysis was done according to "Standard Methods" (1976). Chemicals used were of analytical grade, whenever possible. In all tests involving filtration steps WHATMAN GF/C glass microfibre filters were used. Estimates of hydraulic and organic loads and mixing rates were based on local measurements of pumping capacity, duration of daily pumping periods and cross-flow pond-channel areas.

RESULTS AND DISCUSSION

Periods of bright-green colouration were significantly shorter, in the whole of the three ponds, than during the year of 1985. The predominant algae genera were Euglena and Chlorella, the latter more abondant from April 1986 on, with occasional appearances of the Chlamydomonas and Oscillatoria genera. When the pond colour changed from green to pink, the algae practically disappeared from the processed effluent and several genera of the Chromatiaceae multiplied, namely Thiocystis, Thiospirillum and Chromatium.

The outburst of the pink colour in pond 2 in late March 1986 (fig. 1) followed the pattern previously described (Pinheiro and Novais, 1985). It was sudden, joined by intense sulfide odors and preceded by a short period of grey-brown colouration. In the liquid leaving the pond, sulfide and alkalinity levels rose, while chlorophyll a, suspended solids and sulfate concentrations dropped. The latter was almost immediately masked by a salinity peak, highly visible in the sewage fed to the pond on the same occasion (fig. 2). The following pink period was apparently one of less efficient sewage treatment, as shown by the COD levels in filtered, pond effluent samples, which increased from an average of 80 to around 150 mg/l. Also, as sulfide was consumed, the suspended solids concentration steadily rose, as the purple bacteria population developed.

Apart from odors and high COD levels, the observed colour changes have other inconvenients. The treated wastewater colouration itself is usually less acceptable than the green tint conferred by the algae and the elimination of pathogenics could also be slower in a predominantly anaerobic medium, in spite of the toxic action of sulfide. On the other hand, the harvesting methods and subsequent uses planned for algae might prove themselves useless on purple bacteria.

The large and brief salinity peaks detected in the sewage fed to the ponds (fig. 2) confirm that massive leakage of estuarine waters into the Alcochete wastewater network does occur during short periods of high tide. Twice, on these occasions, the algae culture responded with extensive autoflocculation. In addition to this, considerable amounts of sulfate are brought into the ponds, an effect which could start the sulfate-reducers action in the lower, anaerobic strata, with the resulting outburst of sulfide. This apparently happened in the

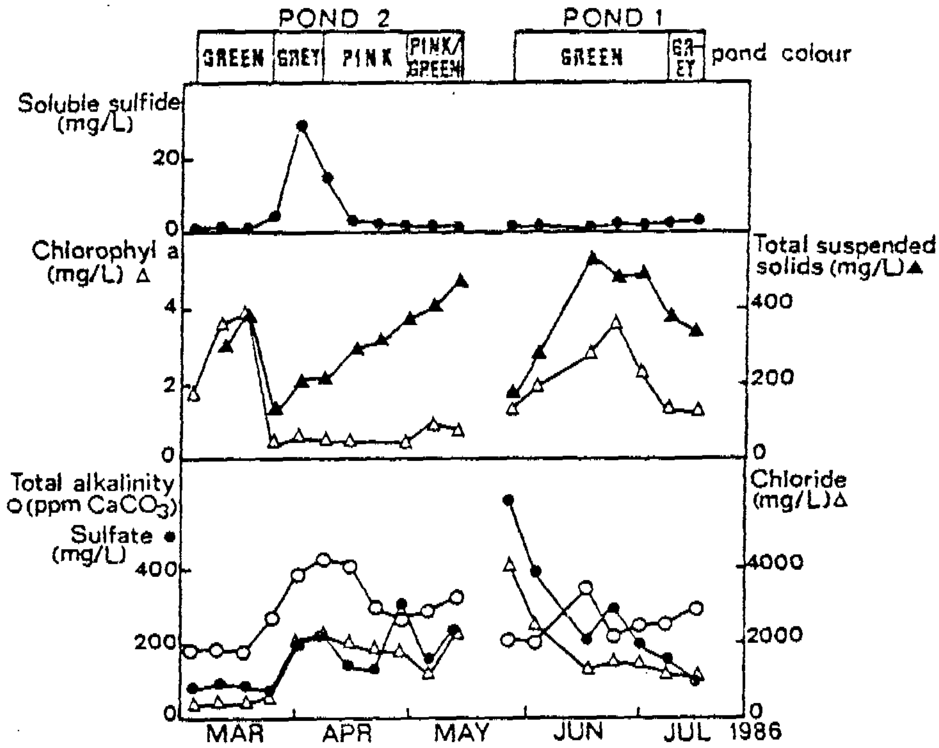

Fig. 1. Alcochete pond effluent analysis

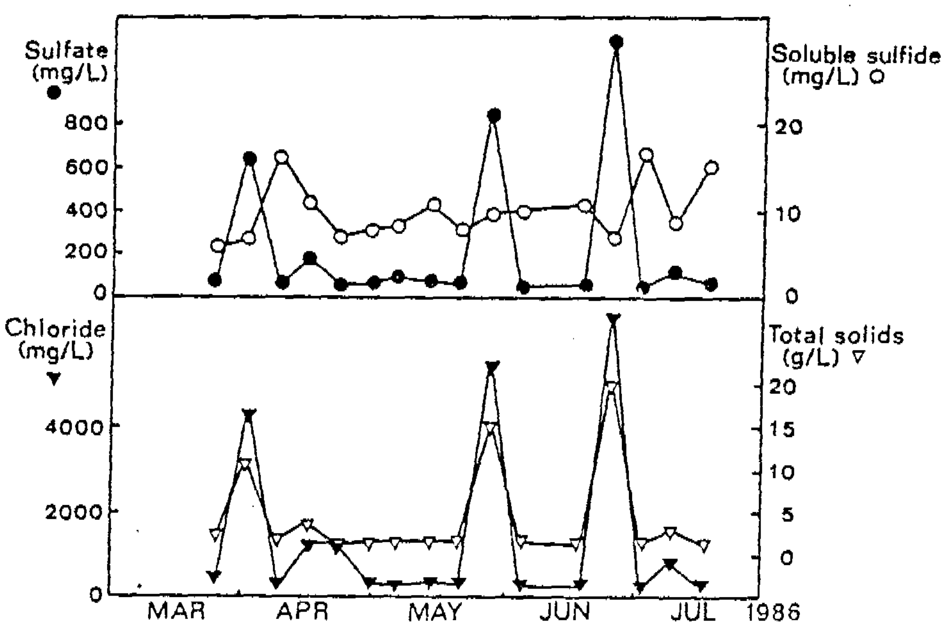

Fig. 2. Alcochete raw sewage analysis

sewage istelf since increased sulfide levels often followed salinity peaks (fig. 2).

Field measurements taken throughout the spring of 1986 revealed very low mixing levels in the ponds, as can be seen from the values given on Table 1. With the recirculation pumps at work, mixing speeds reach 0,5 cm/s, which is still largely insufficient. As a result, the continuous operation of these pumps during several weeks between February and April 1986, failed both to prevent colour transitions from green to pink and to accelerate the recovery of the algae population, after a take over by the purple sulfur bacteria.

TABLE 1 Operational Parameters of the Alcochete High-rate System (Feb-July 1986)

Parameter		Values		
		Max.	Med.	Min.
Total area (m^2)		14 000	-	4 700
Fed sewage	COD (mg/l)	1 072	677	230
	BOD_5 (mg/l)	600	346	40
	Suspended solids (mg/l)	530	331	133
Hydraulic load	(m^3/m^2.day)	0,096	0,040	0,019
Organic load	kg BOD_5/ha.day	347	148	15
	kg COD/ha.day	636	280	88
Residence time	(days)	23.7	11.3	4.7
Mixing*	speed (cm/s)	0.6	-	0.2
	duration hours/day	2.8	-	1.8

*without recirculation

Residence times nearly always far above the design value of 5 days (Table 1) are a consequence of sewage flow rates lower than the installed capacity of the Alcochete wastewater treatment plant. Given this, and until the community meets this capacity, at least one of the ponds could remain inoperative for the most part of the year, being meanwhile properly maintained or used for further studies. Besides this, the low dilution rates are apparently inconvenient because of algae culture instability. In one extreme case, during February and March 1986, the sewage feed to the ponds was interrupted for three weeks, for technical reasons. Though evaporation was negligible, the algae population grew to around 400 mg/l in pond 2 (fig. 1) with autoflocculation and colour transition to pink following shortly. Such a process may well be accelerated in Summer when sewage flow rates are minimum, temperature and insolation are high and evaporation is not negligible.

Excessive amounts of sludge were found to settle at the bottom of each pond, just ahead of the point where the sewage enters it. Thicknesses around 10 cm were regularly measured around that zone, contrasting with the 3-5 cm readings taken in other parts of the pond. As found before (Pinheiro and Novais, 1985), sand made up most of this sludge's dry weight. Still, though these highly settleable solids may not contribute greatly to the pond's organic load, they reduce the height of its aerobic zone precisely where aeration is most needed. In periods of lower photosynthetic activity, sulfate reduction may start here.

CONCLUSIONS

As a result of the observations detailed here, the need arises for a major revision of the Alcochete sewage treatment plant's operation conditions. Particular attention must be given to re-designing the mixing system and to planning the use of less pond area for sewage processing, whenever possible. The source of salt water contamination was located and eliminated in late September 1986. However, in an aging sewage disposal network such as Alcochete's other such sources might easily appear. The monitoring of salinity levels in the raw sewage is, thus, desirable. As a means of reducing sludge accumulation in the ponds, additional pre-treatments, such as a sand remover or a primary settling tank, are another possibility to be considered.

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HIGH RATE ALGAL PONDS: TREATMENT OF WASTEWATERS AND PROTEIN PRODUCTION
IV. Biomass Production From Swine Wastes

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ABSTRACT

Swine wastes from a swine breeding installations were treated in a pilot system of high rate algal ponds, and the Albazod biomass produced was harvested by autoflocculation and centrifugation. The chemical composition of Albazod biomass was determined in order to evaluate the potential of Albazod for utilization as a protein supplement to animal feed. The more important component in the Albazod biomass was crude protein and it varies according to detention time (there is an increasing of crude protein when the detention time decreases). Besides the high protein content (for 3 days of detention time we have obtained 41.46% of dry matter for crude protein), the Albazod biomass is rich in minerals, which will enhance its potential as a favourable ingredient for animal feed.

KEYWORDS

High rate algal ponds, Albazod biomass, chemical composition, swine wastes, animal feed, protein.

INTRODUCTION

Swine breeding installations are an important source of aquatic pollution in many regions (Groeneweg et al, 1980; Jones and Patni, 1972; Cheung and Wong, 1981; Fallowfield and Garret, 1985). In Portugal, they rapresent about 13.4% of the total pollution produced (Gaspar et al., 1984). The treatment of swine wastes is therefore an urgent necessity to prevent the transmission of animal and human diseases and to preserve environmental quality (Oliveira, et al., 1983).

Several processes of biological depuration are suggested to treat swine wastes according to the desirable objectives (Monteiro, 1979). Waste stabilization pond systems are normally adopted (high efficiency in the treatment, relatively very inexpensive to construct, simplicity in the exploration and maintenance, without energy consumption and reuse of the treated water for agricultural irrigation). However these systems don't take into account byproduct recovery originated in the treatment of the wastewaters (Miller et al., 1977).

With the world pollution increase and shortage in animal proteins and their increasing cost, there is a peculiar interest for using the microalgae growing in wastewaters (wastewaters-systems) as a protein source for animal feed (Behr and Soeder, 1981; Burlew, 1953; Shelef, 1982; Soeder, 1980; 1984). In addition to this application, microalgae can be used in aquaculture (De Pauw and De Leenheer, 1985; Edwards, 1980 a,b; Edwards et al., 1981; Groeneweg and Schluter, 1971), as biofertilizer in agriculture (Edwards et al., 1981), as an energy source in biogas digestors (Benemann and Hallenbeck, 1978 ; Golueke and Oswald, 1959), or as a source of chemicals (Aaronson et al., 1980).

In this point of view we began in 1984 to study the treatment of wastewater from the tomato con centrate industry in high rate algal ponds (Rodrigues and Oliveira, 1987) and in 1985 the treatment of pig wastes (Rodrigues and Oliveira, 1985).

The aim of this paper is to study the chemical composition of the Albazod (term introduced by Soeder, 1984 to designate total particulate matter in high rate algal ponds, i.e. algae(=microalgae), bacteria, zooplancton plus detritus) harvested by centrifugation and separated by autoflocculation in high rate algal ponds, as a manner to evaluate the potential of Albazod biomass for utilization as a protein supplement to animal feed.

The effluent obtained after Albazod harvested is normally clear and it can be reused for irrigation of crops on the land.

MATERIAL AND METHODS

The pilot system of high rate algal ponds was installed at the "campus" of the Faculty of Sciences and Technology, in the Department of Environmental Engineering, at Monte da Caparica. It is composed by to miniponds, each with a surface area of about 2.6 m^2 and an average depth of 15 cm (Rodrigues and Oliveira, 1984). They are shallow receways (Soeder, 1981) and the suspension is permanently stirred by means of paddlewheels.

The pig waste used in this study was collected in MAJOPOR, a swine breeding located in Santo Estevão das Galés, near Malveira (about 50 km from Monte da Caparica), where the solids of greater dimensions were separated by a sieve with a mesh width of 2.36 mm. This procedure eliminated mainly the slowly degradable or non degradable matter. In the campus the liquid phase of the effluent was diluted with tap water (dilution 1:10) and it was pumped into the miniponds between 6.00 a.m. and 8.00 p.m.

In the way to determine the treatment efficiency of the pig waste and the chemical composition of Albazod, in the miniponds, two sets of experiments were performed from July to August, 1985. In the first set (July) theoretical detention times in each pond of 5.0 and 7.0 days were used, while in the second set (August) theoretical detention times of 3.0 and 6.0 days were used.

The Albazod was harvested by centrifugation of one liter of the effluent, during 10 to 15 minutes at 3000 - 4000 g. The effluent obtained after centrifugation was analysed (Rodrigues and Oliveira, 1985) and the Albazod was washed two times with distilled water, to separate the effluent from the Albazod. In July, we left settling two liters of effluent in decantation ampoules to compare the chemical composition of the Albazod harvested by centrifugation and autoflocculation.

The concentrated Albazod was dried in a vacuum oven at 80ºC and 1 atm during 8 hours. After drying, each sample was ground and homogenised. The powder samples were maintained in the dark. In these samples several analyses were made to determine the chemical composition of the Albazod. We have used the following procedures (Oliveira, 1971; Oliveira et al., 1983):

- Moisture, by drying at 103 ± 2ºC in an electric oven up to constant weight;
- Total nitrogen, by the Kjeldahl method;
- Proteinaceous nitrogen, by precipitation with trichloroacetic acid followed by nitrogen determination by the Kjeldahl method;
- Proteins (Crude and Pure), using the factor 6.25 to convert the nitrogen value to protein;
- Fats, by extraction using petroleum ether in a Soxhlet apparatus;
- Cellulose, by the method of Belluci;
- Carbohydrates, calculated by difference between the dry matter versus the crude protein, fat and fixed residue together;
- Nitrogen-free extract, calculated by difference between total carbohydrate and fiber;
- Food energy, calculated from the values of crude protein, fat and carbohydrate, according to the Atwater method;
- Fixed residue, by calcination at 525±25ºC for two hours; and
- Total phosphorus, by the vanado-molybdane method.

RESULTS

The results of the determination of the chemical composition of Albazod biomass are present in Tables 1 to 3. In Fig.1 we compare the results obtained in July (5 and 7 days of detention time) harvested by autoflocculation and centrifugation. In Fig.2 we can see the distribution of chemical composition of Albazod harvested by centrifugation with 3, 5, 6 and 7 days of detention time.

TABLE 1. Chemical Composition of Albazod (% dry matter) Harvested in HRAP with 5 days of Detention Time

Harvesting Fraction	Autoflocculation Average[1]	Autoflocculation Range	Centrifugation Average[1]	Centrifugation Range
Moisture	7.42	5.95-10.05	6.62	6.25- 7.30
Total nitrogen	5.20	4.69- 5.67	6.08	5.76- 6.39
Crude protein	32.48	29.31-35.44	37.97	36.00-39.94
Proteinaceous nitrogen	4.31	4.05- 4.61	4.61	4.37- 5.05
Pure protein	26.92	25.31-28.81	28.83	27.31-31.56
Non protein nitrogen	5.56	0.50- 8.81	10.50	8.69-12.31
Fats	4.75	1.89- 8.32	6.71	5.25- 8.16
Carbohydrates	30.59	26.94-32.26	27.85	26.77-28.93
Fiber	5.45	3.23- 8.43	7.74	6.84- 8.64
N-free extracts	25.14	19.03-29.03	20.11	19.93-20.29
Ash	32.18	30.41-34.08	26.43	24.33-29.07
Phosphorus	3.27	3.14- 3.38	3.11	3.00- 3.19
Food energy (cal/100g)	275	268 - 282	293	288 - 297
N/P	1.59	1.39 -1.81	1.92	1.81 -2.03

(1) Mean of three values

TABLE2. Chemical Composition of Albazod (%dry matter)Harvested in HRAP with 7 days of Detention Time

Harvesting Fraction	Autoflocculation Average[1]	Autoflocculation Range	Centrifugation Average[2]	Centrifugation Range
Moisture	9.36	6.65-11.10	6.90	6.10- 8.25
Total nitrogen	4.84	4.68- 5.09	5.61	5.06- 5.91
Crude protein	30.22	29.25-31.81	35.04	31.63-36.94
Proteinaceous nitrogen	4.28	3.99- 4.52	4.79	4.37- 5.00
Pure protein	26.77	24.94-28.25	29.94	27.31-31.25
Non protein nitrogen	3.45	2.56- 4.31	5.10	1.19- 7.44
Fats	2.31	0.75- 4.02	3.73	0.93- 6.60
Carbohydrates	32.00	29.51-34.06	31.21	29.12-32.85
Fiber	8.76	7.50-11.30	8.16	4.92-10.84
N-free extracts	23.24	19.99-26.39	23.06	20.41-27.93
Ash	35.48	33.54-36.90	28.12	25.32-31.10
Phosphorus	3.15	2.87- 3.36	3.23	2.80- 3.65
Food energy (cal/100g)	235	222 - 245	269	251 - 289
N/P	1.54	1.41 -1.77	1.75	1.43 -2.08

(1) Mean of four values
(2) Mean of five values

DISCUSSION

The results obtained for the chemical analysis of Albazod vary to some extent as a consequence for instance of the detention time. There is also variation for each detention time as a consequence of climatic and physico-chemical conditions of the high rate algal ponds units.

An important fraction of Albazod biomass consists of crude protein, whose content increases when the detention time decreases. We have obtained values of 35.04% of dry matter for 7 days of detention time and 41.46% of dry matter for 3 days. Fats, represent the minor fraction, among the major constituents of Albazod biomass.

Carbohydrates present high values. They were determined by difference between the other constituents and the total biomass, including manomeric glucides and cellulose and other complex substances.

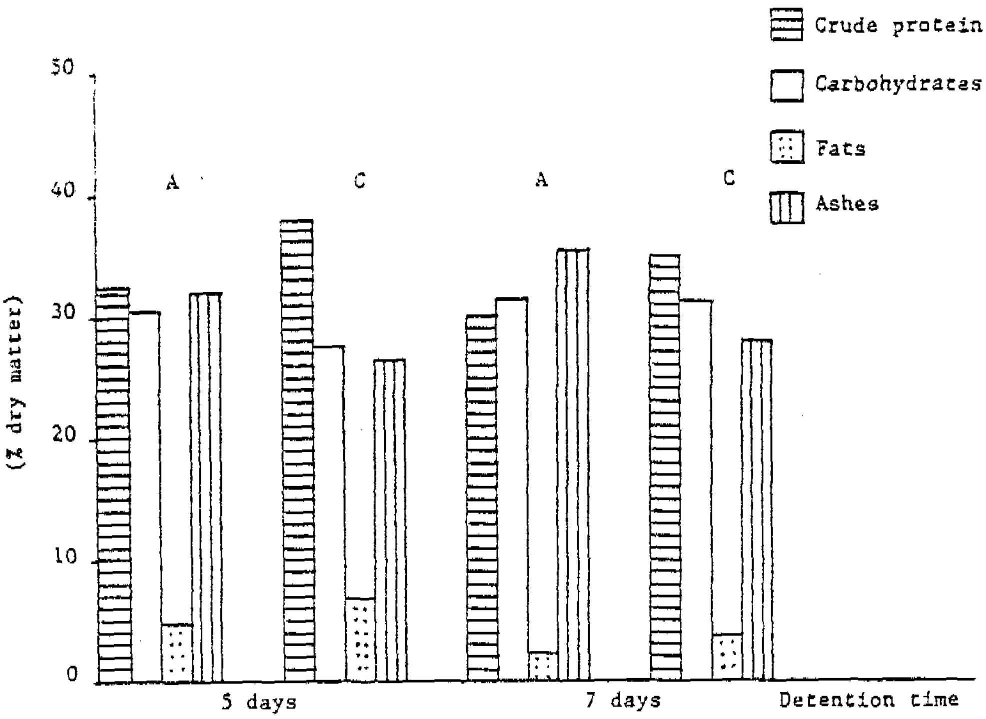

Fig. 1. Mean values of the major components of Albazod (% dry matter) harvested by autoflocculation (A) and centrifugation (C) in HRAP.

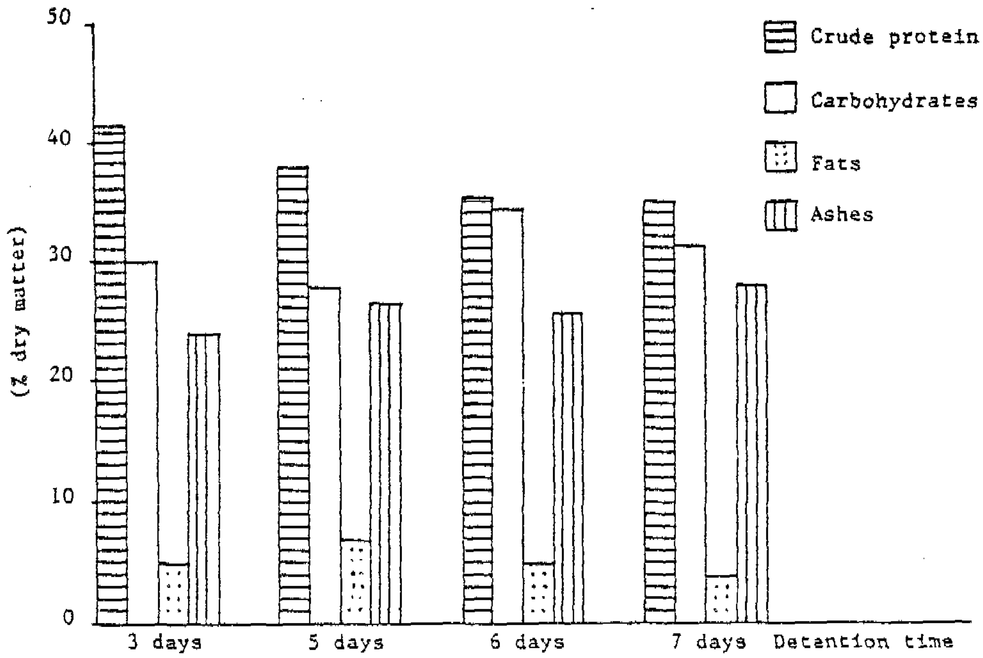

Fig. 2. Mean values of the major components of Albazod (% dry matter) harvested by centrifugation in HRAP.

TABLE 3. Chemical Composition of Albazod (% dry matter) Harvested by Centrifugation in HRAP with 3 and 6 days of Detention Time

Detention Time	3		6	
Fraction	Average[1]	Range	Average[2]	Range
Moisture	6.48	4.20- 9.70	4.60	3.25- 5.10
Total nitrogen	6.63	5.87- 7.80	5.67	5.48- 5.83
Crude protein	41.46	36.69-48.75	35.41	34.25-36.44
Proteinaceous nitrogen	4.69	4.16- 5.32	4.86	4.54- 5.13
Pure protein	29.33	26.00-33.25	30.35	28.38-32.06
Non protein nitrogen	12.14	8.50-15.50	5.07	3.69- 7.32
Fats	4.93	3.08-10.58	4.73	1.10-10.95
Carbohydrates	29.80	24.55-33.11	34.27	28.02-37.62
Fiber	8.07	6.56- 9.24	8.30	4.44-10.28
N-free extracts	21.73	15.78-24.04	25.97	17.74-33.18
Ash	23.81	17.94-27.12	25.60	24.39-26.78
Phosphorus	2.53	2.09- 2.84	2.76	2.56- 2.88
Food energy	297	271 - 328	288	270 - 307
N/P	2.69	2.07- 3.55	2.05	1.90- 2-20

(1) Mean of 5 values
(2) Mean of 4 values

The values obtained for fiber are high probably as a consequence of the presence of green algae (specially Monoraphidium, Scenedesmus and Chlorella) that predominate in Albazod biomass. Green algae have rigid cell walls, composed of polysaccharides, particularly cellulose and hemicellulose, which limit the utilization of the Albazod biomass as a protein source for non-ruminant animals, by the decrease of digestibility they induce.

Albazod biomass harvested by centrifugation presents a higher nutritive value than the Albazod harvested by autoflocculation. Minerals are an important fraction in Albazod biomass harvested by autoflocculation, also as a consequence of some nutrients precipitation, at high pH values, resulting from an active photosynthesis.

CONCLUSIONS

Swine breeding units are significant sources of pollution in the aquatic environment. Swine wastes represent a considerable polluting load, which can be treated in high rate algal ponds with a high efficiency and low detention times (Rodrigues and Oliveira, 1985). Albazod biomass harvested by centrifugation or autoflocculation in high rate algal ponds, has significant content of proteins and minerals. So, it can be used as an ingredient for animal feed, due to its high nutritional value. The utilization of high rate algal ponds in the treatment of swine wastes is potentially, a effective reducing factor of treatment cost, as Albazod biomass produced can be recycled for pig rations and treated water can be used in irrigation or even in aquaculture.

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HIGH RATE OXIDATION PONDS - THE ISRAELI EXPERIENCE

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ABSTRACT

The use of high rate oxidation ponds as means for wastewater reclamation and algal biomass production was extensively studied in Israel. The work started from laboratory scale facilities through outdoor miniponds, pilot plant ponds, field scale ponds and a full scale pond. The research involved optimal operational design of high rate oxidation ponds for various locations and constraints, harvesting of the algal biomass and feeding and toxicological studies of the biomass produced. We have found that the system is reliable and efficient as biological wastewater treatment system and could be very valuable if operated according to environmental and climatic constraints of the given area it is designed for.

KEYWORDS

High rate oxidation ponds, scaling up, facilities, productivity, operation, design

INTRODUCTION

The pioneering work of William J. Oswald on High Rate Oxidation Pond (HROP) at the University of California during the 60's has been extensively continued in Israel where the climatic conditions, the need for both reclaimed water and recovered proteins, the high cost of energy and the restrictions in available land (such as needed for ordinary stabilization ponds) all directed to the utilization of this concept. Harvesting and processing of the algal-bacterial biomass to produce valuable proteinaceous animal feed in one hand and clear effluent of quality high enough to allow irrigation of unrestricted crops (including edible vegetables) are discussed in other papers because of the limited scope of this one. Both the technology and economics of the HROP method were found most encouraging and it is envisioned that when investments in sewage treatment works in Israel will be resumed full scale HROP plants will be constructed in small and intermediate size communities.

DESCRIPTION OF FACILITIES AND RESEARCH CONDUCTED

The work in Israel which had started in 1968 at the Hebrew University of Jerusalem (Shelef et al. 1973), continuing at the Haifa Technion from 1970 to 1981 (Shelef et al. 1974-1981), constitutes one of the most comprehensive studies of HROP. It has involved extensive laboratory work and outdoor studies with continuous, year round operation of ponds of various sizes:
A. Outdoor miniponds (fig. 1), and eight "bath" ponds ($0.34 m^2$ and $0.92\ m^2$ respectively), were used to study basic characteristics of the system such as: 1. Effects of temperature and solar radiation on HROP performance. 2. Nutrients limiting algal production. 3. Organic load at different climatic conditions. 4. Optimal mixing rate. 5. Biological population composition of HROP under different conditions. 6. Effect of aeration on HROP performance.

B. Three pilot plant HROP were constructed: The Jerusalem Pilot Plant pond (110 m^2, fig. 2) and two Technion pilot plant ponds (120 m^2 and 150 m^2, fig. 3). The pilot plant ponds were used to verify the conclusions of the miniponds experiments and to study continuous year round operation of HROP. The pilot plant ponds were operated in order to evaluate optimal operational conditions of HROP in terms of retention time, depth and organic load.
C. Two field scale ponds in Haifa Bay (1000 m^2 each, fig. 4) in which the conclusions of the pilot plant operational strategies were put into practice. The field scale ponds were also used for harvesting large quantities of algal biomass used for different feeding experiments (see below).
D. A full scale HROP (10 hectares) in Eilat (Fig. 5).

Fig. 1: Outdoor miniponds at the Technion (0.34 m^2)

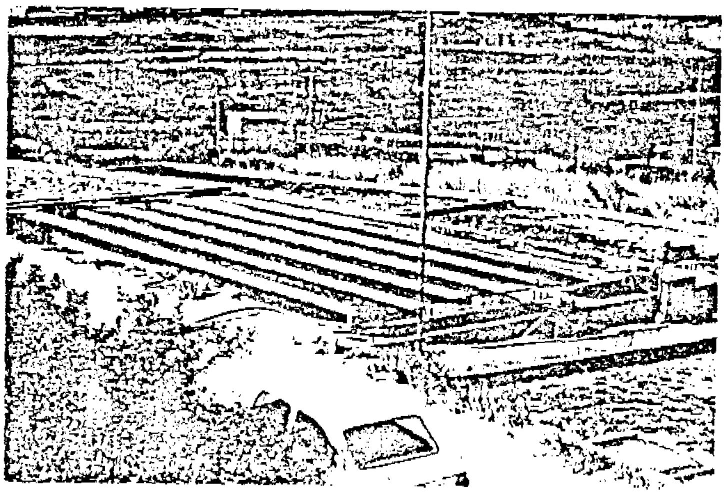


Fig. 2: Pilot plant HROP in Jerusalem (110 m^2)

Fig. 3: Pilot plant HROP at the Technion (120 m^2)

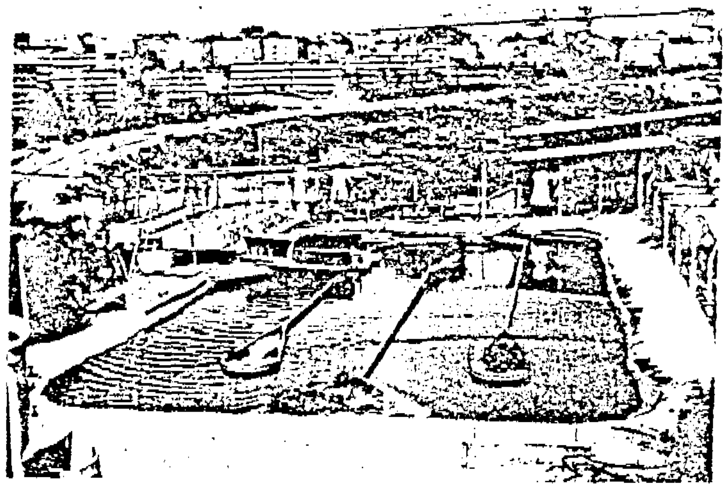


Fig. 4: Field scale HROP in Haifa Bay (1000m^2)

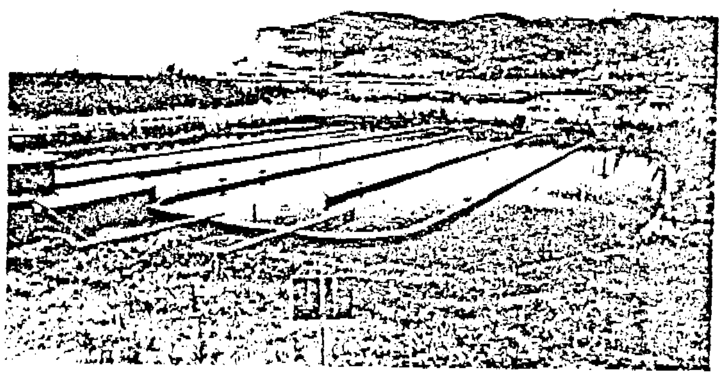
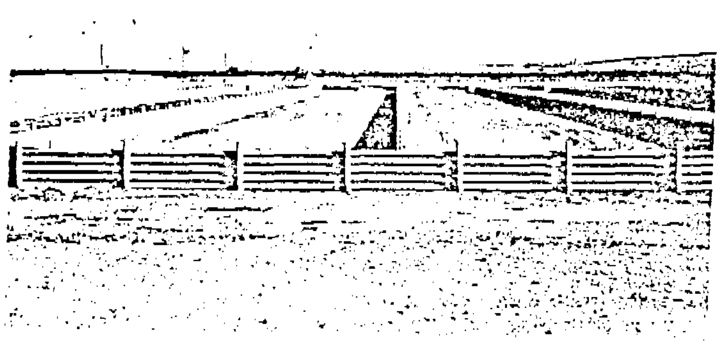


Fig. 5: Full scale HROP under construction in Eilat (10 hectares)



Detail description of facilities, research conducted, results and conclusions were published in the following publications: Shelef et al. 1973, Shelef et al. 1974-1981, Shelef et al. 1980a, Azov et al. 1980, Azov & Shelef, 1982, Azov et al. 1982, Sukenik & Shelef, 1984.

The Technion comprehensive study on Combined Algal Wastewater Treatment and Reclamation - Protein Production involved also algal biomass harvesting using various sizes of equipment units, including: five flotation units, three centrifuges and two drum dryers as well as microstraining units, cross-flow filtration unit and other physical separation units. Extensive animal feeding experiments and nutritional studies has been carried out by Hepher and Sandbank with fish (Hepher et al., 1978, Sandbank & Hepher, 1978, 1980), by Mokady et al. (1978, 1980) with broiler chicken, by Hurwitz and Lipstein (1980) with laying hends and by Finkel et al. (1979) with Macrobrachium. The feeding and nutritional studies were preceded and followed by extensive toxicological studies carried out by Yannai et al. (1978, 1980). The Technion studies have been expanded to include the use of HROP for animal wastes treatment (Shelef, 1979) of dairy wastes (Shelef & Sandbank, 1979) and of nitrogenous industrial wastes (Shelef et al., 1980b).

The HROP Dilemma

High rate oxidation pond is a unique and complicated system as its economics rest upon achieving two goals: secondary wastewater treatment and algal biomass production, both at minimal investment and operational costs. Actually, HROP is a combination of intensified oxidation ponds with algal reactor. Theoretically, it seems that both objectives are complementary because algae supply the oxygen demand for the aerobic degradation of the wastewater borne organic matter, so that maximizing algal production should have resulted in maximizing HROP performance as a wastewater treatment system. However, if one intends to design a HROP that will have equal performance year round, the system should be ruled out in any location, other than the tropics with uniform climatic and solar conditions over the whole year. Otherwise, seasonal changes in water temperature and solar radiation have always significant effects on HROP performances. Considering seasonal variations of these parameters in Haifa (fig. 6) it was shown that expected algal productivity and optimal retention time will change considerably with the season (Azov and Shelef, 1982). Fig. 7 summarizes theoretical considerations of seasonal effects on algal productivity and optimal retention time in Haifa. The upper curve represents mid-summer conditions of high solar radiation and high water temperature which lead to high algal productivity at short retention time. The middle curve represents fall and spring conditions while the lower curve represents mid winter conditions of low solar radiation and temperatures which result in much lower algal productivity at longer retention time.

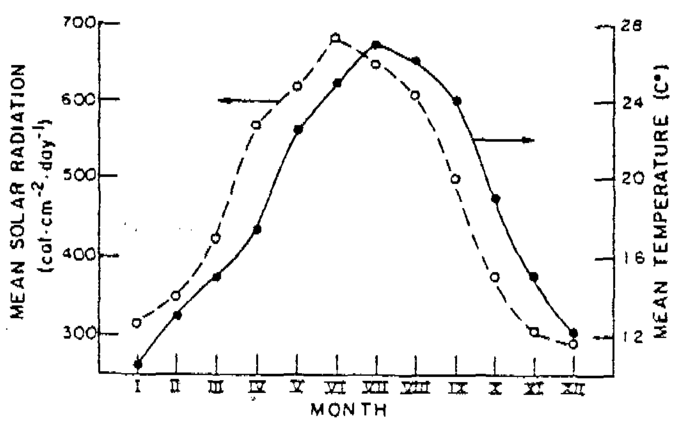
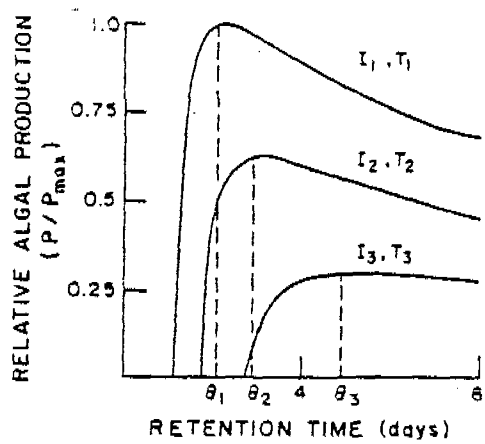


Fig. 6: Seasonal variations in solar radiation and temperature at Haifa (Lat. 32^{O}N)

Fig. 7: Algal production curves at various light intensities (I), and temperatures (T).

I_1, I_2, I_3=0.9,0.5,0.3 cal $cm^{-2} min^{-1}$

T_1, T_2, T_3=28,19,10°C respectively.



In order to put these constraints into practice, the HROP should be operated at short retention times during summer and long retention times during winter. Because sewage flow rate into HROP is practically uniform year round, the only way to obtain the desirable retention time is by varying the dimensions of the pond, i.e. depth and surface area. Theoretically for constant year round sewage inflow, best HROP performance is expected if the pond is shallow and large during winter operation and vice-versa during summer operation. However, this approach is not practical as the large area needed for winter operation will rule out the economics of the system.

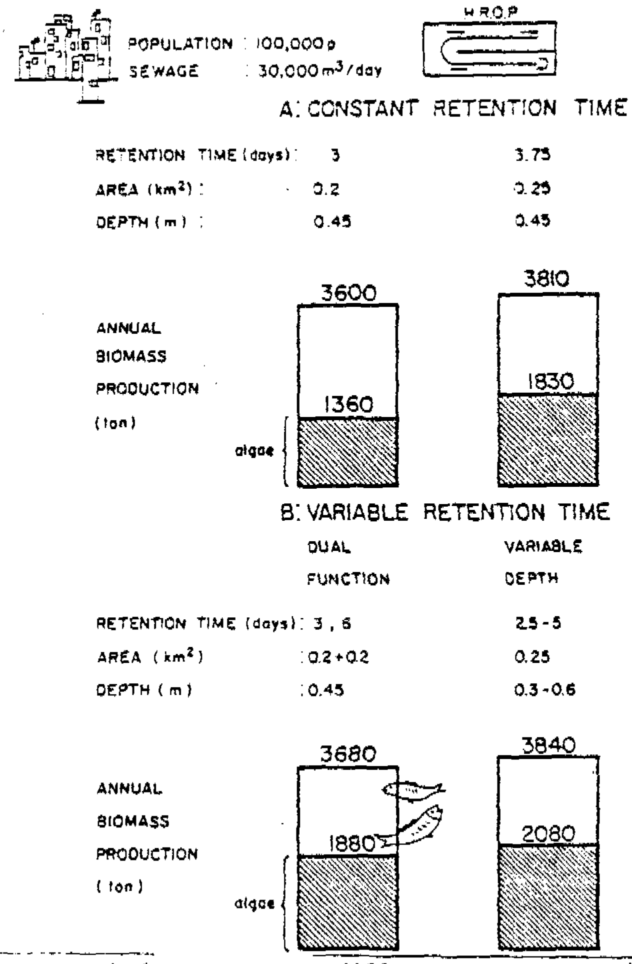
During our long and extensive HROP research in Israel we have designed and analyzed different approaches to HROPoperation that will cope with the above constraints in economically sound ways. The most plausible HROP operation strategies are summarized in table 1, and figure 8.

Table 1: Year round operation of HROP at various operational modes

Season	Winter				Spring				Summer				Fall			
mode*	I	II	III	IV	I	II	III	IV	I	II	III	IV	I	II	III	IV
Retention time (days)	3	4	8.7	4.7	3	4	4.4	3.2	3	4	2.2	2.3	3	4	3.8	3.4
Depth(cm)	45	45	45	52	45	45	45	40	45	45	45	28	45	45	45	38
TSS (mg/l)	290	240	235	235	380	370	315	400	325	350	385	415	320	380	400	360
Algae (mg/l)	32	95	110	110	210	155	185	210	140	175	165	225	150	170	190	220
Net primary productivity gr/m^2/day	4.8	10.7	5.7	12.2	31.5	17.4	18.9	26.3	21.0	19.7	33.7	27.4	22.5	19.1	22.5	24.6
Algae in TSS (%)	11	40	47	47	55	42	59	53	43	50	43	54	47	45	47	61
Protein in TSS (%)	46	52	51	61	47	48	55	51	46	49	52	58	53	51	56	56
BOD removal (%)	86	92	96	94	93	94	94	94	94	95	93	94	93	94	95	94

* I - Constant retention time of 3 days
II- Constant retention time of 4 days
III- Seasonal variations of retention time by varying pond's area at constant depth
IV - Seasonal variations of retention time by varying pond's depth at constant area

Fig. 3: Estimated dimensions and productivity of HROP serving a community of 100,000 inhabitants (see text for details)



The different operational modes presented above may serve different purposes at different HROP locations. Generally, a constant retention time is most adequate for tropical climates where seasonal variations in water temperature are minimal. As seasonal variations in solar radiation and temperature become more significant, the attractiveness of a variable retention time strategy increases. As for such zones, a constant 3 day retention time produces the least biomass with relatively low algal percentage in it (about 38%), because of the virtually lack of algal production during winter months which will cause their washout at 3 days retention time. Moreover, it will require additional energy input during the severest winter months to supply oxygen for complete aerobic degradation of the organic wastes. However, this mode has the least land requirements and should be preferred in locations where land cost over energy cost is large enough to compensate for the additional energy input required during winter operation. A slightly longer retention time (3.75 days), will require additional area but may avoid the need for additional energy input. The third alternative presented is a dual function HROP. According to this method the year is divided into two major seasons, cold and warm, where retention time in the cold season is doubled over that of the warm season (say 6 days instead of 3). This is accomplished by operating two ponds in parallel during the cold season for every pond operating during the warm season. During the warm season the idled ponds are filled with clarified effluents of the operating ponds and used for edible fish culture. This strategy has double the land requirement of the constant 3-day retention time strategy but will operate aerobically year-round with higher crops of total biomass and algae. This approach could be of special interest in agricultural locations. Fish production during the warm season can amount to 100 tons for the given community of 100,000 inhabitants.

In most locations the variable depth strategy should be preferred for optimal year round HROP operation. This operational mode is based on a pond having constant area year round. Seasonal required changes in retention time are produced by varying pond depth using a variable level overflow weir. This strategy is relatively easy to operate and responds better than any other operational modes for seasonal variations in light intensity and temperature. Total biomass production, algal production and protein percentage in total biomass were the highest for this strategy compared to all others studied by us.

CONCLUSIONS

The high rate oxidation pond may serve as an efficient wastewater treatment system combined with the production of valuable by products. According to environmental and economical constraints it is possible to design a HROP that will achieve both goals in a reliable manner. The algal-bacterial biomass produced in HROP may be used for numerous purposes such as protein rich animal feed, raw material for the extraction of natural pigments, fatty acids, vitamins etc. natural food for fish larvae, soil fertilizers etc. An adequate operational strategy is essential for the economic operation of HROP at different locations. It is recommended that seasonal variations in solar radiation and temperature will be taken into account by changing the retention time in the pond. This may be done by either operating deeper ponds (at Longer retention times) during the cold season or by using several ponds during the cold season and using the idled ponds in summer for commercial fish raising.

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HARVESTING OF ALGAE FROM HIGH-RATE PONDS

BY FLOCCULATION-FLOTATION

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ABSTRACT

Microalgae grown in stabilization ponds and in high rate oxidation ponds (HROP) treating wastewater, were harvested by flocculation with aluminium sulfate and floated in a laboratory electro-flotation unit, where bubbles were produced electrolytically. It was shown that the higher the solids concentration, the lower is the air: solids ratio needed to achieve 90% algae removal. The simultaneous flocculation of suspended algae and release of bubbles while flocs grow, give equal suspended solids removal as the classical flush mixing flocculation followed by slow mixing-flotation stage. Air: solid ratios between 0.009 and 0.013 were sufficient for flocs flotation by dissolved air flotation (DAF) and electro flotation. Continuous flocculation-flotation in a DAF pilot plant unit showed that flush mixing can be replaced by injection of chemicals in-line provided a retention of 15 sec is given before the inflow to the flotation tank. Slow mixing could be eliminated altogether when bubbles were generated simultaneously with the flocculation stage. DAF was further simplified, replacing the air compressor used for pressuring air into water, by a venturi suction at the undespressive side of the pump in the pressurised line. DAF in its simplified version is preferred for algae harvesting in fresh water algae cultures and in HROP systems, while it is recognized that electroflotation may be preferred in harvesting marine microalgae.

KEYWORDS

Harvesting of microalgae; Algae removal; Flocculation; Flotation; Dissolved Air Flotation; Electroflotation.

INTRODUCTION

Ideally, methods for microalgae harvesting from stabilization ponds and High Rate Oxidation Ponds (HROP) should match the simplicity and low energy demands of pond systems. However, the techno-economic problem of harvesting micro-algae lies in the nature of pond effluents with dilute suspensions with 0.015% to 0.05% algal-bacterial solids of microscopic size (5-25μ). In order to discharge pond effluents into receiving water bodies, microalgae have to be removed to avoid an increase in the oxygen demand and nutrients loading during algal biomass decomposition. On the other hand, recovery of the biomass can be of a significant economic value. Algae removal can be done by many solid-liquid separation processes such as microscreens (Kromanik et al., 1979), paper filtration (Dodd, 1977), sedimentation in "isolation ponds" (Benemann it al 1980), centrifugation and a variety of other methods described by Mohn, (1978), but since the first comprehensive work on algal harvesting (Golveko et al., 1965), the most reliable and cost effective methods are based on flocculation followed by sedimentation or flotation, especially in the field of wastewater treatment.

If microalgae are to be used as a potential source of animal nutrients (Oswald et al., 1968) (Shelef et al., 1977) as raw material for the extraction of chemicals (Aaronson et al., 1980), algae removal can be considered as a "harvesting process". Already in the fifties, flocculation-flotation was proposed for harvesting algae (Cook, 1951). Work carried out by Van Vuuren et al., (1965) indicated that algae could be succesfully separated by flocculation-flotation.

These processes were adopted by Wachs et al., (1972), after observing that flocculated algae have a natural tendency to float, and flocculation-flotation was adopted thereafter for production of microalgae for feeding experiments (Shelef et al., 1976). Flocculation-flotation has also been used to harvest microalgae from animal wastes treated in HROP (Lincoln et al., 1977) and from cultures of marine microalgae (Sukenik et al., 1984). Although the basic principles used in the present work are still the same, the main purpose of this work is to demonstrate the evolution of the process and the modifications recently introduced to simplify the flocculation-DAF process and equipment thus improving its economics and enhance the use of wastewater borne microalgae harvesting.

MATERIALS AND METHODS

Flocculation is a prerequisite for microalgae flotation and in order to study both processes together, flotation was performed in a laboratory electroflotation jartest unit.
This unit was assembled using a Phipps and Pard, multiple-stirrer. Each of the six jars was provided with a pair of 4 cm^2 electrodes which were connected in series to a rectifier (2A-24V), and perspex paddles replaced the original metal paddles. Flocculation was performed on HROP effuent with alum, stirring at 80 RPM during 1 minute and at 20 RPM for 10 additional minutes, followed by either 10 minutes settling or 10 minutes of bubbles degagement by electrolysis.

Algal removal effociency was determined by the % light transmittance at 420 nm.
Experiments were performed to determine the air: solids (A/S) ratio needed to float algae flocs at different algal concentrations, and the effect of performing flocculation while bubbles are simultaneously degaged during floc formation.

Electroflotation was also performed using a 2 m^3 unit, with stainless steel titanium dioxide electrodes covering the bottom of the unit at various electrical charges varying from 6 to 36 coulombs l^{-1}. Dissolved air flotation was performed on a 1000 l unit which was built originally following DAF principles (Vrablik,- 1959), and design data by bare et al., (1975), to which modifications were done to simplify the equipments and define operational conditions. In one experiment the recirculated pressurized flow was kept constant at 8 l per minute at 3 atm pressure while the in flow rate was varied. This results in different A:S ratios and different loads of solids. Results were measured by TSS determinations.

RESULTS AND DISCUSSION

Aluminium sulfate was used throughout these experiments, because the algae harvested with alum have a lower inorganic content as compared to algae harvested with $Ca(OH)_2$ and alum in Israel is cheaper than $FeCl_3$. More recently polyacrilamide based flocculants have been succesfully used for algae herevesting, (Viviers et al., 1981, Sandbank et al., 1987) and Chitosan, a derivative of chitin shows promise because it is not toxic (Venkataraman et al., 1980). As shown in Figure 1, alum flocculation has a marked pH dependence with an optimal pH range of 5.5-5.8. This agrees with results of McGarry (1970), Al-Layla et al., (1975) and Moraine et al., (1980). Polyelectrolytes and chitosan are less pH dependant and usualy are effective at the typical HROP effluent pH ranges of 7.2 to 8.5. Since they are efficient flocculants and dewaterstability of the float is better than with alum, polyelectrolytes have a wide scope of application provided they compete price-wise and they do not leave toxic residues in the flocs if the algal biomass is intended for animal feed.

In this respect, it should be noted that an approval of a flocculant as a non-toxic for water treatment puroposes is not sufficent when algal biomass has to be used as feed, since toxic residues may not remain in the filtered water but still accumulate in the harvestable flocs.

The effect of two different pond effluents on alum demand is shown in Figure 2 and stresses the importance of performing jar-tests experiments for every new type of effluent to be treated.

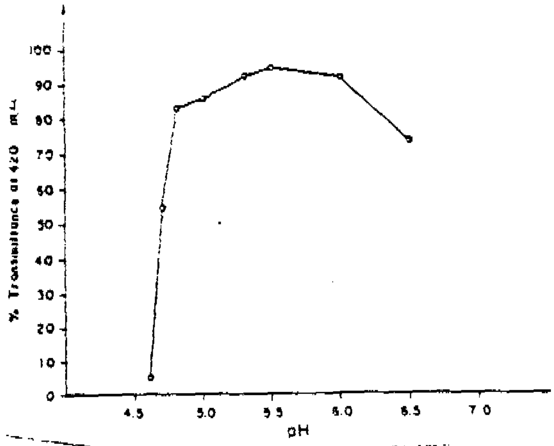

Fig. 1. Effect of pH on the flocculation of algae by aluminium sulfate

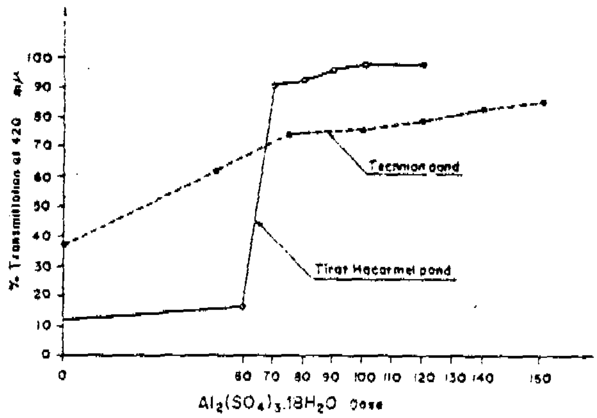

Fig. 2. Determination of the optimal dose of aluminium sulfate

Results of the electroflotation jar-tests indicate (Fig. 3), that the higher the algal concentrations, (say 270 mg/l and 360 mg/l, the lower air: solids (A:S) ratio (0.008) is needed to achieve 90% removal, as compared to A:S of 0.02 in suspensions with 90 and 180 mg/l. This should be related to the higher probability of particles collision in the suspensions with high concentrations of algae.

As shown in Figure 4, rapid mixing followed by the simultaneous release of bubbles into the flocculation tank during flocculation, achieves the same removal efficiency as that obtained by rapid or flash mixing, slow mixing during 10 min. and subsequent release of bubbles.

The entrappment of bubbles inside the flocs during floc formation leads to a marked increase in the upflow velocity of flocs as compared to the velocity of flocs with bubbles absorbed on their surface. (Vrablik, 1959). This same effect can be obtained when entrapping bubbles of supersaturated O_2 produced photosynthetically by algae within the pond. The practical meaning of this experiment is that rapid mixing is sufficient before flotation and that there is no need for a separate slow mixing chamber. Ten minutes retention in the flotation unit are sufficient for flocculation-flotation. This concept was tried on a 2 m^3 electroflotation unit (Table 1) and in a 1 m^3 DAF unit. Results obtained with the electroflotation unit showed good removal efficiencies at the following operational conditions: 150 mg/l alum, 20 ampere, 12 coulomb/l, 10 minutes retention, 6 m/h surface loading; while harvesting microalgae from pond effluents containing 300 mg/l T.S.S.

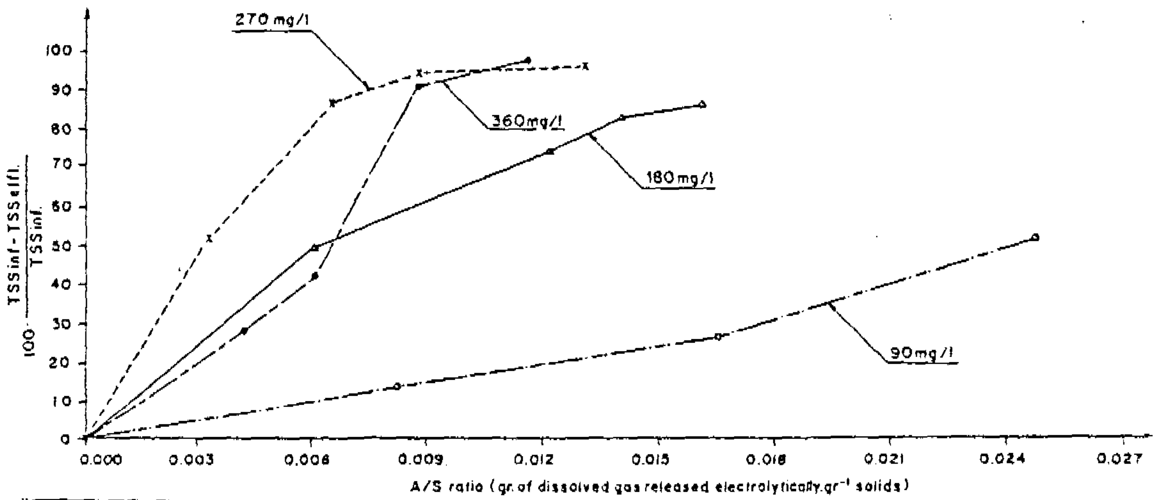

Fig. 3. Effect of the Air:Solids ratio on efficiency of flotation

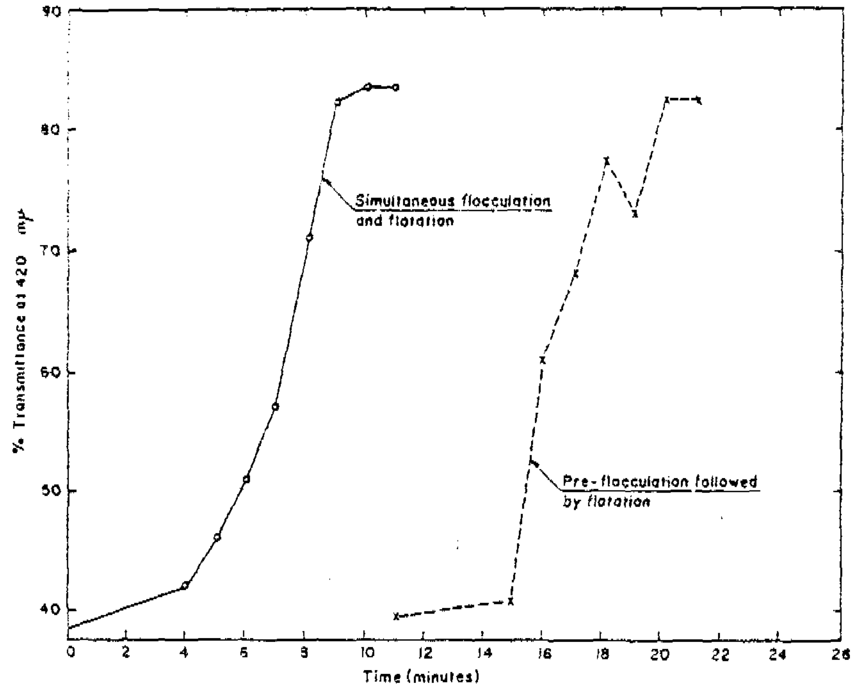

Fig. 4 Effect of performing flocculation and flotation simultaneously

Table 1. - Results of simultaneous flocculation - electroflotation in a 2 m^3 pilot plant unit.

Current, amp	Electric charge coulombs/1	Optical density (Klett 420 nm)	% Efficienty	Effluent TSS g/m^3	Effluent COD g/m^3
<u>0.15 g/1 Alum</u>					
Pond effluent		0,604		303	950
10	6	0,071	88.2	41	131
20	12	0,050	91.11	30.5	129
40	24	0,060	90.0	32.5	116
60	36	0,058	90.4	29.5	122
<u>0.20 g/l Alum</u>					
Pond effluent		0,540		288	890
5	3	0,096	82.2	32.5	130
10	6	0,066	87.7	31.5	-
20	12	0,046	91.4	28.0	-
30	18	0,051	90.5	26.0	-
40	24	0,039	92.7	21.5	-

The electroflotation process could find applications for harvesting of marine microalgae as a source of lipids for example, because less energy is needed for electrolysis at a high conductivity of the liquid media.

Electroflotation showed to have two major disadvantages: 1) the cathode is prone to scale formation and 2) power rectifiers are relatively expensive. An important advantage is that gas generation rate is directly related to current and can be easily changed according to oxygen concentration in the pond, in order to maximally use the photosynthetically generated supersaturated oxygen.

The dissolved air flotation unit was operated without slow mixing, using conventional flocculation-flotation techniques (Fig. 5).

The results of the DAF operated at a constant recirculation flow, while changing the main flow of pond effluent, showed that at an hydraulic load of 6 $m^3/m^2/h$ and 10 min., retention time resulted in al optimal A:S ratio of 0.0137 (Table 2).

Table 2. Effect of variating the A:S ratio in a DAF harvesting microalgae from HROP effluent.

Inflow (Q = l/min	Solids load ($gr/min/m^2$)	A/S	$\frac{TSS_{inf} - TSS_{eff}.100}{TSS_{inf}}$
150	52.5	0.4gr/52.5=0.009	78.6
100	35.0	0.0137	86.0 - 90.9
75	26.2	0.0183	83
50	17.5	0.0274	74.2
38	13.3	0.0361	80.0
25	8.7	0.0548	69.8

Operation of the modified DAF unit (Fig. 6) at the following conditions: a 10% recirculation rate; 3 atm. pressure; 10 min. retention, final pH 5:5; 160 mg/l alum; in-line injection of alum and acid with a retention of 15 sec. in the feeding line; and 0.0137 A:S ratio gave the following results. (Table 3).

Table 3 - Supernatant quality of the DAF unit treating HROP effluent.

	VSS	TSS	Total N	PO_4	COD	BOD
HROP effluent	327.7	388.8	76.7	34.0	644.4	139
DAF unit outflow	21.3	35.4	26.3	3.9	113.5	8
% removal	93.5	90.9	65.7	88.5	82.4	94.1

The solid concentration in the floc varied from 3 to 5% and could be increased by overnight decantation to 6-9 percent solids. This high solid content is of high importance considering the further dewatering and drying stages needed for commercial high value proteinaceous animal feed.

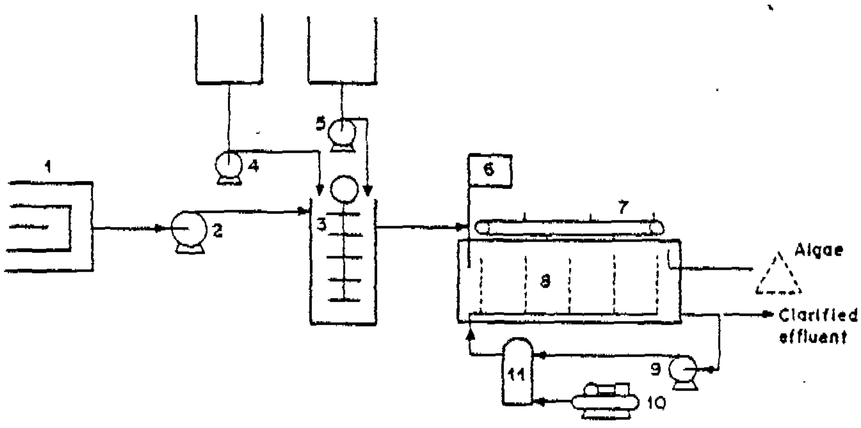

FIG. 5 Dissolved air flotation diagram.

1. H.R. Pond
2. Pump
3. Rapid mixing
4. Dosing pump flocculant
5. Dosing pump acid
6. pH meter & Controller
7. Scrapper
8. Flotation unit
9. Pressurization pump
10. Air compressor
11. Pressurization tank
12. Flotation channel

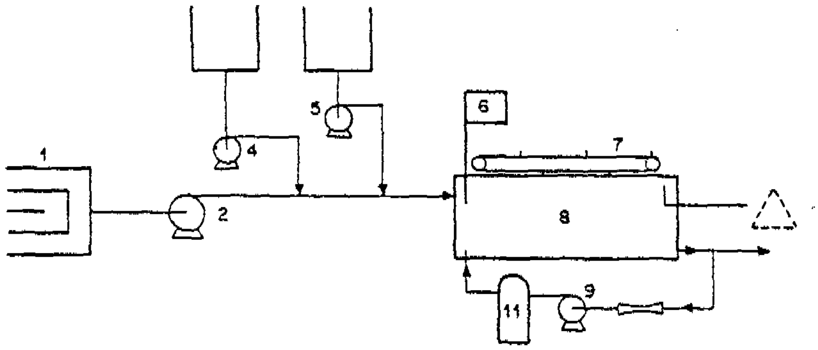

FIG.6 Modified D.A.F.(without flash mixing and compressor).

CONCLUSIONS

Flocculation-flotation is a technically feasible process for harvesting microalgae from HROP. Providing an initial concentrating factor of 300 to 600 which is the most important step for any further dewatering and drying of algal byproducts and for producing adequate effluent quality for discharge into receiving water bodies. Optimal conditions for flocculation and bubble entrappment or attachment have to be determined by jar test esperiments. Bubble entrappment can be achieved by releasing the pressurized stream in the flocculation area of a DAF tank so that the upflow velocity of the flocs increased. This results in shorter retention time and the slow mixing tank can be avoided altogether. Rapid mixing is possible by in-line injection of chemicals, making the rapid mixing tank and stirrer superflous. The replacement of the air compressor by venturi suction, further simplifies the system. If one maximizes oxygen bubbles of released from supersaturated pond effluents, flotation could be done with minimum recirculation. Simplified DAF is to be preferred to electroflotation for harvesting algae from HROP. However the latter may find applications for harvesting marine microalgae grown in saline culture media in mixtures of sea and wastewater or in supplementing oxygen to HROP oxygen supersaturated effluents.

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REMOVAL OF ORGANICS BY THE COMBINED PROCESS OF A STABILIZATION POND AND A HIGH-RATE OXIDATION POND AT A SEA-BASED WASTE DISPOSAL SITE

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ABSTRACT

Wastewater was collected from the sea-based North Port Waste Disposal Site, Osaka City, and a bench-scale treatment of the combined process of a stabilization pond and a high-rate oxidation was applied for more than 120 days. The maximum BOD removal rate coefficient and COD-manganese removal rate coefficient were 0.090 /day and 0.018 /day, respectively. These values were obtained when the retention time of the stabilization pond was 25 days. But, if the retention time was beyond or below 25 days, both removal rate coefficients were smaller. Therefore, we concluded that the optimum retention time of a stabilization pond was 25 days. This was reconfirmed by the Gel-Permeation Chromatography of the treated wastewater, a method for evaluating wastewater treatability proposed by Tambo and Kamei (Water Research, 12, 931-950, 1978). Furthermore, the Gel-Permeation Chromatography pattern of the treated wastewater in the bench-scale experiment coincided with that obtained by the actual field test. In addition, judging from Tambo's evaluation method, it could be concluded that higher-molecular organics were decomposed biologically into lower-molecular organics at the middle and bottom layer of the stabilization pond. BOD and COD-manganese values of below 10 mg/l and about 130 mg/l, respectively appear to be the maximum effective level of the combined process.

KEYWORDS

Wastewater treatment; sea-based waste disposal site; stabilization pond; high-rate oxidation pond; gel-permeation chromatography

INTRODUCTION

In this decade, adequate sites for waste filling in inland areas have dramatically decreased with the development of urbanization in Japan. Consequently, we have turned to the sea for waste disposal sites and their construction has started at a few cities.

Sea-based waste disposal sites are located and constructed in the sea; being independent artificial islands, they are never connected to land. Both the sea bottom is soft and water depth is usually between 4 and 8 m, making it -from the point of both technical and economical views- difficult to construct concrete basins in this environment for wastewater treatment. Besides, there are large quantities of the generated wastewater during waste dumping but they reduce when the inner water area is filled completely. The combined process of a stabilization pond (SP) and a high-rate oxidation pond (HROP) is thought to be the best wastewater treatment process for sea-based waste disposal site.

We constructed it in 1973, and reported the BOD removal mechanisms in the HROP (Yamada and Honda, 1978) and prediction method for wastewater's quality (Yamamoto et al., 1984). The effluent BOD value is consistently below 30 mg/l.

However, the improvement of the treated wastewater's COD-manganese (COD-Mn) value is required recently. Furthermore, we must construct a new waste disposal site until 1987. There are many points which must be solved to improve the treated wastewater's quality. Among them, we marked the role of the SP as a pretreatment process for the HROP. The volume of the SP is proportional to the progress of sea-filling, because, except for the HROP and the land produced by sea-filling, the remaining inner water area is the SP. Therefore, it is necessary to obtain SP's design criteria. For this purpose, we studied bench-scale experiments and the actual process.

MATERIALS AND METHODS

Site Description

The North Port Waste Disposal Site is being constructed outside the breakwaters in the sea west of Osaka City (Fig. 1). The site covers a total area of 2,080,000 m^2, out of which 420,000 m^2 is used as a refuse fill site (Site I) and the rest accommodates dredged soil from river and port facility construction and maintenance works. At Site I, about 1,000 tons of garbage and inorganic rubbish, 600 tons of refuse incineration ash, 200-600 tons of sewage sludge, and 400 tons of sewage sludge incineration ash have been disposed daily since 1973.

The amount of generated wastewater is nearly proportional to the amount of dumped waste with wastewater concentration increasing gradually according to the development of sea-filling. The generated wastewater was treated by a combined process of a SP and a HROP. The HROP uses five, 3,600 kgO_2/day, floating aerators and was made by dividing a part of the SP. Its volume and average depth are about 70,000 m^3 and 5 m, respectively.

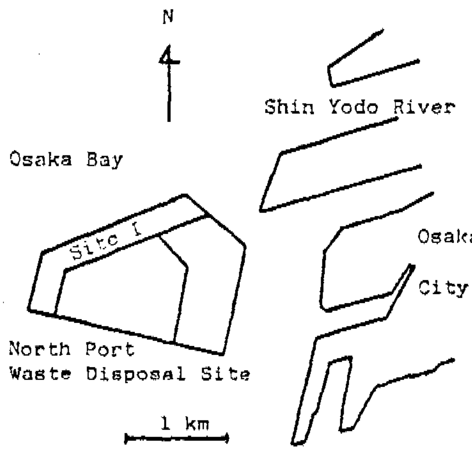

Fig. 1. The location of the North Port Waste Disposal Site.

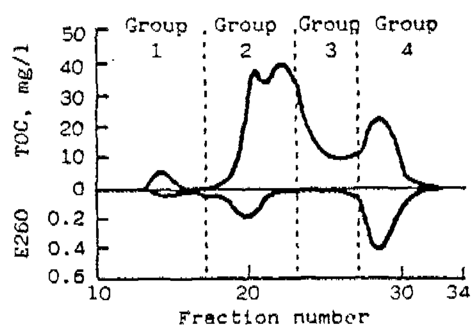

Fig. 2. Gel-permeation chromatogram of the wastewater generated at the North Port Waste Disposal Site (8. November 1982) used for the bench-scale experiments.

Bench-Scale Experiment

Wastewater Wastewater generated at Site I was used in this study. Its characteristics are shown in Table 1. These show that the generated wastewater was sea water which has been highly polluted with organics. However, the selfpurification seems to have been vigorously performed judging from the value of viable cell count. The wastewater's color was blackish-green and its smell was a mixture of hydrogen sulfide and the odor of decay. The wastewater's Gel-Permeation Chromatogram (GPC), Fig. 2, is a typical shape of raw organic wastewater such as sewage (Tambo and Kamei, 1978).

TABLE 1 Characteristics of Wastewater Generated at The North Port Waste Disposal Site (8, November 1982) Used for The Bench-Scale Experiment

Items	Value	Items	Value	Items	Value
pH	8.05	BOD	1,750	NO_3-N	0.14
Cl^-	12,700	TOC	670	KJN	176
SS	26	NH_4-N	11	Viable Cell Count	1.5×10^9
COD-Mn	612	NO_2-N	<0.01		

Units: Viable Cell Count, Colony Forming Units (cfu)/ml; all items except pH are mg/l.

Test unit Ninety liters of wastewater were stored in the SP for the set number of days, after which each of them was transported the HROP. The experimental SPs were made of 2 m length of 25 cm PVC pipes. Samples were collected from three layers, 10 cm (surface), 84 cm (middle) and 182 cm (bottom) under the water surface level. The HROPs were made of PVC vessels 50 cm long, 64 cm wide and 30 cm high. The water depth was 28 cm. Aeration was performed continuously at the rate of 3 l/min. All units were covered with transparent board to prevent evaporation and lighted 12 hr/day. The luminous intensity on the surface of the SPs and HROPs was 800 lux and 1,200 lux, respectively.

Analysis Unless otherwise specified, analysis were performed according to the Standard Methods for the Examination of Water and Wastewater (1980). Chlorophyll a (Chlo.a) was determined by Saijo's method (1975). The incubation condition for viable cell count was at 25 °C for 5 days. After incubation, colonies which appeared on each plate were counted and the viable count (i.e., colony forming unit) per ml was obtained.

For GPC analysis, the water-swollen Sephadex G-15 was packed to 90 cm in a 2.6 cm dia-meter and 1 m long glass column. This was used as fraction column. Prior to GPC separation, water samples were first filtered with a 0.45 μm membrane filter paper and, if necessary, concentrated by a rotary evaporator at 40 °C. The volume of each fraction when eluted with distilled water was 10 ml. Up to 40 fractions were taken. The TOC and UV absorbance at 260 nm (E260) of each fraction was measured.

Field Experiment

We prepared a pond at Site I. Its surface area and average depth were about 11,300 m^2 and 3 m, respectively. We dumped primarily refuse incineration ash and inorganic rubbish into the pond from 15 October to 31 December 1982. The average dumping amount per day was 1,100 tons. Summing up, about 52,000 tons of waste were dumped in this period. The pond was filled except for the small SP. The wastewater produced during filling was pumped and treated by the HROP in Site I. But after the filling, pumping was stopped to monitor the changes in water characteristics.

RESULTS AND DISCUSSION

Bench-Scale Experiment

SP treatment Table 2 shows the results obtained in the SP experiments. Concurrent, uniform decreases in BOD, COD-Mn and TOC values through the course of the experiment were not found clearly. Chlo.a values increased gradually. The SP, especially the surface layer, slowly became to an aerobic SP, though this was not evident in the changes in DO level. The viable cell count decreased at each individually analysed day except 101st day, and the number counted in the middle and bottom layers were more than in the surface layer. Nevertheless, BOD, COD-Mn and TOC values did not decline: Why the viable cell count decreased is not evident. The pH was almost stable between 7.20-7.35 except for 123rd day. During this period, between a pH of 7.35 and 7.55 the ammonification of KJN may have occured.

Several procedures have been proposed and used to determine the reduction of organics in SPs (Eckenfelder and Ford, 1970; Thirumurthi, 1979), resulting in considerable controversy over which is the most suitable design criteria (Mara, 1974; Mara and Gromiec, 1975). In these formulas, we selected the following,

$$\frac{Ce}{Ci} = e^{-KbT} \quad \text{and} \quad \frac{Ce}{Ci} = e^{-KcT}$$

Ce = BOD or COD-Mn in treated wastewater (mg/l),
Ci = BOD or COD-Mn in raw wastewater (mg/l),
Kb = first-order BOD removal rate coefficient (1/day),
Kc = first-order COD-Mn removal rate coefficient (1/day),
T = retention time (days).

The Kb and Kc value in the SP are shown in Fig. 3. They were calculated from the data in Table 2. Kb in the surface layer was 0.010 /day, the middle layer was 0.011 /day and the bottom layer was 0.015 /day. The Kb value increased in accordance with the water depth. In the SP, it is widely said that the change of structure from a high-molecular to lower molecular compounds happens more often in the anaerobic zone than the upper aerobic or microaerobic zone. If organic compounds of high-molecular order which are less responsible to bacterial activity, are decomposed into lower molecular compounds in middle and bottom layers, it should follow that the Kb values in these layers are larger than that in the surface layer. Therefore, we thought that this

TABLE 2 Changes in Water Characteristics in the Stabilization Pond Experiment

Days	Sampling Layer*	Water Temp. °C	DO mg/l	pH	Chlo.a mg/m^3	BOD mg/l	COD-Mn mg/l	TOC mg/l	Viable Cell Count** cfu/ml
25	Surface	19.8	0.2	7.35	0.5	500	411	200	3.9×10^{9}
	Middle	19.8	0.2	7.25	0.3	813	491	280	2.1×10^{10}
	Bottom	19.8	0.2	7.35	0.3	588	522	230	3.2×10^{10}
45	Surface	14.6	0.1	7.35	12	788	471	160	3.3×10^{10}
	Middle	14.4	0.1	7.25	11	900	527	310	4.0×10^{10}
	Bottom	14.4	0.1	7.25	56	825	547	180	4.7×10^{10}
74	Surface	11.4	0.2	7.35	173	750	491	230	3.0×10^{9}
	Middle	11.3	0.2	7.35	119	613	502	240	4.0×10^{9}
	Bottom	11.0	0.2	7.25	15	575	547	230	3.0×10^{9}
101	Surface	13.4	0.3	7.25	89	619	542	305	$<10^{6}$
	Middle	13.6	0.2	7.25	119	631	563	335	$<10^{6}$
	Bottom	13.2	0.2	7.20	170	625	537	300	$<10^{6}$
123	Surface	14.2	0.1	7.45	154	650	426	270	$<10^{5}$
	Middle	13.9	0.1	7.55	-	600	451	250	5.0×10^{5}
	Bottom	14.1	0.1	7.35	96	619	512	280	3.7×10^{5}

*; Surface, Middle and Bottom are 10 cm, 84 cm and 182 cm under the water surface level, respectively. **; cfu is the abbreviation of Colony Forming Unit.

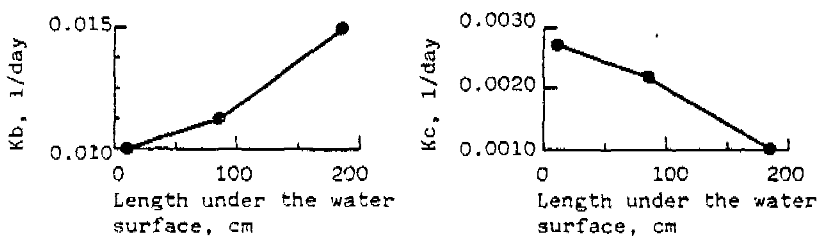

Fig. 3. Calculated values of BOD removal rate coefficient(Kb) and COD-Mn removal rate coefficient(Kc) obtained from stabilization pond experiment.

reaction is performed more often in middle and bottom layers. This can be confirmed by the fact that viable cell count in the middle and the bottom layers was greater than in the surface layer.

Kc in the surface layer was 0.0027 /day, the middle layer was 0.0022 /day, and the bottom layer was 0.0010 /day. Kc value decreased in accordance with the water depth. This is directly opposite to our findings for Kb.

Grouping of organic compounds for the treatability evaluation can be performed by a GPC (Tambo and Kamei, 1978). By using Sephadex G-15 gel with a two stage elution of distilled water and 0.1M-NH_4OH solutions, general organic compounds in a wastewater system are characteristically separated into six segments. On the GPC obtained by the 2.5 × 90 cm gel column, those six characteristics segments are marked with respect to the elution fraction number, and the treatability of each segments is characterized by the ratio of TOC/E260. In these segments, we use four divisions for evaluating the treatability of the wastewater: Group 1, Group 2, Group 3, and Group 4. Group 1 can be almost entirely removed by the chemical coagulation. The ratio of TOC/E260 is usually about 100. Group 2 responds poorly to removal by aerobic biological treatment (ABP), however, activated carbon adsorption (ACA) is very effective. Group 3 can be very effectively removed by the ABP. In this case, the ratio of TOC/E260 is 1,000 or more. It can be also removed effectively by ACA. Group 4 is very effectively removed by ACA. The ratio of TOC/E260 is about 50. In Group 1, Group 2 and Group 4, only a portion of TOC which is insensitive to E260 can be removed by ABP.

From the above, we can use Tambo's evaluation method to conclude that the key to the pretreatment process for ABP is how fast or effectively Group 3 increases. We show the GPC pattern of water treated in an SP for 25 days (Fig. 4). Both in the middle and bottom layers, Group 3's ratio of

TOC/E260 increased to more than 1,000 when compared with the ratio, calculated from Fig. 2, of raw wastewater. Group 2, which is not effectively treated by ABP, turned into biologically treatable Group 3. This shows that higher-molecular compounds were decomposed into lower-molecular compounds. Therefore, we can conclude that the decomposition of higher-molecular compounds occured in the middle and bottom layers of the SP through 25 days. However, if retention time in the SP continued beyond 25th day, the GPC pattern did not show an improvement of biological treatability. Refering from Tambo and Kamei (1978), the GPC pattern in the surface layer shows the typical wastewater pattern when less than optimum treatment by an ABP is used. We can see that Group 3's ratio of TOC/E260 is about 200, while Group 1, Group 2 and Group 4 have ratios of about 50.

From the above, we can conclude that the role of the SP is the improvement of wastewater's biological treatability under anaerobic condition.

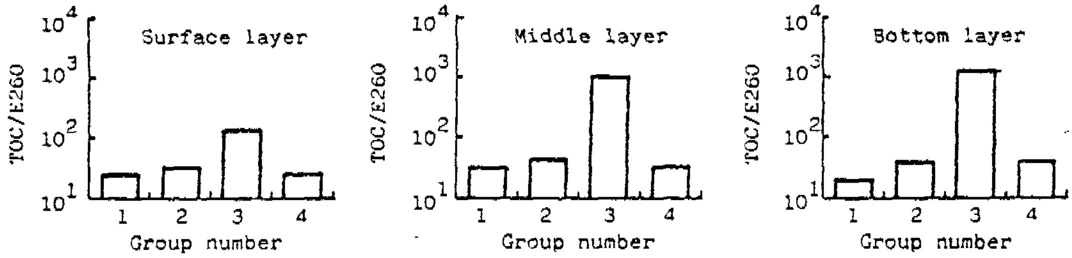

Fig. 4. Gel-permeation chromatogram pattern of the water treated in the stabilization pond for 25 days.

<u>The Combined Treatment</u> Table 3 shows the results obtained in the combined experiment of the SP and the HROP. The decrease in BOD and TOC values as the experiment proceeded is evident. Also, COD-Mn values decreased but were slightly scattered. The cause of this scattering may have been the production of nitrite-nitrogen by nitrification, judging from the changes in pH values. The Chlo.a value increased slightly. As turbulent conditions prevent phytoplankton's growth (Middlebrooks <u>et al</u>., 1982), they were not found in our experiments. The viable cell count decreased. Organic compounds which can be utilize easily by bacteria, the source of BOD, were consumed.

The value of Kb and Kc in the combined experiment of the SP and the HROP are shown in Fig. 5. They are calculated from data in Tables 2 and 3. When the wastewater was pretreated in the SP,

TABLE 3 Changes in Water Characteristics in The Combined Process Experiment of The Stabilization Pond and The High-Rate Oxidation Pond

Days	No. of SP Pre-treatment days*	Water Temp. °C	DO mg/l	pH	Chlo.a mg/m^3	BOD mg/l	COD-Mn mg/l	TOC mg/l	Viable Cell Count** cfu/ml
25	0	19.9	7.8	8.15	2.0	250	241	170	5.4×10^{10}
45	0	14.8	9.2	8.20	-	48	226	90	8.6×10^{11}
74	0	11.4	10.7	8.10	0.1	6.0	176	70	8.0×10^{10}
101	0	13.9	9.7	6.95	17.7	2.2	276	90	6.7×10^{7}
123	0	14.8	10.3	7.70	-	1.9	131	105	4.3×10^{6}
45	25	15.8	8.5	8.50	0.1	50	296	100	1.1×10^{12}
74	25	12.8	10.2	8.75	2.2	7.0	191	100	1.1×10^{11}
101	25	15.8	9.4	7.75	7.6	2.9	241	85	-
123	25	14.9	9.7	7.95	-	8.4	301	105	3.3×10^{6}
74	45	12.8	10.4	8.75	0.1	48	251	150	4.2×10^{11}
101	45	15.8	9.4	8.60	9.9	7.5	186	95	5.5×10^{9}
123	45	15.1	10.1	8.55	-	5.6	216	100	8.7×10^{7}
101	74	15.6	9.0	8.25	4.1	58	286	135	1.3×10^{9}
123	74	15.1	9.7	8.20	9.9	24	261	125	7.6×10^{7}
123	101	14.5	9.5	8.35	75	105	321	150	1.3×10^{8}

*; SP is the abbreviation of stabilization pond.
**; cfu is the abbreviation of colony forming unit.

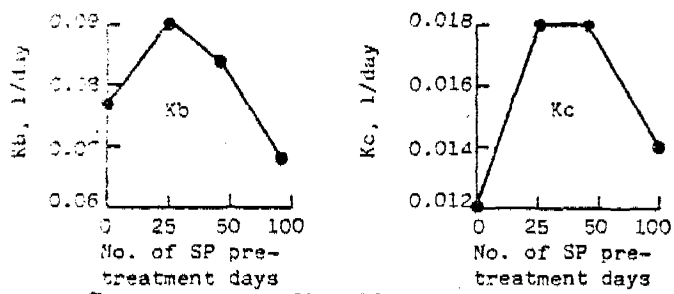

Fig. 5. Calculated values of BOD removal rate coefficients (Kb) and COD-Mn removal rate coefficients (Kc) obtained from high-rate oxidation pond experiment pretreated in stabilization pond (SP).

the initial BOD and COD-Mn values were 1,750 mg/l and 612 mg/l, respectively. When the wastewater was pretreated in the SP for 25 days, for example, the calculated average values of BOD and COD-Mn at 25 days in Table 2, 634 mg/l and 475 mg/l, respectively were used as the assumed initial values to compute Kb and Kc.

The maximum Kb value of 0.090 /day was obtained when wastewater was pretreated in the SP for 25 days. But beyond or below 25 days, Kb values became smaller. For this reason, if the wastewater was pretreated in the SP for 25 days then treated in the HROP, the BOD was most effectively removed. This result coincides with the result obtained from the GPC pattern's changes shown in Fig.4. Twenty-five days of pretreatment in the SP gives the wastewater characteristics which signify the most effective improvement for the ABP.

The GPC pattern of water treated in the HROP for 49 days after pretreatment in the SP for 25 days are shown in Fig. 6. Group 3 disappeared completely.

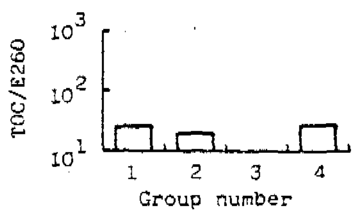

Fig. 6. Gel-permeation chromatogram pattern of the water treated in the high-rate oxidation pond for 49 days after pretreated in the stabilization pond for 25 days.

The maximum Kc values of 0.018 /day were obtained when the wastewater was pretreated in an SP for between 25 and 45 days. The Kc value declined beyond or below these days. The COD-Mn value of 130 mg/l appears to be the maximum effective level of the combined process; nevertheless, the BOD value was below 10 mg/l.

From the above, we can conclude that the optimum retention time of the SP as a pretreatment process for the HROP is between 25 and 45 days.

<u>Field Experiment</u>

BOD and COD-Mn values increased with the development of waste filling. The maximum BOD value was 1,800 mg/l, which was obtained when the filling finished, then the BOD values reduced gradually. On the other hand, the maximum COD-Mn value of 500 mg/l was obtained between 30 days after the start of the waste dumping and 15 days after the end of the filling. Then, COD-Mn value decreased gradually.

The GPC pattern of the wastewater collected at the end of the filling is shown in Fig. 7. The values of BOD and COD-Mn were 1,800 mg/l and 500 mg/l, respectively. The major portion was found in Group 2 and the ratio of TOC/E260 at Group 3 was about 400. The wastewater's biodegradability was low. However, after 1 month had passed (Fig. 8), Group 3 increased instead of a decrease in Group 2. Additionally, the ratio of TOC/E260 was at the high levels of around 3,000. The wastewater's biological treatability was improved.

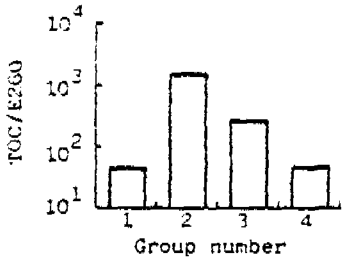

Fig. 7. Gel-permeation chromatogram pattern of the wastewater collected at the end of the waste filling of an experimental pond in the North Port Waste Disposal Site (6, January 1983).

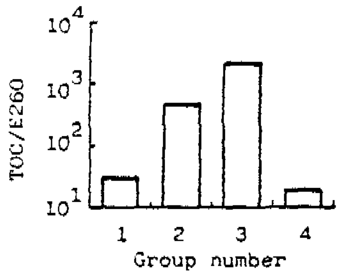

Fig. 8. Gel-permeation chromatogram pattern of the wastewater collected 1 month after the end of the waste filling of an experimental pond in the North Port Waste Disposal Site (14, February 1983).

The retention time in the SP, 1 month, is near to the optimum days for improvement of the biological treatability obtained in our bench-scale experiments.

CONCLUSION

Using wastewater generated from a sea-based waste disposal site, both bench-scale experiments and actual field experiments were performed to research the organics removal mechanisms active in the combined process of a SP and a HROP. Also, the SP was evaluated as a pretreatment process for HROPs. The following results were obtained.

1) In the SP, judging from the GPC's pattern changes, the decomposition of higher-molecular compounds occurs in the middle and bottom layers.
2) When used as a pretreatment process for the HROP, the optimum retention time in the SP was between 25 and 45 days. This result was obtained from kinetic analysis of the bench-scale experiments, and the GPC pattern's changes of both the bench-scale experiments and actual field tests.
3) BOD and COD-Mn values of below 10 mg/l and about 130 mg/l respectively appear to be the maximum effective level of the combined process.

We confirmed that Tambo's proposal (1978) is useful for studying the treatability of the wastewater. Furthermore, we constructed a combined process of a SP and a HROP at a new waste disposal site in 1986 by using the obtained values of Kb and K_S, and the optimum retention time in the SP. The volume of SP and HROP are 73,900 m^3 and 369,000 m^3, respectively. The HROP has six floating aerators of 3,600 kgO_2/day each.

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NUTRIENT REMOVAL IN HIGH RATE STABILIZATION PONDS IN COLD CLIMATES: SCANDINAVIAN EXPERIENCES.

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ABSTRACT

Results both from an pilot-plant investigation and from full-scale stabilization ponds operating with phosphate precipitation are presented. Since the removal of organic matter by coagulation is more important than that by biodegradation, the ponds may be heavily loaded. Biodegradation during winter can be improved by pond aeration, and chemical precipitation in an aerated pond may give a very good and stable effluent.

KEYWORDS

Wastewater, stabilization ponds, chemical precipitation, organics and phosphate removal.

INTRODUCTION

The use of traditional stabilization ponds is quite limited in Scandinavia, primarily as a result of the low temperature and snow and ice cover during winter time. Several pond-systems were closed in the 60-ties and 70-ties when more advanced plants, based upon chemical and biological/chemical treatment, were built.

The renewed interest in pond systems, is based on the urge to find reliable, stable, easy-to-operate and cheap wastewater treatment methods for small communities, tourist resorts, mountain recreational areas and so on. In many of these situations, the environmental protection authorities have found it necessary to use very strict effluent quality criteria in order to protect the often small and sensitive receiving waters that one can find in the mountain areas. In addition to suspended and organic matter, phosphorus has normally to be removed.

PHOSPHORUS REMOVAL IN STABILIZATION PONDS

Phosphorus removal may be combined with stabilization pond treatment by three different modes, pre-pond precipitation, in-pond precipitation and post-pond precipitation, see fig. 1.

The basis for the pre-pond and in-pond precipitation modes, is the fact that a very considerable amount of the organic pollutants in municipal wastewater is present in colloidal and particulate form. By the addition of precipitating chemicals, like aluminium, iron or lime, not only will the phosphate precipitate, but the particulate organic matter will coagulate as well. This is demonstrated in table 1 which shows the treatment results from 56 Norwegian non-biological chemical precipitation plants (Ødegaard, 1987).

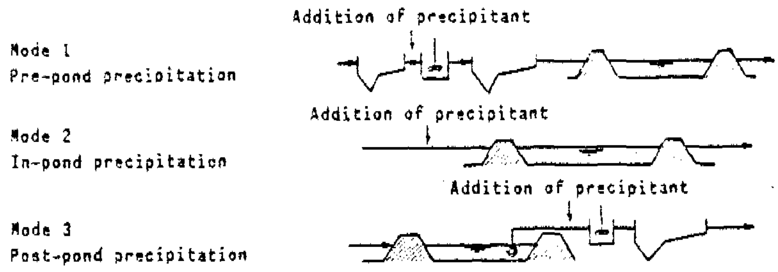


Fig. 1. Three modes of combining chemical precipitation and stabilization ponds.

TABLE 1 Treatment results from 56 Norwegian chemical treatment plants (Ødegaard, 1987)

Process	Number of plants	SS				BOD				COD				TOT. P			
		n 3)	in	out	%	n 3)	in	out	%	n 3)	in	out	%	n 3)	in	out	%
PP 1)	23	294	172	27	84,3	210	216	42	80,6	303	313	92	70,6	338	5,5	0,54	90,2
SP 2)	33	371	218	22	89,9	287	238	36	84,9	322	404	74	81,6	419	7,1	0,50	93,0
PP & SP	56	665	387	24	87,6	497	229	39	83,0	625	360	83	76,9	757	6,2	0,52	91,6

1) Primary precipitation 2) Secondary precipitation

3) Number of samples that the average values are based upon.

The advantage of the post-pond precipitation system is the improved clarification of the stabilization pond effluent.

Pilot study

A comprehensive pilot-plant investigation was carried out in order evaluate the three different treatment modes. The results from these experiments are presented in detail elsewhere (Balmer & Vik, 1978). In table 2 are only summarized the mean results over the year obtained in the high-rate ponds.

TABLE 2 Mean results from pilot experiments in high rate ponds over one year of operation (res.time = 4 d)(Balmer & Vik, 1978)

Type of effluent	Mean pond resid.time d	Mean org. load $gCOD/m^2 \cdot d$	COD_{UF} mgO/l	COD_F mgO/l	TotP mgP/l	PO_4-P mgP/l	SS mg/l
Raw	-	-	278	-	5.4	-	210
Effluent							
No prec.	3.9	60	189	96	4.7	3.6	73
In-pond prec.	4.1	58	85	54	0.55	0.22	35
Pre-pond prec.	3.9	22	83	50	0.72	0.11	46
Post-pond prec.	3.9	60	64	-	0.47	-	19

The best results were obtained by the post-precipitation mode, but both the pre-pond and the in-pond precipitation systems produced effluents almost of the same quality. The difference could be attributed to the better final particle separation in the post-pond precipitation mode.

At the very high load applied, the pond without precipitation only gave about 32 % COD-reduction, 13 % TotP-reduction and 65 % SS-reduction on a yearly basis, not much more than one would expect from primary treatment.

It is interesting to note then, that when alum was added to the influent of such a high-loaded pond, the effluent quality improved dramatically both in organic matter, phosphorus and suspended solids, with treatment efficiencies in the range of 70 % for COD, 90 % for TotP and 83 % for SS.

The organic matter removal in the in-pond precipitation mode was actually almost the same as in the pre-pond precipitation, even if the organic load in the latter was much lower as a consequence of the pre-precipitation taking place. This demonstrates that at these high loadings, the coagulation of organic matter plays a much more important role than the biodegradation in the ponds.

It is demonstrated, however, that both the pre-pond and the in-pond precipitation systems resulted in relatively high effluent SS-concentrations and that considerable improvement in both COD and phosphorus reduction could have been obtained with improved clarification of the effluent. This is also demonstrated by the fact that the results in the post-pond precipitation were a little bit superior to the other systems.

PRACTICAL EVALUATION OF DIFFERENT TREATMENT MODES

When returning to the application of the pond precipitation systems for small communities and especially for tourist recreational areas, it is very important to focus on the practical aspects of the different modes. Three matters should be addressed: The need for operator attendance, the sludge production and the possibility of odour.

The pre-pond and post-pond precipitation modes both require a traditional chemical treatment step either before or after the pond system. We know from experience that traditional chemical treatment plants require considerable qualified operator attendance in order to control the dosage of precipitant.

Even if the post-pond precipitation system may produce excellent treatment results, it is not without problems. Differences in water quality between day and night and floating algae due to oxygen supersaturation may cause operational problems in the post-precipitation step. High organic loading in the ponds may also lead to odour problems.

The principal advantage if the pre-pond precipitation system is that the major part of the sludge is taken out in the pre-precipitation step and that the sludge accumulation consequently is low. This is demonstrated in table 3 where the measured sludge accumulation in the different high-rate ponds of the pilot plant investigation is shown.

TABLE 3 Sludge accumulation in experimental high-rate ponds (res.time = 4 d) after one year of operation (Balmer & Vik, 1978)

	Height of accumulated sludge Infl.end cm	Middle cm	Effl.end cm	TS in pond %
No precipit.	20-30	10-15	5-10	4-6
Pre-pond prec.	5	5-10	5	1-2
In-pond prec.	90	60	30	3-5

Table 3 also demonstrates the major drawback of the in-pond precipitation mode, the large sludge accumulation. A high-rate pond of 4 days detention time would have to be desludged at least once a year. When the in-pond precipitation system is used in recreational areas, with highly variable loads, however, the pond is usually designed for the peak hydraulic load. This means that the pond may accumulate sludge for many years without the need for desludging, since the sludge accumulation is proportional to average load. The big advantages of the in-pond precipitation system, are that this kind of plant requires much less operator attendance and that both investment cost and operating cost are low. This mode has therefore gained the highest popularity in practice.

EXPERIENCES WITH FULL-SCALE PRECIPITATION PONDS

In table 4, the basic data for seven full-scale precipitation pond plants in Norway and Sweden are shown. The systems included shows a great variety in size, shape and pond configuration. All the plants are ice-covered during the winter. The Losby and Kjeller ponds were high-rate precipitation pond systems for residential areas, built as temporary solutions while waiting for connection to a bigger, central plant. The Stugun and Lungsjöen plants are also built for the permanently living population of the two communities. In these four plants the hydraulic load is as normal for residential areas. Except for the one in Lungsjöen, the mean organic load is quite high in these plants.

The plants at Nordseter, Edsaasdalen and Bjørnrike are all serving touristic areas with a tremendous variation in load. At Edsaasdalen, for instance, the permanent load is about 40 inhabitants while the load at its maximum, in springtime, is about 1200 inhabitants. The Nordseter plant, to which we shall return in more detail later, has undergone several changes since it was built in 1972. The results shown in table 5 are those obtained during 1973 to 1977 when the plant operated as a traditional in-pond precipitation plant.

TABLE 4 Operating conditions of various precipitation ponds in Scandinavia

Location	Pond area m^2	Numb. of ponds	Max. PE	Mean flow m^3/d	Mean resid. time, d	Mean org. load $gCOD/m^2 \cdot d$	Load at max PE m^2/PE	Mean prec. dosage g/m^3	Point of prec. addit.5)
Losby, N1)	6700	1	1800	450	18	29	3.7	155 2)	1
Kjeller, N1)	13000	1	6400	2100	9	140	2.0	85 2)	1
Nordseter, N	8000	3	800	260	31	9	10.0	150 2)	2
Stugun, S	9300	3	1000	260	30	24	9.3	100 2)	2
Lungsjöen, S	1425	2	70	19	110	1.5	20.3	36 3)	1
Edsaasdal, S	6800	3	1200	56	157	3.3	5.7	350 4)	2
Bjørnrike, S	6750	3	1500	80	124	3.5	4.5	150 2)	2

1) These plants are no longer in operation. 2) $Al_2(SO_4)_3 \cdot 14\text{-}16H_2O$. 3) Fe_{III}. 4) $Ca(OH)_2$.
5) Inlet to pond number.

TABLE 5 Mean treatment results in some Scandinavian in-pond precipitation plants

LOCATION	COD, g/m^3 in	out	%	Tot P, g/m^3 in	out	%	SS, g/m^3 in	out	%	Ref.
Losby, N	426	136	68	9.1	2.2	70	283	53	81	Ødegaard, 1973
Kjeller, N	864	265	69	8.8	1.7	81	672	48	93	Balmer et al, 1978
Nordseter, N1)	265	83	69	4.9	1.2	76	152	30	80	------ " ------
Stugun, S	652	109	83	7.1	1.1	85	-	-	-	Hanæus, 1984
Lungsjöen, S	109	80	27	3.1	0.39	87	-	-	-	---- " ----
Edsaasdal, S	398	126	68	6.2	0.38	94	-	-	-	---- " ----
Bjørnrike, S	292	66	77	3.6	0.32	91	-	-	-	---- " ----

1) The results represent the situation before the plant was altered in 1979. See fig. 3 and fig. 4 for later results.

Table 5 shows that the effluent quality of the different plants does not seem to be correlated to the load in any way. The treatment results regarding organic matter are about what one would expect from pure chemical treatment plants, or a little poorer, primarily caused by the organic suspended solid concentration in the effluent. The variation in the purification efficiency of total phosphorus can be explained by inadequate dosage of precipitant in the cases where the total phophorus in the effluent is higher than 1 mgP/l.

The full scale experiences seem to confirm the ones from the pilot investigation, in that the precipitation ponds are primarily chemical treatment plants, and to a much lesser extent biological treatment plants. The loading may therefore be quite high compared to traditional biological oxidation ponds. The high load results, however, in two distinct disadvantages:

a) The necessecity of frequent desludging
b) The possibility of anaerobiasis and odour as a consequence of this.

In order to deal whit these problems, some modifications to the traditional design may be implemented, such as those carried out at the Nordseter plant, to which we shall return later.

The process of desludging is normally carried out as follows:

1. The pond to be desludged is drained for water.
2. The bottom sludge is allowed to dry up for some time.
3. The sludge is moved by front loaders to an area where it is allowed to freeze during winter.

Freezing of sludge represents an excellent dewatering alternative, and desludging is therefore often carried out in autumn just before the temperature falls below zero.

EXPERIENCES FROM THE NORDSETER PLANT

The plant at Nordseter, Norway is situated in a recreational district 700-1000 m above sea-level. It was constructed in 1972 in order to provide treatment for hotels and cottages in the area. A military camp was also connected to the plant in 1975. The load is consequently highly variable. The plant was originally designed to treat the wastewater from 600 PE. This has after some process modifications been raised to 800 PE in 1979 and 1300 PE in 1986.

The plant has three ponds with a total surface area of 8000 m^2, see fig. 2. The precipitant, a technical grade alum, was from the start added to the pipe between the first and the second pond (fig. 2).

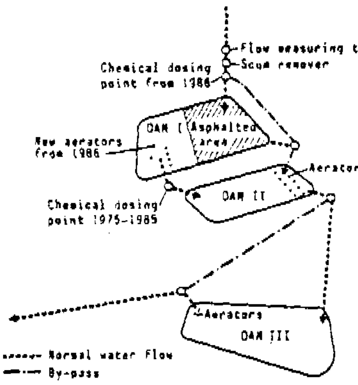


Fig. 2. The lay-out of the Nordseter plant

Table 5 shows typical results from the first period of its operation.

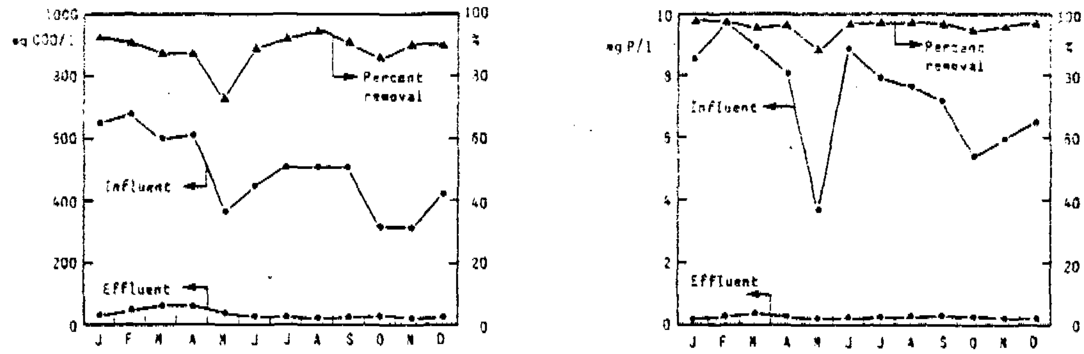
The average treatment results were quite good, but the effluent quality varied a lot over the season especially with respect to organic matter. Occasional poor phosphate removal could be attributed to dosing equipment failure. The BOD-removal was found to be excellent in the summer, but during the ice cover periods, the treatment results reduced to the same level as could be expected from a treatment plant with chemical precipitation only.

In order to improve the effluent quality, especially with suspect to organic matter, aerators were installed in 1979 at the outlet of dam 2 and 3, in order to meet the oxygen demand in the critical late winter period when the loading is high (tourist season) and the photosyntetic activity is low (ice cover).

The treatment results over the period from 1979-1985 are shown in fig. 3 and 4. The results are remarkable both with respect to effluent quality and stability.

When looking at the results in fig. 3 and 4 in more detail, one can find a consistent drop in treatment efficiency in May, being the month when the ice-cover melts. It can be seen that the COD-removal during the winter-time is poorer than during summer-time indicating, as one might expect, a more advanced biodegradation during the aerobic period in summer than the possible anaerobiasis under ice-cover in winter. On the contrary, the SS-concentration is higher during summer as a result of algae in the effluent.

Since the recipient is a sensitive and small mountain brooke, the environmental authorities have set very strict effluent criteria when they allowed the load to be raised to 1300 PE in 1986. The average yearly effluent concentrations of shall not be higher than 10 mg BOD_7/l and 0.4 mgP/l and the maximum concentration shall not exceed 20 mgBOD_7/l and 0.8 mg P/l.



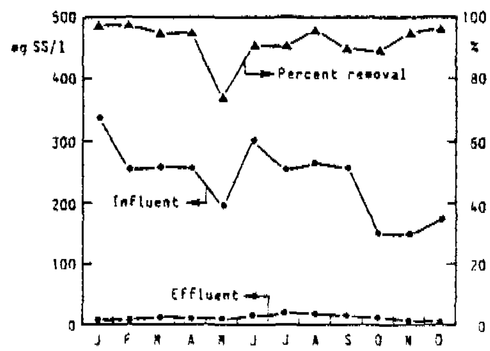


Fig. 3. Variation over the year of influent, effluent and percent removal based upon mean value for each month in the period 1979–1985 at Nordseter precipitation-pond plant.

In order to meet these very strict criteria, both the chemical and the biological process had to optimized. The changes now carried out are as follows:

a. The chemicals are now added flow-proportional to the inlet of the first pond.
b. The bottom of the half of the first pond closest to the inlet has been asphalted and the two halves of the first pond has been divided in order to be able to desludge the first part separately.
c. Aerators have been installed at the outlet of the first pond, as well as those already at the outlet of pond 2 and 3.

CONCLUSIONS

It is demonstrated by the Scandinavian experiences that good treatment results for small communities and touristic areas can be obtained by the use of precipitation ponds even under cold climate. The following conclusions and recommendations could be put forward based on our experiences:

a) The in-pond precipitation mode is the simplest and cheapest and may give satisfactory results (70–90 % BOD-reduction, 85–95 % Tot P-reduction).
b) The pond system should be divided in at least two, preferably three ponds and a maximum of flexibility with respect to by-pass possibilities and choice of precipitant dosing point should be incorporated in the design.
c) Chemical precipitation plays a more important role than biodegradation in the removal of organics in precipitation ponds. The design load should therefore be determined by the acceptable frequency of desludging and acceptable odour emission.
d) The sludge accumulation is in the order of 1–1.5 m^3/PE·year. In ultra high-rate systems (mean residence time $\leq$ 6 d or < 3 m^2/PE calculated on the pond following chemical addition only), and high-rate systems (total mean residence time $\leq$ 30 d or $\leq$ 10 m,2/PE), the chemicals should be added flow proportional to the inlet of the first pond which preferably should be deeper (1.7–2.0 m) than the others in order to serve as sludge storage.
e) In moderately loaded systems (total mean residence time 30–60 days or 10–20 m^2/PE) the easiest operation is achieved by constant dosing of precipitant to the inlet of the second pond, since the first pond equalizes both flow and quality of the water.

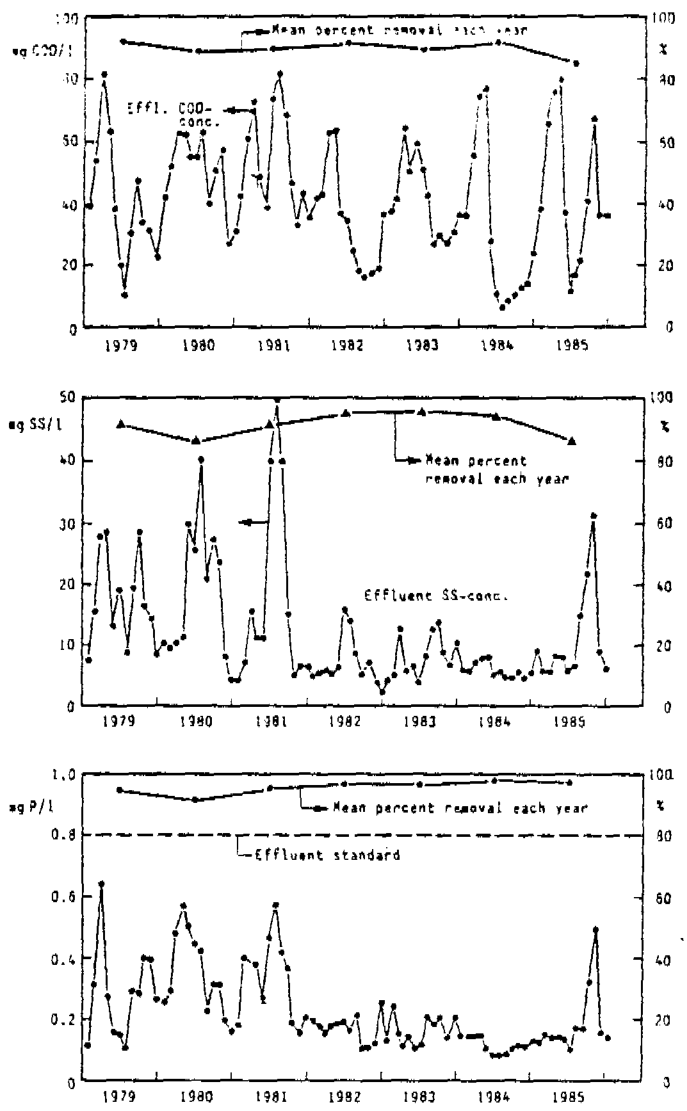


Fig. 4. Variation in effluent concentration and percent removal in the period 1979–1985 at Nordseter precipitation-pond plant.

f) Odour must be expected from ultra high rate and high rate systems. Odour nuisance can be greatly reduced by aerating the effluents of each pond. This will also result in improved removal of organic matter.

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THEME 7

MACROPHYTES

AERATION AND WATER HYACINTHS IN WASTE STABILIZATION PONDS

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ABSTRACT

The investigation was based on two facultative stabilization ponds initially designed to operate in parallel, and now receive wastewater in excess of their capacities from a fast expanding housing estate in the Caribbean Island of Trinidad. Because of the deterioration of the effluent quality relative to acceptable standards, an attempt was made to upgrade the ponds using water hyacinths at the early stages. However, from the results, it was clear that the introduction of water hyacinths in the test pond did not lead to any substantial improvement in the effluent because of the high loading on the pond. Therefore the ponds were modified to operate in series with surface aerators installed in the first pond. Initially, the effluent quality was monitored in terms of total suspended solids, volatile suspended solids, biochemical oxygen demand, faecal coliform bacteria, pH and dissolved oxygen with aeration in the first pond and no aquatic plants in the second pond. Although there was a significant improvement in the effluent quality, the values remained above the standards. As a result, water hyacinths were introduced in the second pond and the effluent quality monitored together with aeration in the first pond. The effluent quality improved with total suspended solids and biochemical oxygen demand values both as low as 10 mg/l in certain months, but additional treatment needed to reduce faecal coliform.

KEYWORDS

Sewage treatment; upgrading stabilization ponds, aerated lagoons, water hyacinths.

INTRODUCTION

In any country, the protection of water resources against pollution is essential to the development of a sound economy and for the maintenance of good public health, and it is important that pollution be controlled. In developing countries, as the standard of living increases, household privies, septic tanks, package treatment plants and other sophisticated methods of disposing excreta may create economic and health problems.

Waste stabilization ponds are shallow man-made basins utilising natural processes and are used in many parts of the world to treat wastewater from both small and large communities. For tropical developing countries, the factors which make the stabilization pond treatment most suitable are, land may be inexpensive especially in rural areas, low operating cost, maintenance requirements are minimal, shortage of technical skill and severe financial constraints. A properly designed pond system will provide an effluent to the required quality with low operating costs. However, in many places, the existing stabilization ponds are either badly designed, or overloaded due to expanding population. The primary objective of this study is to improve the effluent quality of a poorly designed overloaded waste stabilization pond system in the Caribbean island of Trinidad.

AERATED LAGOONS

Stabilization ponds overloaded due to increased organic loading or reduced temperature often produce odours and insufficient organic load removals. Aeration and mixing provide distribution of dissolved oxygen for decomposition of organic matter when oxygenation by algae and wind mixing are not sufficient. Aerated lagoons (or ponds) are evolved from facultative stabilization ponds and they are activated sludge units operated without sludge return. Floating aerators are most commonly used to supply the necessary oxygen and mixing, and are spaced to provide uniform blending for dispersion of dissolved oxygen and suspension of microbial solids.

WATER HYACINTHS IN STABILIZATION PONDS

Aquatic plants can be grown in waste stabilization ponds for the purpose of purifying wastewater. Among the different aquatic plants, water hyacinth (Eichhornia Crassipes) is the most popular plant grown in wastewater ponds. The water hyacinth is a free floating fresh water plant growing in the natural state in many countries of the world. It is one of the fastest growing plant in existence, resistant to many insects and diseases and could thrive in raw sewage. Water hyacinth systems are considered to combine filtration and fixed-film biological conversion processes. The suspended solids passing through the roots get entrapped, accumulate and finally settle by means of gravity. The organic pollutants are degraded into carbon dioxide, ammonia, phosphate and sulphate as end products which are absorbed by water hyacinths through the roots. Ponds with water hyacinths growth are much more effective than conventional treatment processes in the removal of pollutants. Also, they are capable of controlling algal overgrowth, thus preventing the presence of large amounts of algae in the effluent.

Therefore, water hyacinths have been used to upgrade full-size stabilization ponds receiving domestic sewage. Dinges (1978) used water hyacinth in the final treatment unit to upgrade the effluent from a large system consisting of an activated sludge plant and two aerated tanks operating in parallel followed by three stabilization ponds. In another study McDonald and Wolverton (1980) found significant improvement in the effluent quality when water hyacinths covered the entire lagoon. Recently Orth et al (1987) obtained encouraging results from pilot scale studies using water hyacinths for the treatment of raw wastewater discharged by small factories and housing areas of an industrial estate.

ARRANGEMENT OF PONDS AT TRINCITY, TRINIDAD

The investigations were based on a treatment system which consists of two facultative stabilization ponds initially designed to operate in parallel and now receive wastewater in excess of their capacities from a fast expanding housing estate in Trincity in the island of Trinidad. When the pond system was constructed in 1973, each pond was 90 m long, 66 m wide and 1 m deep. The ponds were designed to treat 1140 m^3/d of sewage with 5-day bio-chemical oxygen demand (BOD_5) and suspended solids loadings of 325 kg/d and 380 kg/d respectively from about 1100 houses. At present the pond system receives a sewage flow of 3000 m^3/d from about 2500 houses and from a commercial complex. If the expansion programme of the housing estate continues, it is anticipated that there will be about 6200 houses by the year 2000. Also, around 50 hectares have been earmarked for commercial and industrial development. The pond system was adequate in the early stages of the development, but clearly falls short of what is required for the treatment of wastewater emanating from the completed estate and the future projections. The flow rates and the corresponding hydraulic parameters for one pond are presented in Table 1.

Table 1 Hydraulic Parameters for Different Flows

Sewage flow m^3/d	Retention time d	Application rate m^3/m^2d	Loading rate m^3/m^2d
1140	5.2	17.27	0.19
2250	2.6	34.09	0.38
3000	2.0	45.45	0.51

Because of the deterioration in the effluent quality relative to the acceptable standards from both ponds, the effluent from one of the ponds was monitored before and after introduction of water hyacinths and some of the results are given in Table 2 (Thomas and Phelps, 1987).

Table 2 Initial Performance Results

Component	Without Water Hyacinths Mean Reduction %	With Water Hyacinths Mean Reduction %
Total suspended Solids	35	41
BOD_5	78	69
Faecal coliform	79	90

From the results, it was clear that the introduction of water hyacinths in the test pond did not lead to any substantial improvement in the effluent quality because of the high loading on the pond. Therefore, it was evident that to improve the effluent quality, it was necessary to either build additional ponds or to introduce new treatment facilities and processes. Because there is no more land available for expansion and to keep the capital cost as well as operational and maintenance costs to a minimum, it was decided to retain and upgrade the existing ponds.

AERATION AND WATER HYACINTHS IN POND UPGRADING

A convenient and fairly economical method of increasing the capacity of a waste stabilization pond is to convert it to an aerated lagoon by the use of floating mechanical aerators. Therefore, in order to investigate this, the ponds were modified to operate in series with six 1.5 kw surface aerators installed in the first pond and the modified arrangement is shown in Figure 1.

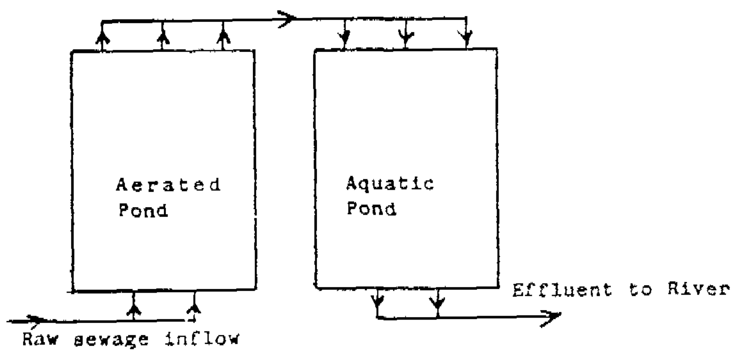

Fig. 1 Layout of ponds

The depth of the aerated pond was increased to 3 m and operational parameters for the present flow of 3000 m^3/d are given in Table 3.

Table 3 Operating Data on the Modified Pond System

	First Pond	Second Pond
Inflow rate (m^3/d)	3000	3000
Length (m)	90	90
Width (m)	66	66
Mean depth (m)	3.0	1.0
Surface area (m^2)	5940	5940
Cross sectional area (m^2)	198	66
Hydraulic - retention time (d)	5.9	2.0
- loading rate (m^3/m^2d)	0.51	0.51
- application rate (m^3/m^2d)	15.15	45.45

The effluent quality was monitored in terms of the parameters total suspended solids, volatile suspended solids, BOD_5, faecal coliform, pH, and dissolved oxygen. Although there was a significant improvement in the effluent quality, the concentrations remained above the acceptable standards. As a result, water hyacinths were introduced for the full coverage in the second pond and the effluent quality examined together with aeration in the first pond.

It was found that the ratio of BOD_5 to total suspended solids decreased in general from influent to effluent with aeration in the first pond alone as well as with aeration in the first pond and water hyacinths in the second pond as shown in Table 4.

Table 4 Ratio of BOD_5 to Total Suspended Solids

	With aeration in the first pond	With aeration in the first pond and water hyacinths in the second pond
Influent	0.98	0.90
Effluent	0.46	0.56

The general reduction in the ratio from influent to effluent is due to the conversion of the volatile organic matter in the ponds to the non-volatile suspended matter. However, the increase in ratio observed in the effluent with water hyacinths in the second pond is due to the reduction of algal solids in the effluent. The non-volatile suspended solids averaged about 20% in the wastewater and a reduction of 69% of volatile suspended solids was with aeration only in the first pond, whereas a reduction of about 87% was recorded with both aeration in the first pond and water hyacinths in the second pond.

Water hyacinth systems are capable of producing effluents with total suspended solids concentrations and BOD concentrations both as low as 10 mg/l as is evident from the effluent measured over a seven-month period and shown in Figures 2 and 3 respectively.

The dispersed points on the Figures 2 and 3 indicate varying degree of total suspended solids and BOD_5 removal due to differing conditions on the pond system. Initially with aeration only in the first pond, the total suspended solids reduction averaged 64% and with the water hyacinth growth in the second pond the total suspended solids reduction improved to 87%. The mean BOD_5 concentration of the effluent was 31 mg/l with a reduction of 85% with aeration in the first pond. With water hyacinths in the second pond, the effluent BOD_5 averaged 14 mg/l with a reduction of 92%.

The percentage reductions in total suspended solids and BOD_5 recorded with aeration in the first pond and water hyacinth growth in the second pond are higher than the values reported in an earlier study (Thomas and Phelps, 1987). However, the percentage reductions

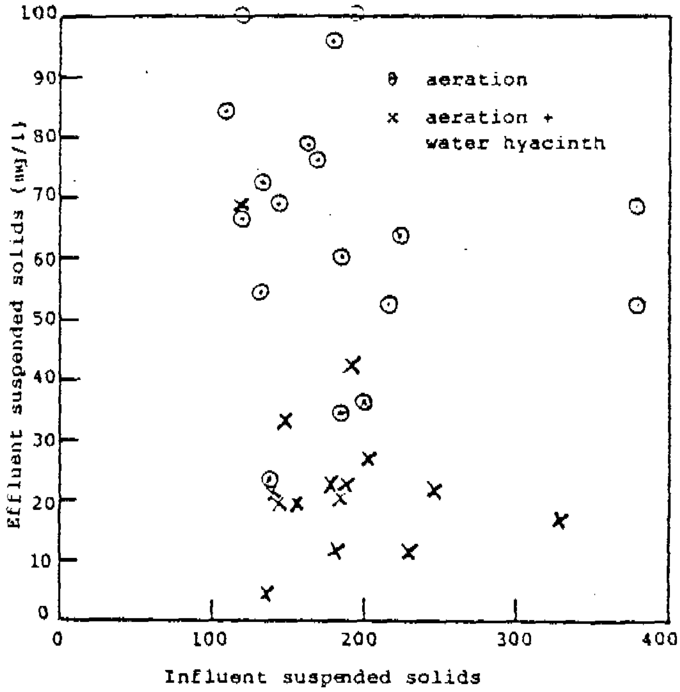

Fig. 2 Effluent suspended solids concentrations

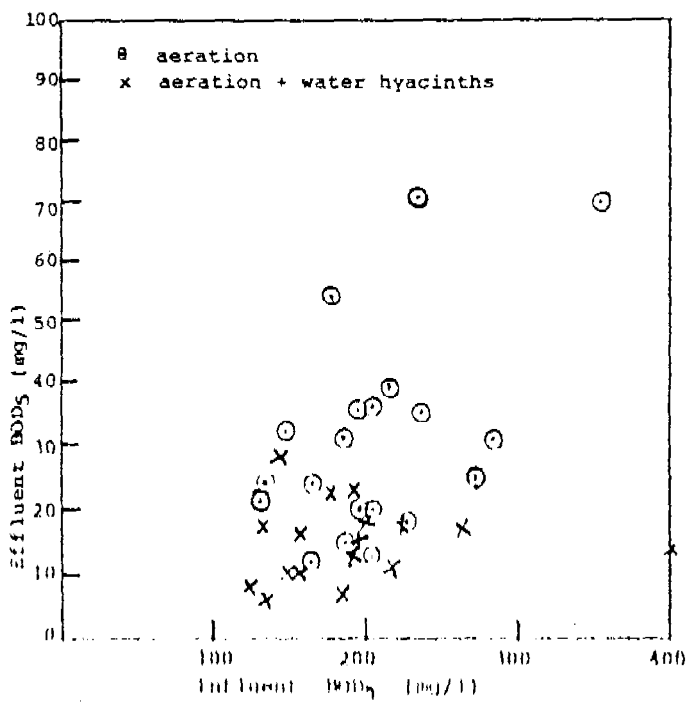

Fig. [illegible] Effluent BOD_5 concentration are about the same as those obtained by Wolverton and McDonald (1979) from an investigation on a single aquatic pond with 54 days retention time. Average BOD_5 loading and the removal rates for the present study are shown in Figure 4.

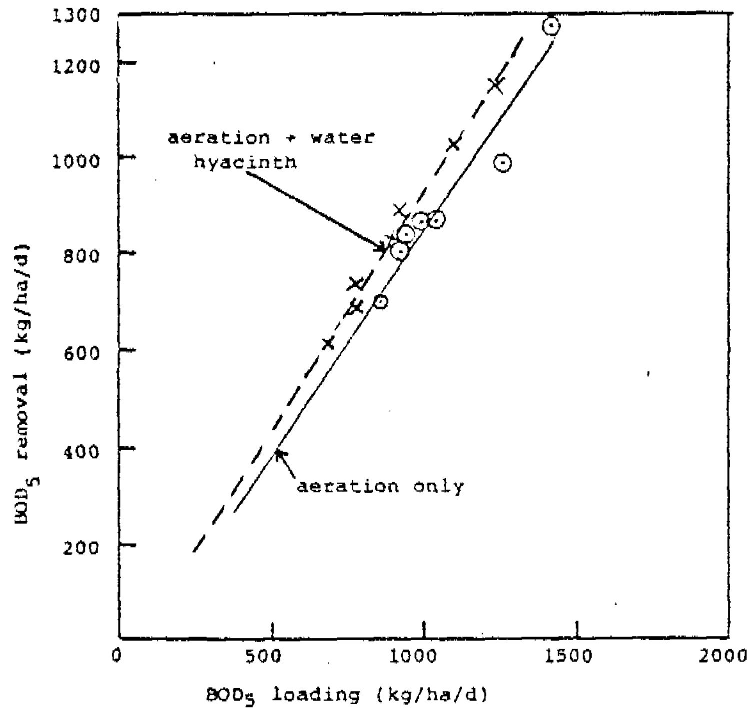

FIG. 4 BOD_5 Removal as a function of BOD_5 loading

The average results of faecal coliform tests are compared with the calculated values at 27°C in Table 5.

Table 5 Faecal Coliform Concentrations

Influent Concentration per 100 ml	Calculated effluent concentration per 100 ml	Experimental effluent concentration per 100 ml
With aeration in the first pond		
2.61×10^7	0.26×10^5	1.40×10^5
With aeration in the first pond and water hyacinths in the second pond		
2.60×10^7	0.26×10^5	1.10×10^5

Although about 99% reduction in faecal coliform was recorded in both cases, the effluent faecal coliform concentrations did not satisfy the required standards. This is in agreement with Mara (1978) that aerated ponds are not particularly effective in removing faecal coliform and further treatment may be necessary.

During the experiments with aeration in the first pond, the influent and effluent pH values averaged 7.4 and 8.1 respectively and with aeration in the first pond and water hyacinths in the second pond, the influent and effluent pH values averaged 7.5 and 7.7 respectively. The increase of 0.7 units with only aeration was due to the depletion of carbon dioxide in the second pond by the algal photosynthesis, whereas the increase of only 0.2 units in the other case was due to the water hyacinths in the second pond.

The dissolved oxygen content of the effluent decreased from an average value to 7.9 mg/l with aeration in the first pond to 5.9 mg/l with aeration in the first pond and water hyacinth growth in the second pond. This is because the oxygen produced by water hyacinths during photosynthesis does not contribute to the oxidation process within the pond, thus creating anaerobic conditions in the second pond.

SUMMARY AND CONCLUSIONS

From the results described above, the applicability of aeration and water hyacinths in waste stabilization ponds can be confirmed even with a high organic loading on the pond system. Improved effluent quality results were obtained with the two ponds modified to operate in series with aeration in the first pond and water hyacinths grown in the second pond as a result of reduced organic loading on the aquatic pond. This is in agreement with the conclusion made by the authors (Thomas and Phelps, 1987) in a previous study. High percentage reductions of total suspended solids, 5-day biochemical oxygen demand and faecal coliform were recorded with the effluent total suspended solids concentration and 5-day biochemical oxygen demand both as low as 10 mg/l. However, to achieve an acceptable level for faecal coliform concentration additional treatment in the form of maturation ponds or disinfection is needed. Environmental impacts that could result from the use of water hyacinths in sewage ponds include odours, presence of flies and mosquito nuisance. For rural areas and housing estates especially in tropical climates, the use of aeration and water hyacinths in a stabilization pond system is suggested.

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BOD$_5$ REMOVAL IN FLOATING AQUATIC MACROPHYTE-BASED WASTEWATER TREATMENT SYSTEMS

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ABSTRACT

Floating macrophytes cultured in ponds used for secondary wastewater treatment (BOD$_5$ removal) apparently serve two functions: they provide a substrate for bacterial attachment, and their vascular lacunae act as a conduit for the transport of O_2 from the atmosphere to the wastewater. In the present study, floating macrophyte systems containing pennywort (Hydrocotyle umbellata) were found to be 10% more effective (on an annual average) at removing wastewater carbon (BOD$_5$) than those containing the more productive water hyacinth (Eichhornia crassipes). This enhancement in BOD$_5$ removal is thought to be due to pennywort's superior O_2 transport capability. In a separate six month long study, plant harvesting was found to have no effect on BOD$_5$ removal in a water hyacinth-based treatment system. These data demonstrate that operational strategies used in floating macrophyte-based nutrient removal systems (i.e., frequent harvest of highly productive plants) are inappropriate for those systems utilized for carbon removal.

KEYWORDS

BOD$_5$ removal; water hyacinth; pennywort; O_2 transport; plant harvest.

INTRODUCTION

Floating aquatic macrophyte-based treatment systems (FAMS) consist of shallow (0.5-1.0 m) ponds containing aquatic plants which receive wastewater at a retention time ranging from one day to several weeks. In the United States, FAMS are most commonly used as an advanced treatment process, to remove nitrogen (N) and phosphorus (P) from secondary domestic effluent. The plant which has been utilized to the greatest extent in FAMS is the water hyacinth (Eichhornia crassipes), a productive tropical plant with a high nutrient uptake capability (Reddy, 1987). Water hyacinths are frequently harvested from FAMS in order to promote plant growth and to provide an ultimate means of removing nutrients from the wastewater (Boyd, 1970; Dinges, 1978; Wolverton and McDonald, 1979; Duffer, 1982; DeBusk et al., 1981,1983; Hauser, 1984; Reddy et al., 1985; Hayes et al., 1987).

Recent pilot-scale studies have shown that FAMS can also be used to remove carbon (BOD$_5$) from primary domestic effluent. In contrast to FAMS used for advanced wastewater treatment (e.g., P uptake), in which plant assimilation is a dominant contaminant removal process, FAMS used for BOD$_5$ removal are thought to function as fixed film reactors, with the submersed plant structures acting as a substrate for bacteria (Stowell et al., 1981; Tchobanoglous, 1987). The plants may also supply some of the O_2 required for the oxidation of wastewater carbon. Several researchers have noted that many floating macrophytes can transport atmospheric O_2 from the foliage into the root zone (Armstrong, 1964; Moorhead and Reddy, 1987). Oxygen not required for root respiration may diffuse into the wastewater and be utilized by bacteria for the oxidation of BOD_5. The role of the plants in enhancing BOD_5 removal in FAMS, whether that of providing a substrate or transporting O_2 to the heterotrophic bacteria, is still poorly understood. In this paper we present results of meso-scale outdoor studies in which we examine the effects of macrophyte species and harvest regime on BOD_5 removal from primary domestic sewage effluent.

METHODS

The following outdoor tank studies were conducted at the Community Waste Research Facility of the WALT DISNEY WORLD Resort Complex, located near Orlando, FL, USA.

Experiment I

Two batch incubations (each 2 to 3 weeks long) were conducted in four 1000 L tanks (1.7 m^2 surface area, 0.5 m deep) to compare BOD_5 removal in systems with and without floating macrophytes. In the first incubation, two tanks were filled with primary domestic effluent and stocked with water hyacinths at a standing crop of 10 kg (fw) m^{-2}. Two other tanks were filled with primary effluent and covered with an opaque screen, which was situated so as to block sunlight but not interfere with atmosphere-water gas exchange. Biochemical O_2 demand (BOD_5) concentrations (APHA, 1985) of the wastewater were measured four times over a 15 day period. Dissolved O_2 measurements of the wastewater were also periodically conducted at 0900 hr at a depth of 10 cm. The second incubation, 21 days long, was similar to the first, except that the tank devoid of macrophytes was not screened.

Experiment II

BOD_5 removal by two FAMS, one containing water hyacinth and the other, the floating macrophyte pennywort (*Hydrocotyle umbellata*), was examined for a one year period. Plants of each species were stocked at a standing crop of 10 kg m^{-2} into duplicate, 1000 L tanks. Primary sewage effluent in the batch fed tanks was changed at 3 to 4 day intervals. Initial and final BOD_5 concentrations of the wastewater were measured weekly. The plants were weighed at 2-4 week intervals, harvested, and restocked to 10 kg m^{-2}. Four plant samples were collected monthly from each tank for the determination of dry weight/wet weight ratios and average root length.

Experiment III

The effect of plant harvest on BOD_5 removal was evaluated in four, 3000 L (5.9 m^2 surface area, 0.5 m deep) tanks which received a semi-continuous flow (1.5 L min^{-1}; 15 min on:15 min off) of primary sewage effluent. Water hyacinths were stocked into the tanks at a standing crop of 15 kg m^{-2}. Standing crop changes were estimated semi-monthly by weighing the plants contained within floating (0.25 m^2) plastic mesh cages (three cages per tank). In one set of duplicate tanks, the plants were harvested back to 15 kg m^{-2} at each weighing. In the other set of tanks, plant harvesting was not conducted. Influent and effluent BOD_5 concentrations were measured twice and once weekly, respectively.

RESULTS

Experiment I

During the first week of the incubation, BOD_5 removal from primary sewage effluent in tanks containing water hyacinth was more rapid than that in the screened tanks without macrophytes (Fig. 1). However, after 14 days, 94% of the BOD_5 had been removed from the wastewater in both treatments. Wastewater O_2 concentrations were low during the study, averaging approximately 0.5 mg L^{-1} in tanks both with and without macrophytes.

BOD_5 removal from the tanks without macrophytes was enhanced slightly when no shading was employed. However, the BOD_5 removal rate in this treatment was still not equal to that of the tanks containing water hyacinth (Fig. 1). Wastewater O_2 concentrations in the water hyacinth treatment ranged from 0.2 to 0.8 mg L^{-1} during the incubation. In the tanks without macrophytes, O_2 concentrations varied markedly, averaging 0.3, 11.0 and 3.4 mg L^{-1} during weeks 1, 2, and 3 of the study.

Experiment II

The small tank FAMS containing pennywort were found to be more effective at removing BOD_5 from wastewater than those containing water hyacinth, with BOD_5 removal rates averaging 88% and 79%

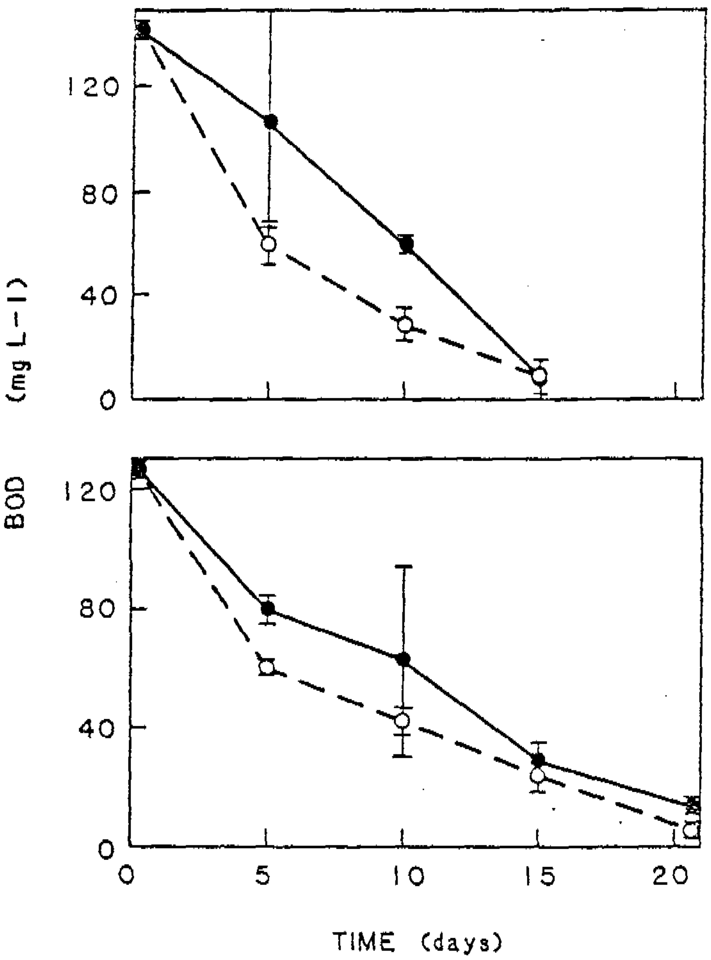

Fig. 1. BOD_5 removal ($\overline{x}$, $\pm$1 S.D.) from primary domestic effluent in tanks with (open circles) and without (closed circles) water hyacinth. Incubations in tanks without water hyacinth were conducted both with (top) and without (bottom) an opaque cover.

in the pennywort and water hyacinth treatments, respectively (Fig. 2). Influent BOD_5 concentration averaged 191 mg L^{-1} during the year, with effluent concentrations ranging from 3 to 38 mg L^{-1} in the pennywort systems and from 13 to 80 mg L^{-1} in the water hyacinth systems. On a mass basis, annual BOD_5 removal averaged 254 and 281 kg ha^{-1} day^{-1} in the water hyacinth and pennywort treatments, respectively. Wastewater BOD_5 removal was typically greater during the summer than during the winter for both species (Fig. 2).

The productivity of water hyacinth was greater than that of pennywort during all months except January and February (Fig. 2). Annual mean yields were 14.2 and 6.9 g dry wt m^{-2} day^{-1} for water hyacinth and pennywort, respectively. Water hyacinths had a more extensive root mass than pennywort in the present study: root length for water hyacinth ranged from 3.6 to 7.5 cm (mean of 5.6 cm), whereas the root length of pennywort varied from 2.1 to 2.9 cm (mean of 2.5 cm).

Experiment III

Semi-monthly water hyacinth harvesting had no effect on BOD_5 removal during this six month study (Fig. 3). The influent wastewater BOD_5 concentration averaged 180 mg L^{-1}, with effluent concentrations averaging 65 and 72 mg L^{-1} in non-harvested and harvested treatments, respectively. Mass BOD_5 removal for these treatments averaged 330 kg BOD_5 ha^{-1} day^{-1}, or approximately 60% of the influent loading.

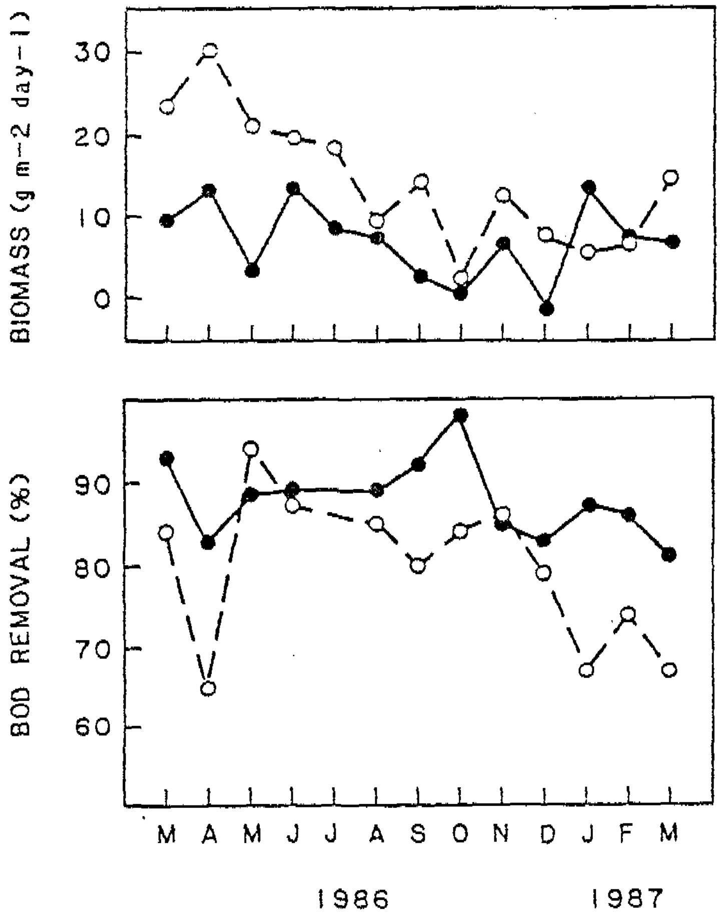

Fig. 2. Annual biomass production and BOD_5 removal from primary domestic effluent in tanks containing water hyacinth (open circles) and pennywort (closed circles).

In contrast with BOD_5 removal, plant harvest had a marked effect on water hyacinth biomass production. Average water hyacinth yield in the harvested treatment over the six month study was 13.3 g dry wt m^{-2} day^{-1}. In the non-harvested treatment, yields averaged only 6.3 g dry wt m^{-2} day^{-1}. The non-harvested plants grew from an initial stocking of 10 kg m^{-2} to a standing crop of 32 kg m^{-2} by mid-March (Fig. 4). In the harvested treatment, the water hyacinth standing crop fluctuated from between 15 and 21 kg m^{-2} for the six month period.

DISCUSSION

Stowell et al. (1981) suggested that FAMS function in a manner similar to a horizontal trickling filter, with submersed plant tissues acting as a support medium for bacteria. The data from Experiment I in the present study show that the presence of floating macrophytes does indeed enhance BOD_5 removal from primary sewage effluent. However, because the heterotrophic organisms which oxidize carbon in FAMS have not been characterized as to their type and location, the importance of floating macrophytes as a substrate remains speculative.

The role of floating macrophytes in transporting O_2 from the atmosphere to the water column is better documented. Moorhead and Reddy (1987) recently reported that from 100 to 400 mg O_2 m^{-2} hr^{-1} can be released from water hyacinth root tissues into the surrounding medium. Because the presence of floating macrophytes inhibits photosynthetic O_2 production by microalgae and also reduces direct atmosphere-water gas exchange, the transport of O_2 through the plants is probably crucial in supporting the oxidation of wastewater carbon by bacteria (Reddy, 1984; Weber and Tchobanoglous, 1986).

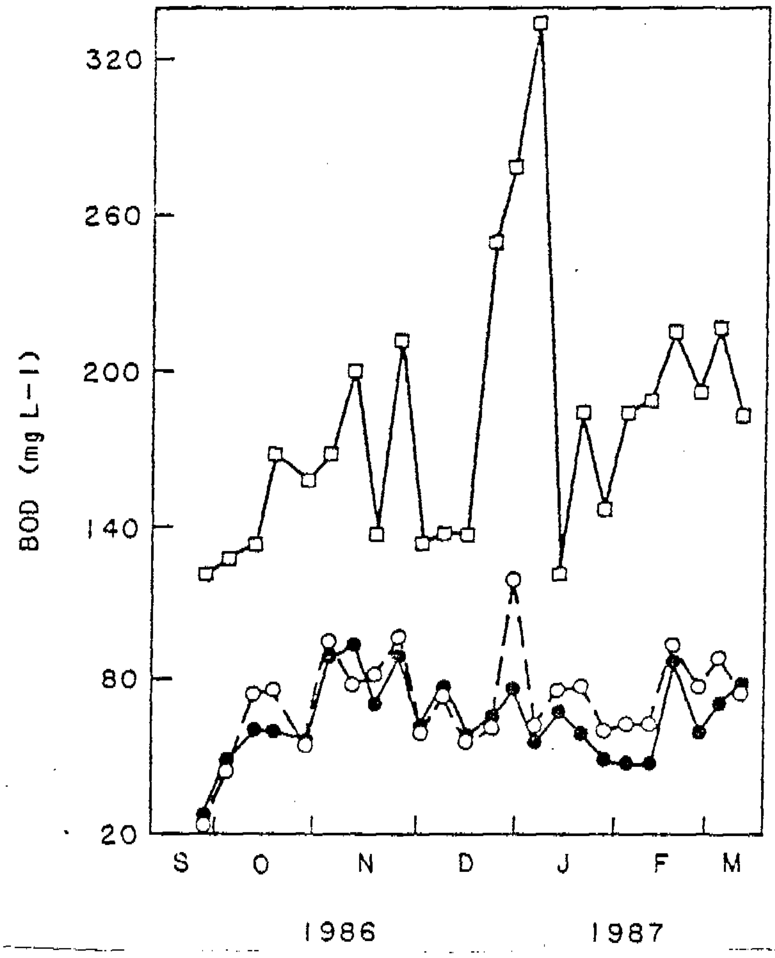

Fig. 3. Influent (open squares) and effluent BOD_5 concentrations of primary domestic effluent in harvested (open circles)- and non-harvested (closed circles) water hyacinth culture tanks.

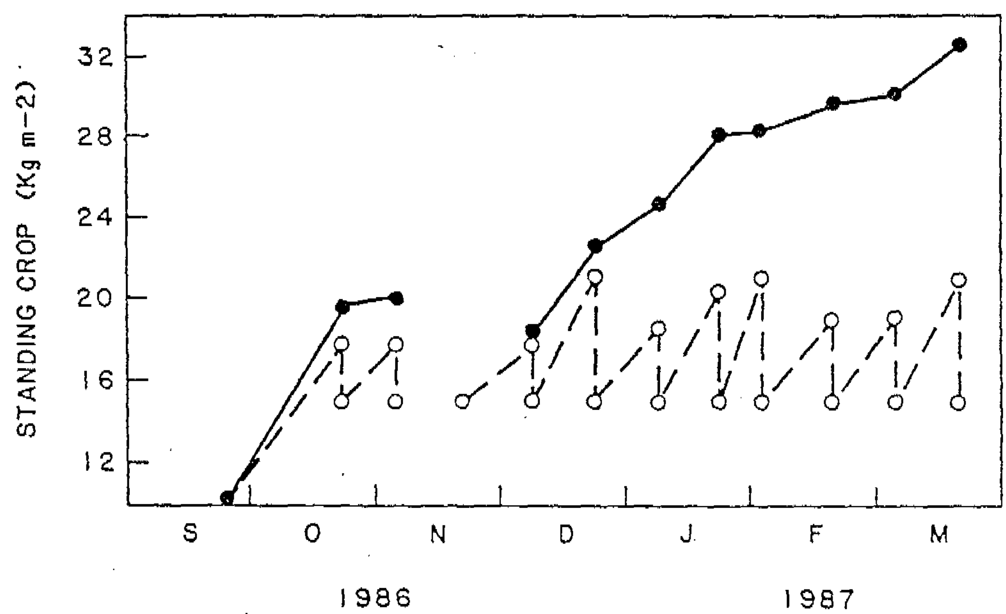

Fig. 4. Plant standing crop in harvested (open circles) and non-harvested (closed circles) water hyacinth culture tanks.

Floating macrophytes which possess a high root surface area provided with a continuous O_2 supply may actually be an "ideal" substrate for an aerobic bacterial film. Oxygen concentration data from Experiment I suggest that macrophyte-transported O_2 can be utilized by bacteria as quickly as it enters the rhizosphere, since high BOD_5 removal was attained with no observed increase in wastewater O_2 concentration.

Laboratory studies have shown that O_2 transport (per unit weight of root tissue) by the floating macrophyte pennywort occurs at a rate 2.8 times that of water hyacinth (Moorhead and Reddy, 1987). This higher O_2 transport rate probably accounts for the superior BOD_5 removal, and higher wastewater O_2 concentrations observed in Experiment II in the tanks containing pennywort.

In the present study, we found roots of pennywort to be shorter as well as less highly branched than those of water hyacinth. Thus, in stands of equal standing crop biomass, the total substrate available for bacterial attachment on pennywort is probably lower than that on water hyacinth. This difference in available substrate may explain why BOD_5 removal in pennywort systems exceeded that in water hyacinth systems by only 10%, even though the difference in O_2 transport rates is 3-fold.

Plant harvest is routinely practiced in most FAMS in order to maximize macrophyte growth and uptake of certain wastewater contaminants (Wolverton and McDonald, 1979; Hayes et al., 1987). Harvest may affect wastewater carbon removal by disrupting the substrate or by influencing O_2 transport. Young water hyacinth plants transport O_2 at a higher rate (per unit weight of root tissue) than older plants (Moorhead and Reddy, 1987), which suggests that O_2 transport in FAMS, and hence, BOD_5 removal, can be enhanced by maintaining a standing crop of young plants through frequent harvest. Although semi-monthly harvesting was found to stimulate water hyacinth biomass production in the present study, no effect on BOD_5 removal was observed. The plant yield and BOD_5 removal data from Experiments II and III indicate that macrophyte growth rate has no effect on BOD_5 removal.

In summary, FAMS containing pennywort were found to remove BOD_5 from primary domestic effluent at a higher rate than those containing water hyacinth. It is likely that there exist other floating species, particularly those with large, diffuse root mats and high O_2 transport capability, which are superior to pennywort in promoting wastewater BOD_5 removal. The poor relationship found between plant harvest and carbon removal affords considerable flexibility to the operation of FAMS used for secondary wastewater treatment. For example, frequent harvesting in such systems would be conducted only when the removal of P or heavy metals is required, or when the value of the macrophyte-based product is high.

ACKNOWLEDGMENTS

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REMOVAL OF HEAVY METAL AND SEWAGE SLUDGE USING THE MUD SNAILS, *CIPANGOPALUDINA CHINENSIS MALLEATA* REEVE, IN PADDY FIELDS AS ARTIFICIAL WETLANDS.

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ABSTRACT

The effects of the application of reed-sewage sludge compost on the heavy metal incorporation and the growth of young snails born from the adult mud snails, *Cipangopaludina chinensis malleata* REEVE, put into submerged paddy soil were investigated. The biomass and growth of the snails in paddy soil with a compost were superior to those in soil without a compost. The Zn and Cu concentrations in the flesh portion of snails were extremely high as compared with those in the paddy soil surrounding the snails.. This may be because snails ingest sewage sludge which is a main organic component of the composts and sewage sludge usually contains large amounts of Zn and Cu, suggesting that this type of snail may be useful in eliminating sewage sludge and Zn and Cu in paddy soil when composted sewage sludge has been applied.

KEYWORDS

Mud snail; *Cipangopaludina chinensis malleata* REEVE; heavy metal; Cd; Zn; Cu; paddy field; reed-sludge compost; *Phragmites australis*.

INTRODUCTION

Application of composted sludge to agricultural land as fertilizer is now becoming popular as a way of disposing of sewage sludge produced at sewage treatment plants. The authors have succeeded in increasing the yield of rice plants by applying a reed-sludge compost, which was made of sewage sludge and *Phragmites australis* (a principal plant in Japanese wetlands), to the paddy fields as an artificial wetland (Kurihara *et al.*, 1986). Application of the reed-sludge compost to agricultural land seems to be an important sewage treatment method from the viewpoints of biological treatment of sewage and crop production, because such an application means that inorganic nutrient substances in a sewage that flow in wetlands are uptaken into *Phragmites* (Wathugala *et al.*, 1985) and a sludge produced from municipal sewage are used in agricultural land as fertilizer. The problems in this case are the growth inhibition of crops due to the pollution of soil with heavy metals and toxic effects of the heavy metals on the human body via the crops. Paddy fields in Japan, on the other hand, are inhabited by a great amount of mud snails, *Cipangopaludina*. They had been relished by many Japanese as a protein source, since they did not damage rice plants and were quite edible. However, they have become almost extinct over the past fifteen years because of the introduction of agricultural machinery.

In this paper, some results will be presented on the biological treatment of sludge applied to paddy fields and biological elimination of heavy metals in paddy soil through the use of these mud snails.

MATERIALS AND METHODS

The field experiment was conducted on fine-textured gray lowland soil at Miyagi Prefecture in northeastern Japan. The soil had ca. 3.0% total C, ca. 0.3% total N, and pH 6.5-7.0. It had a clay fraction dominated by smectite and 33.5 me/100g of CEC. The paddy field used for the experiment (4 x 2 m^2) was divided into 2 equal portions, and one of them was applied with composted sewage sludge containing marsh reed, Phragmites australis, at the rate of 1 kg/m^2, which corresponds to the customary amount for application in Japan, on May 20th and both plots were planted with rice plants (Sasanishiki) at hill intervals of 27 cm on May 27th. Plots with and without compost thus prepared were manually weeded. Adult snails, Cipangopaludina chinensis malleata REEVE (shell width : 32±1 mm), were put into each prepared plot on June 4th at the density of 5 individuals per m^2. Each plot was enclosed with plastic net barriers permitting water flow but preventing the migration of the snails from one plot to another. All young snails born from these adult snails that were put in the paddy field were collected on July 29th and immediately had their maximum shell widths measured, and then they were returned to the place from which they were taken. Flesh portion on the dry weight basis was calculated from a formula for the relationship between shell width and the dry weight of the flesh portion, and shell weight was calculated from a ratio of shell to flesh portion on the dry weight basis.

To investigate the effect of population density on the maximum amount of sludge assimilated by the snails, the small-size group with a 12 - 13 mm shell width, medium-size of 16 - 18 mm and large-size of 27 - 32 mm, all of which had been starved for more than one week, were placed into each of 10 cm diameter vessels filled with water at the varied densities. As much sludge as the snails could consume was supplied and the food consumption and defecation during the 2-day period were measured at a constant temperature of 25°C. The assimilation was measured as the difference between dry weights of the sludge consumed and feces excreted. The sludge assimilation under the continuous feeding condition was also examined. Twenty medium-size snails (238 g D. W. of flesh portion per m^2) and 2 large-size ones (224 g D. W. of flesh portion per m^2), which had already been starved, were placed in the vessels with 10.5 cm diameter separately and the daily changes in the rate of assimilation were recorded under the daily supply of as much sludge as the snails could consume.

For measurement of heavy metal contents, all newborn snails were collected on September 25th, just prior to harvesting the rice plants. The flesh portion and the shell of snails dried at 70°C for 3 days was ground, digesting the powders of flesh portion and shell in aquaregia after pre-digesting in aquaregia using Uniseal, and analyzed for Cd, Zn, Cu, Ni, Pb, Cr, Mn and Fe by atomic absorption spectrophotometry. Heavy metal concentrations in the paddy field soils were also measured at this time.

RESULTS

1. Sludge Consumption By Snails.

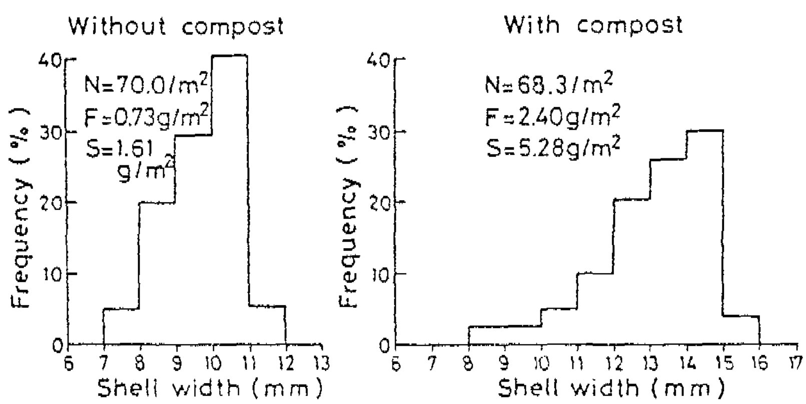

Fig.1. The size distributions of shell width of the newborn snails from plots with (right) and without (left) reed-sludge compost. N : the number of individuals, F : dry weight of the flesh portions, S : dry weight of the shells

The size distribution of shell width of the young snails collected from plots with and without compost is illustrated in Fig.1. The growth of snails in the plot with compost was superior to that in the plot without compost, suggesting that the snails feed on the sludge which are main organic component of the reed-sludge compost. However, very little difference in the numbers of individuals between both plots could be discerned.

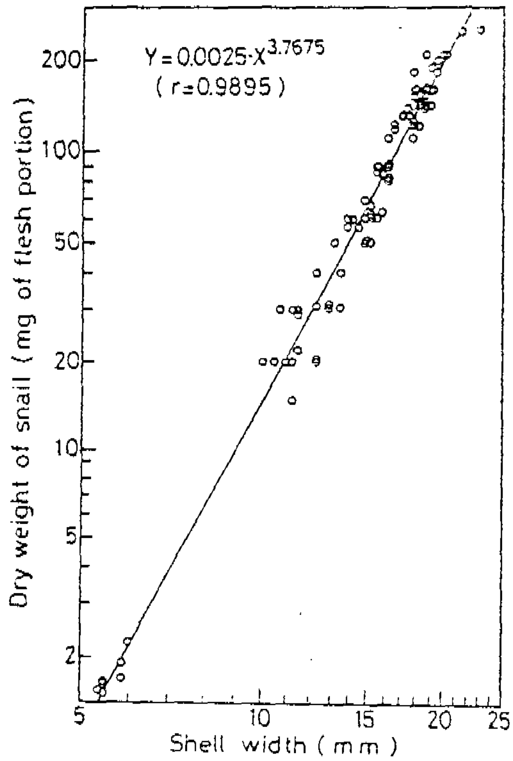

Fig.2. The relationship between the maximum shell width and the dry weight of the flesh portion of newborn snails.

Next, the relationship between the maximum shell width and the dry weight of the flesh portion were indicated as shown in Fig.2, where a high correlation is apparent when the log shell width is taken as abscissa and the log dry weight of the flesh portion as ordinate. The ratio of shell to flesh portion on the dry weight basis was measured as 2.2 ± 0.48 (n = 20).

TABLE 1. Rates Of Sludge Assimilations Of Snails Per Unit Weight Of Flesh Portion Under Continuous Feeding Conditions At Beginning And Latter Term Of Experimental Periods

	Shell width (Density)	Sludge assimilation rate* (mg/g dry wt./day) Beginning	After 15th day
Medium-size	15.5-17.9 mm (238 g/m^2)	9.98 ±1.40**	5.06 ± 1.02
Large-size	26.4-36.5 mm (224 g/m^2)	7.53 ± 1.24**	4.08 ± 0.97

* The values are means of 6 samples with their S.D.
** Correspond to the maximum amount assimilated.

The effect of population density on sludge assimilation regarding the three size groups is illustrated in Fig.3 (above). The amount of daily assimilation per unit body weight showed a sharp decline when the population density was low. However, the decline in the assimilation rate was retarded when density exceeded by 70 g per m^2. The rate of assimilation per unit area increased remarkably with the increase in population density and approached a maximum value when the population density increases to 70 g per m^2 for small-size snails, 100 g per m^2 for the medium-size ones and 130 g per m^2 for the larger ones. The maximum values of assimilation per m^2 per day were 3.6 g for small-size snails, 2.3 g for the medium-size ones and 1.5 g for the larger ones (Fig.3, below).

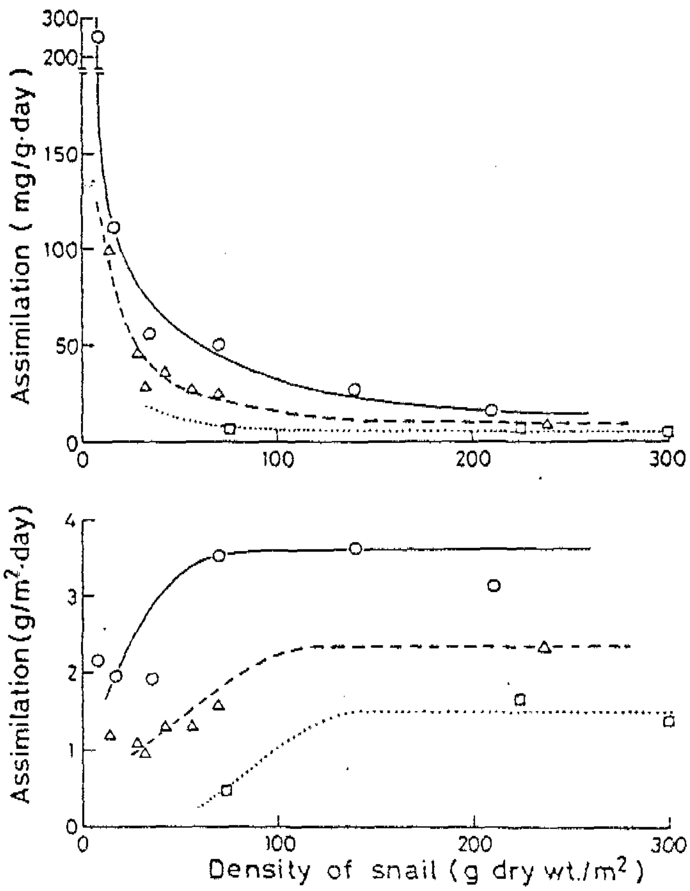

Fig.3. The effect of population density on the rate of sludge assimilation per unit weight of flesh portion of the snails on the dry basis (above) and per unit area (below)
○ : small-size group, △ : medium-size group,
□ : large-size group.

The daily changes in the amounts of sludge assimilated by medium-size and large-size groups under the daily supply of sludge were examined. The amount of sludge assimilated by both size groups decreased gradually with time at the earlier term and eventually it showed a steady value after a 15-day cultivation. Table 1 shows the amounts of assimilation per day at the beginning and the latter term of the experimental periods. It seemed from the table that the assimilation rate of both size groups under the continuous feeding at the latter term dropped to about 52% of the assimilation rate at the beginning.

2. Heavy Metal Concentrations In Snails.

Table 2 indicates the heavy metal concentrations in the flesh portion and the shell of newborn snails collected from the plots applied with compost on September 25th, together with the heavy metal concentration in the paddy soil. Zn and Cu concentrations were extremely high in the flesh portion of the snails as compared with the concentrations in the shell and the soil. Cd concentration in the shell was higher than that in the flesh portion and the soil, but for other heavy metals, the values in the flesh portion were much lower than those for the soil.

TABLE 2. Heavy Metal Concentrations In Paddy Soil, Flesh Portion And Shell Of Newborn Snails

	Heavy metal concentrations (μg/g dry weight)							
	Cd	Zn	Cu	Ni	Pb	Cr	Mn	Fe
Paddy soil	0.40	108	20.1	11.6	18.3	15.1	365	29700
Snail								
flesh portion	0.73	703	301	trace	trace	3.4	31.3	271
shell	3.48	99	7.5	-	-	-	-	-

- : not determined

From the values of biomass of snails (Fig.1) and the heavy metal concentrations (Table 2), the quantities of Cd, Zn and Cu accumulated in the newborn snails (including the flesh portions and the shells) per m^2 of plot applied with compost were calculated to be 20.12 μg for Cd, 2210 μg for Zn and 762 μg for Cu.

DISCUSSIONS

Of the results obtained from the present study, the most important is that the Cd, Zn and Cu concentrations, especially the latter two, in the flesh portion of the snails were exceedingly high as compared with the concentrations in the paddy soil surrounding the snails. The high Zn and Cu concentrations in the flesh portion of young snails is assumed to be the result of the selective intake of sewage sludge by the snails, causing an accumulation of Zn and Cu originated from the sludge. In fact, the concentration of Zn and Cu in sewage sludge is known to be particularly high, for example 681 mg/Kg for Zn and 232 mg/Kg for Cu in the sewage sludge produced from domestic sewage treatment plants in Japan (Mori, 1986). These results indicate that snails living in paddy field where composted sewage sludge has been applied are unfit as human food.

As indicated in the results, the amount of sewage sludge assimilated and the amounts of Cd, Zn and Cu accumulated in the newborn snail population showed a considerably high value. Therefore, if large quantities of snails are introduced into the paddy soil, it can be expected that a considerable portion of the heavy metals together with sewage sludge can be eliminated from the paddy field ecosystem by the harvesting of snails.

According to the results in Fig.3 (below), the maximum daily amount of sludge (on the dry basis) to be assimilated by snails at the density of over 70 g per m^2 for small-size snails, 100 g per m^2 for the medium-size ones and 130 g per m^2 for the larger ones were 3.6 g, 2.3 g and 1.5 g per m^2, respectively. Since the assimilation rate under the continuous feeding condition dropped to 52 % of the maximum (Table 1), the amount of sludge to be eliminated by the snails during the period from June to September would amount to 225 g per m^2 for small-size snails, 144 g per m^2 for the medium-size ones and 94 g per m^2 for the larger ones.

Since 1 kg of the reed-sludge compost applied to 1 m^2 of paddy field contains 120 g of sludge on the dry basis (Kurihara et al., 1986) and it includes 0.3 mg of Cd, 82 mg of Zn and 28 mg of Cu, the biomass of snail (small- or medium-size) which correspond to 100 g of the flesh portion per m^2 could exhaust the amount of the sludge applied, and the heavy metals accumulated in the flesh portion and the shell would be caluculated to be 0.84 mg for Cd, 92 mg for Zn and 32 mg for Cu (from Table 2) which are almost equivalent to the amount in the sludge applied, based on the assumption that the amount of heavy metals accumulated in the snails is in proportion to the biomass of the snail. Although these values were derived from those obtained from the feeding experiment, it is suggested that large amounts of sludge and heavy metals produced from the sewage treatment plants can be eliminated by the snails. Since migratory bird, egret which do not damage rice plant, coming over the paddy field in summer and autumn sometimes consume a large amounts of these snails, it is expected that the harvesting by egrets would make human harvesting not necessary. It is, therefore, hoped that the mud snail, Cipangopaludina, can act as a link in the recycling system between urban and agricultural activities

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THEME 8

REUSE

DEEP WASTEWATER RESERVOIRS IN ISRAEL:
(1) LIMNOLOGICAL CHANGES FOLLOWING SELF-PURIFICATION PROCESS

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ABSTRACT

Ma'ale Kishon, one of the largest reservoirs storing wastewater for irrigation in Israel, has surface area of 1.3 km^3, depth of 9 m and volume of 12 million m^3. Secondary effluent enters the northern basin and after a mean retention period of 6 weeks, flows into the southern one. Due to this two basins structure, changes taking place during the water storage can be detected. The differences between the two basins were quantitatively studied by an interdisciplinary team.

The sanitary, physico-chemical and biological variables indicated an improvement in water quality, leading to an increased ecosystem stability and a safer environmental performance. These results suggest, that deep reservoirs of the type here described, serving for wastewater disposal and reuse may have a wide application also in other countries having warm, subtropical climate.

KEYWORDS

Wastewater reservoirs; limnology; hypertrophic; selfpurification; wastewater reuse for irrigation.

INTRODUCTION

Storage of wastewater in deep reservoirs is widely used in Israel. Secondary effluents are collected during the winter and spring and reused for irrigation of industrial crops in the summer.

Ma'ale Kishon, one of the largest reservoirs in this country, has a surface area of 1.3 km^3, an average depth of 9 m and a volume of 12 million m^3. The reservoir is divided into two basins, northern and southern. Effluent enters the northern basin and after a retention period of approximately 6 weeks, flows into the southern one.

An interdisciplinary team investigated the reservoir in detail. Part of these studies dealing with the comparative limnology of the two basins is the objective of the present paper.

METHODS

Research trips were held monthly or biweekly, during the period April 1985 - July 1986. Measurements and sampling were done from a floating raft anchored in the middle of each basin. Profiles of dissolved oxygen (YSI Model 58), pH (Radiometer Model 29), light penetration (Lambda Li-Cor 185 Quantum Meter) and primary productivity (light and dark bottles, oxygen method) were measured on site between 9 and 13 AM. Water for laboratory analyses and primary productivity experiments was sampled by a transparent Wildco Alpha Bottle (Model 1120) and filtered to remove most of the filter feeders. Duplicate light and dark bottles were exposed for 1-4 hours, according to season, at the surface, 0.3 and 1.0 m depth. BOD, COD, TSS, NH_4^+, NO_3^-, $PO_4^=$, SPC and coliform bacteria were determined according to Standard Methods (1980). Algal cells were counted using hemocytometer and chlorophyll-a was determined after extraction with hot methanol as described in Eren et al. (1986). Zooplankton was collected by the same Wildco Alpha Bottle (a transparent plastic tube of 2.3 l capacity) and filtered through a 60 µm net. These samples were first preserved in 4% formaldehyde and after a few days transfered to 70% alcohol.

In order to summarize the differences between the profile measurements in the two basins, the values of DO, pH, chlorophyll-a, gross primary productivity and respiration in each basin were averaged over the whole investigation period (13 visits) - for each depth separately.

Light penetration was expressed as compensation depth, which is accepted as an approximate limit of euphotic zone, where net oxygen is produced by algae.

RESULTS

Part of the variables characterizing both basins of the reservoir is summarized in Table 1. The reduction of organic matter content (BOD, COD), suspensions (TSS) and mineral nutrients (NH_4^+, NO_3^-, $PO_4^=$) was connected with a number of changes. Concentrations of heterotrophic bacteria (SPC) and algae decreased in the southern basin by 50 and 40% respectively. The counts of the herbivorous rotifers and daphnids decreased by 75 and 44% respectively and on the contrary, the counts of the omnivorous-carnivorous cyclopoid species increased in the southern basin by 154%. Water transparency, expressed as compensation depth, improved by 37%. In parallel, the numbers of coliforms decreased by 96%.

Averaged profile data of DO, pH, chlorophyll, gross primary productivity and respiration are given in Figs. 1-5.

Mean content of dissolved oxygen in the upper meter of the reservoir fluctuated between 7 and 8 mg l^{-1} in the southern basin, and between 4 and 5 mg l^{-1} in the northern basin. Integral of the area under the curve (expressing mean amount of DO in the water column found during a single measurement at early midday) - was by 35% higher in the southern basin (Fig. 1). Mean pH (Fig. 2) was in all depths slightly higher in the southern basin. Chlorophyll-a content is presented in Fig. 3. Mean surface concentrations of 60-70 µg l^{-1} were similar in the two basins but below the depth of 1 m, the southern basin had less chlorophyll-a than the northern. Integral of the area under the curve (expressing mean amount of chlorophyll-a in a 5 m thick water layer during a single measurement), was by 29% lower in the southern basin. Gross primary productivity (GPP) and respiration (R) in both basins are compared in Figs. 4 and 5. Mean GPP and R calculated for the upper, 1 m thick water layer (9^{00}-13^{00} AM), were lower in the southern basin by 31 and 48% respectively.

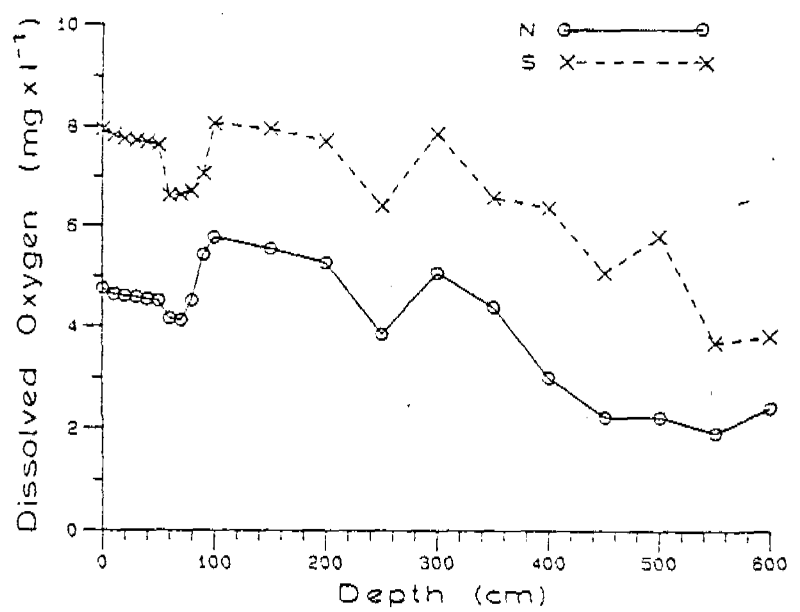

Fig. 1. Mean concentrations of dissolved oxygen (early midday) as a function of the depth. N - northern basin; S - southern basin.

TABLE 1 Data on the Two Basins of Ma'ale Kishon Reservoir (April 1985 - July 1986)

Variable		Northern Basin			Southern Basin			% Change
		No. of Samples	Average	SD	No. of Samples	Average	SD	
BOD	mg l^{-1}	14	15.0	14.2	13	5.5	2.9	- 63
COD	mg l^{-1}	17	68.0	16.7	15	53.0	11.0	- 22
TSS	mg l^{-1}	15	18.0	19.5	14	13.0	11.1	- 28
NH_4^+	mg l^{-1}	9	46.6	6.3	9	30.0	5.6	- 36
NO_3^-	mg l^{-1}	9	0.5	-	9	3.0	-	+ 83
$PO_4^=$	mg l^{-1}	8	22.4	5.0	8	16.8	4.5	- 25
Coliforms,* cells	ml^{-1}	10	10,000		14	400.0		- 96
SPC** cells	ml^{-1}	13	$2.2x10^7$	$2.1x10^7$	13	$1.1x10^7$	$1.3x10^7$	- 50
Algae** cells	ml^{-1}	13	$2.5x10^4$	$2.4x10^4$	13	$1.5x10^4$	$2.2x10^4$	- 40
Zooplankton,*** specimens	l^{-1}:							
Rotifers		70	155.2	470.6	71	39.0	162.8	- 75
Daphnids		70	36.6	62.7	71	20.5	59.1	- 44
Cyclopoids		70	6.2	16.6	71	15.8	41.4	+154
Compensation Depth	cm	10	233	90	10	356	94	+ 35

* Geometric mean.
** 13 visits mean, calculated for the upper, one meter thick water layer.
*** 13 visits mean, calculated for the whole water column.

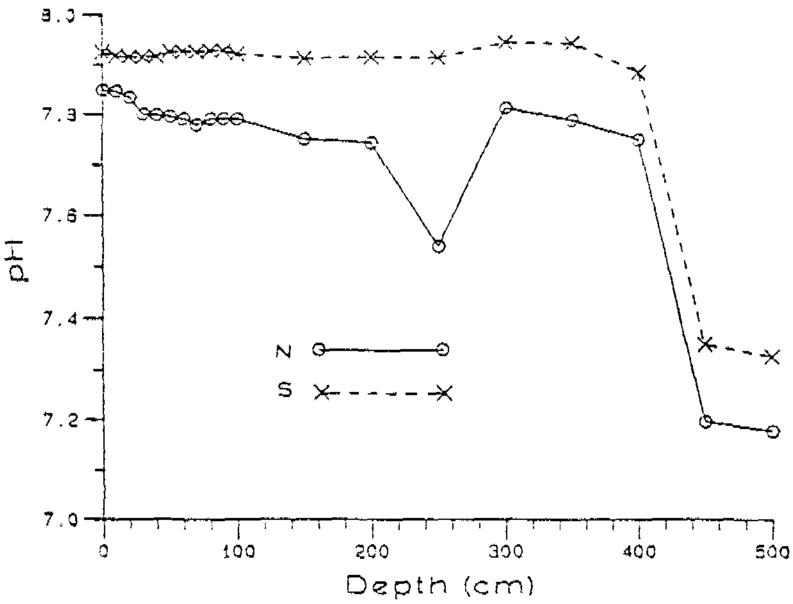

Fig. 2. Mean pH as a function of the depth. Abreviations as above.

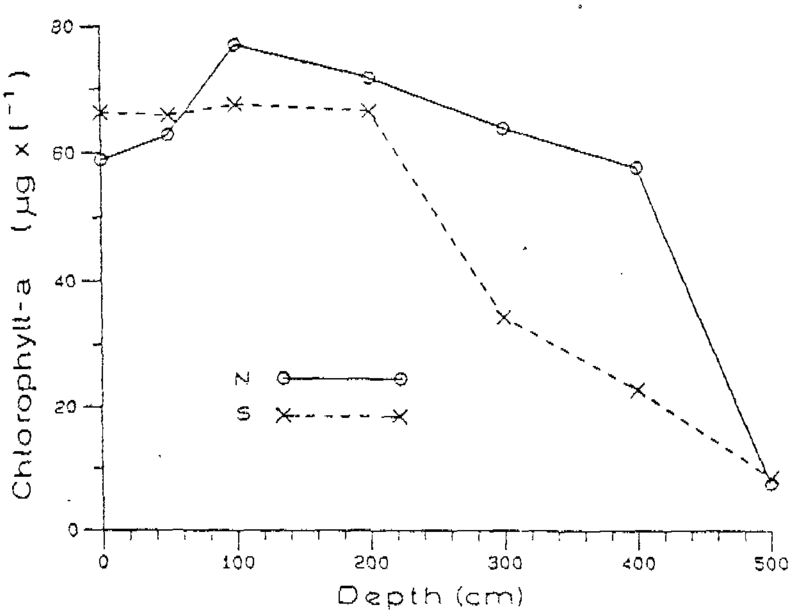

Fig. 3. Mean concentrations of chlorophyll-a as a function of the depth. Abreviations as above.

DISCUSSION

Wastewater reservoir of the type here described belongs to the category of hypertrophic water bodies, exhibiting high concentration of biomass, high rate of metabolic activities and wide fluctuations in the biological and physico-chemical variables (Barica and Mur, 1980). In such conditions the progressive limnological changes connected with the self-purification process are masked and difficult to demonstrate.

The particular structure of Ma'ale Kishon Reservoir, having two basins operated in chain, provided a unique opportunity to determine water quality improvement within the system and to relate it with the respective changes in the limnological variables. Moroever, the averaging of profile measurements over a long time intervall, allowed an integrated comparison of the two basins.

The self-purification process expressed in reduction of organic matter, suspensions and mineral nutrients, resulted in an increased transparency of the water column, higher mean content of oxygen, decreased concentration of bacteria, algae and zooplankton as well as decreased biological productivity and respiration. The above changes conform with the definition of an increased ecosystem stability (Odum, 1971) and improve the environmental performance of a reservoir.

Our present results and former experience (Dor, 1986; Eren et al., 1986; Dor et al., in press) suggest, that deep reservoirs serving for the wastewater disposal and reuse, may be successfully operated in warm, subtropical climates. However, applicability of the here provided quantitative evaluations to the other systems of this kind - still has to be tested.

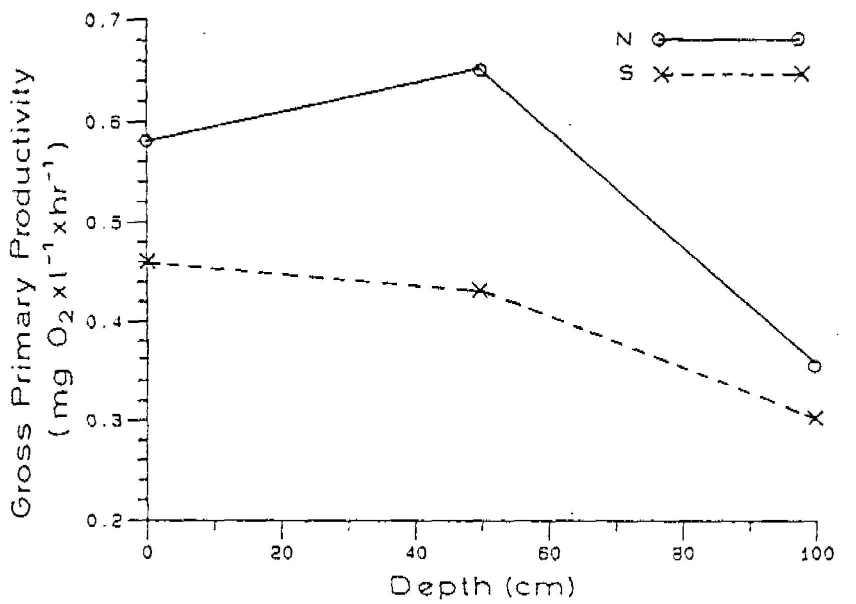

Fig. 4. Mean rate of gross primary productivity as a function of the depth. Abreviations as above.

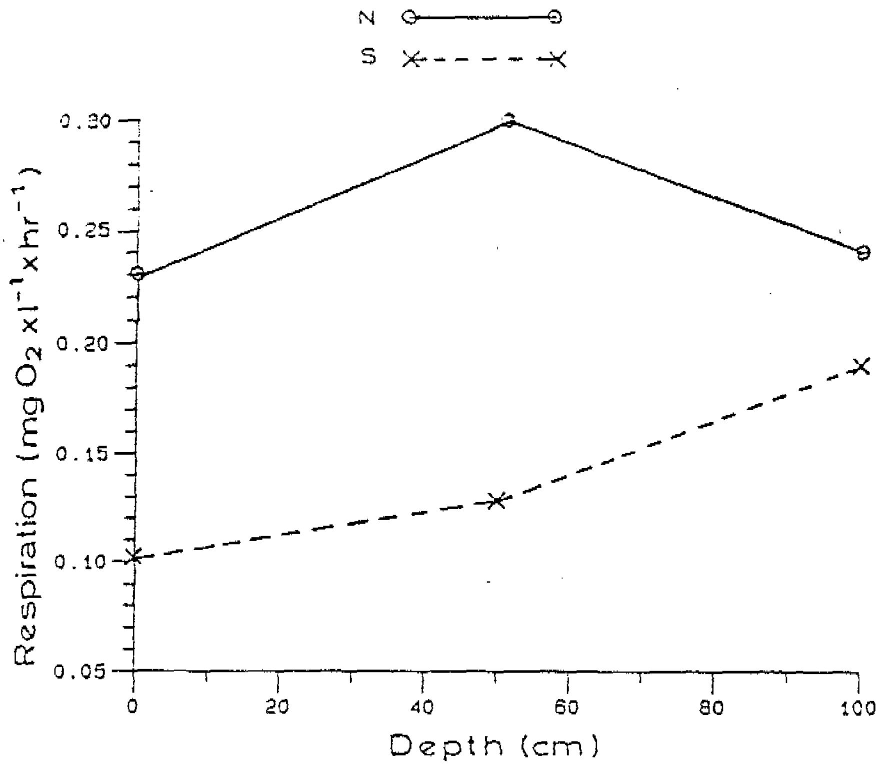

Fig. 5. Mean rate of respiration as a function of the depth. Abreviations as above.

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THE USE OF POLYCULTURE SYSTEM (ALGAE - MICROCRUSTACEANS) FOR DOMESTIC WASTE TREATMENT

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ABSTRACT

Domestic sewage and some types of agroindustrial waste, although representing high pollution sources, could become important energetic and protein sources because they are wastes deriving from foods which still contain significant amount of organic matter and nutrients used by human beings.

Based on this concept, different scale experiments (laboratory, semi-pilot and pilot scale) have been carried out to verify the viability of the system which composed anaerobic pond, algae pond and zooplankton pond for the treatment of domestic sewage and at the same time to reuse zooplankton as animal food. Scenedesmus sp and Daphnia sp were introduced into the algae and zooplankton pond, respectively, to be maintained as predominant biological element.

The best operational performance was obtained when the zooplankton pond was operated with semicontinuous flow system, renewing 50% of the zooplankton pond water with effluent of algae pond every 2 or 3 days.

The productivity of Daphnia sp in this operational condition reached around 35 mg/ℓ in dry weight. It corresponds to 700 kg of microcrustaceans production per day for a city with 100000 inhabitants.

BOD removal efficiency obtained during the experiment was 95-97% and total N and total P removals were 42-59% and 37-48%, respectively. The final effluent showed high transparency. In the zooplankton pond the algae concentration was reduced from 10^6 cells/mℓ to 10^2 cells/mℓ which corresponds to 99,99% of removal efficiency.

KEYWORDS

Stabilization pond; polyculture; aquaculture; domestic sewage treatment; algae culture; Daphnia culture.

INTRODUCTION

The domestic wastewater and some type of agroindustrial wastes, although they are not significative pollutants, may become an important energetic and protein source, since they result from food which contains high amounts of organic matter and nutrients used by living beings.

Based on this concept, CETESB has been developing a series of experiments in the last ten years aiming at purifying wastewater by recycling the pollutants and transforming them into useful products.

From 1977 to 1980, pilot scale experiments were carried out in accelerated photosynthetic ponds with the view of treating domestic wastes and making use of algal protein (Kawai, Jureidini and Grieco, 1984). This study showed that, although the final quality of the effluent indicated a highly satisfactory quality in terms of terciary treatment, it required relatively sophisticated equipment for the present Brazilian economical situation.

As an alternative to solve this problem, the use of the polyculture system was taken into account, based on acquatic food chain in which fitoplankton and zoopkankton participate.

Several papers have been internationally published on this subject, however, in most cases, in laboratory scale (Tarifeño-Silva et al., 1982; Gordon et al., 1982).

The technical information to be presented in this paper refers to the experiments carried out in semi-pilot and pilot scale from 1984 to 1986.

EXPERIMENTAL PROCEDURES

Before the experimental phase in semi-pilot scale, a series of "in vitro" experiments was conducted in batch and continuous flow aiming at identifying the biological behaviour of the species for establishing the operational basic parameters of the pilot system.

At the beginning, the possibility of using the rotifer Brachionus sp and of the clodocera Daphnia sp was raised due to their high ability of reproduction and filtration, besides there was much information available. The laboratory assay has proved to work better with Daphnia similis on account of the populational stability and the purifying ability which it presents in the wastewater.

Semi-Pilot Scale Laboratory

An experimental unit, the flow sheet of which is shown in Figure 1, has been installed at CETESB, in São Paulo.

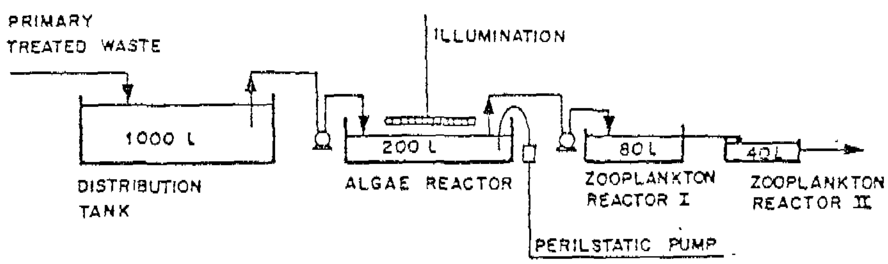


Fig. 1. Flowsheet of semi-pilot scale experimental unit

The domestic waste treated at primary level at SABESP - São Paulo State Basic Sanitation Agency's Waste Treatment Plant flowed through the distribution tank, the algae reactor and next through the zooplankton reactors I and II by means of a perilstatic pump. Commom fluorescent lamps were installed to keep a constant illumination of 5 Klux on the surface of the algae reactor and a slow circulation of water through air bubbling was kept constant. The liquid volumes in this reactors were kept at 200 ℓ, 80 ℓ and 40 ℓ and the detention times were 5, 4 and 2 days, respectively. Chlorella sp and Daphnia similis cultures were used in amounts compatible with the reactors.

Pilot Experiment

The pilot scale experiment was carried in SANASA. Initially, preliminary experiments were made in a system made up by anaerobic, facultative and zooplankton ponds, operated in series, by maintaining detention times of 3, 7 and 5 days, respectively. During six months of operation (from November 1984 to April 1985), there were some operational problems due to some instability in the behavior of the facultative pond interfering negatively in the productivity of Daphnia sp.

On account of this, an experimental system was implemented according to the flow-sheet below (Figure 2):

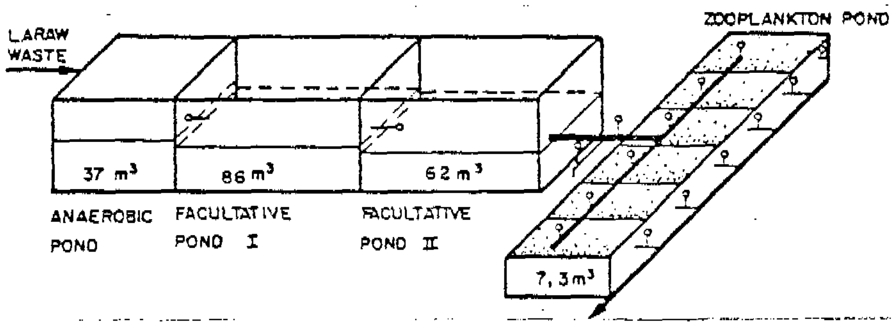


Fig. 2. Flowsheet of pilot scale polyculture system

All units were built in masonry. Some of the wastes, mostly domestic, which got to the plant was introduced into the system after grating. The connections between the units were made of PVC pipes equipped with valves which kept a difference in level of 20 to 50 cm between valves. A slow circulation of water was kept in the facultative ponds I and II, by using hydraulic pumps.

During the experiment; the first three units, corresponding to the anaerobic, facultative ponds I and II were always operated in continuous flow with a detention time of 3, 7 and 5 days, respectively. The zooplankton ponds were operated in the following way:
Pond 1 - daily replacement of 30% of its volume by effluent from facultative pond II; Pond 2 - replacement of 50% of its volume by effluent from facultative pond II every other day; Pond 3 - the same every three days; Pond 4 - the same every four days; Pond 5 - the same every five days; Pond 6 - the same every six days. This periodical replacement corresponded to the detention time of 3, 4, 6, 8, 10 and 12 days.

A mixed culture of Scenedemus sp and Chlorella sp was inoculated in facultative ponds I and II. In the zooplankton ponds Daphnia similis was seeded (it was offered by fish pisciculturist Mr. Sainen, Mogi das Cruzes, São Paulo).

The sample collection of the influent and effluent from the first three units were carried out on a 24 hours basis every 10 days and instantaneous sample collections at the outlet of the zooplankton ponds, after a slow mixing of the medium.

RESULTS

Semi-Pilot Scale

According to the data shown in Table 1, we can see that the BOD and the total coliform bacteria were reduced, in the average, 95 and 99,9% respectively. The algae in the zooplankton reactor I were reduced, in the average, from 2×10^8 org/mℓ to 8×10^3 org/mℓ, which corresponds to a removal efficiency of 99,99%. The population of microcrustaceans, in the average, was around 15 org/mℓ and dry weight at 0,02 mg/mℓ, enough to keep a high transparency of the water during most of the experimental period.

TABLE 1 Analytical Results of Operational Parameters Obtained by Semi-Pilot Scale Experiment.

REACTOR / PARAMETERS	ALGAE REACTOR INFLUENT	ALGAE REACTOR EFFLUENT	ZOO REACTOR I EFFLUENT	ZOO REACTOR II EFFLUENT	TOTAL EFFICIENCY (%)
TEMPERATURE (°C)	19,5				
BOD (mg/l)	125	29	12	7	94,0
COLIF. FECAL (NMP/100 ml)	10^7	10^6	10^4	10^4	99,9
ALGAE CONC. (org./ml)	-	2×10^8	8×10^3	-	99,9
ZOOPLANKT. CONC. (org / ml)	-	-	15	-	-

According to the obtained values, we can estimate that each Daphnia removes around 2×10^3 algal cells a day, in addition to the bacteria, fungi, protozoa and organic particles naturally present in the waste as a result of their predatory habits.

It is important to notice that 50 days after the beginning of this experiment there was a Brachionus sp contamination from the wastes which temporarily damaged the operation of the experimental system. A further investigation showed the anaerobic conditions of the wastes for some days, most of these rotiferous which cannot survive in the absence of dissolved oxígen are destroyed.

Pilot Scale

Table 2 shows average BOD removal efficiencies (95 to 97%); COD (79 to 88%); Total N (42 to 59%) and Total P (37 to 48%) that have been obtained during the experimental operation.

In general, from two to three days after the introduction of the effluent from the facultative pond into the zooplankton ponds, a high proliferation of Daphnia was detected; this increased water transparency considerably.

Near the outlets of the ponds the densities of these organisms ranged, depending on the pond, from 0.2 to 5 org/mℓ and the algae from the facultative pond were reduced from 10^6 cells/mℓ to around 10^2 cells/mℓ. In terms of dry weight, the productivity of zooplankton was around 20 to 43 mg/ℓ.

Talking into account the little variation in the removal efficiency of pollutants in the zooplankton ponds, added to the zooplankton reproductivity potential, we can suppose that the operation on 50% of the volume renewal basis at every 2 or 3 days would work satisfactorily.

Considering a zooplankton average production around 35 mg/ℓ, in dry weight, this would correspond to around 7 g of microcrustaceans in terms capita/day contribution, thus, for a 100 thousand inhabitant town, the daily production of these microorganisms would theoretically reach 700 kg.

At the end of May a gradual reduction of the Daphnia population in the zooplankton ponds was detected. This was due to the beginning of winter. It came near extintion between July and the middle of September, when springtime starts. Since then a slow recovery of this population has been observed.

TABLE 2 Treatment Efficiency of Polyculture System

PARAMETER / POND	RAW SEWAGE	ANAER. POND EFFL.	1st FAC. POND EFFL.	2nd FAC. POND EFFL.	ZOOPLANKTON PONDS					
					1	2	3	3	5	6
BOD (mg/ℓ)	284.0	118.0	61.6	32.5	15.0	14.5	13.2	14.5	8.2	12.9
REMOV.EFFIC. (%)					95	95	95	95	97	95
COD (mg/ℓ)	534.0	234.0	196.3	125.4	114.4	91.3	118.3	89.8	80.4	93.8
REMOV.EFFIC. (%)					79	83	88	83	85	82
N. KJELDHAL (mg/ℓ)	15.77	12.35	8.03	6.41	8.88	7.63	7.64	8.38	7.25	6.53
REMOV.EFFIC. (%)					44	42	42	47	55	59
N. NO_2 (mg/ℓ)	0.00	0.002	0.017	0.073	0.39	0.21	0.62	0.29	0.35	0.36
N. NO_3 (mg/ℓ)	0.00	0.00	0.00	0.078	0.05	0.05	0.07	0.05	0.10	0.07
TOTAL P (mg/ℓ)	4.45	4.79	3.98	3.40	2.73	2.60	2.65	2.80	2.30	2.50
REMOV.EFFIC. (%)					39	42	40	37	48	44
SESTON (mg/ℓ)	-	-	-	-	20.6	43.1	29.3	27.2	30.6	40.6
pH	7.0	7.0	7.4	7.5	7.9	8.1	8.2	8.0	8.0	8.0
DETENTION TIME (day)	-	3	7	5	3	4	6	8	10	12

During the period of the experimental operation, no significative contaminations by predatory organisms, e.g. Brachionus sp have been detected.

DISCUSSION

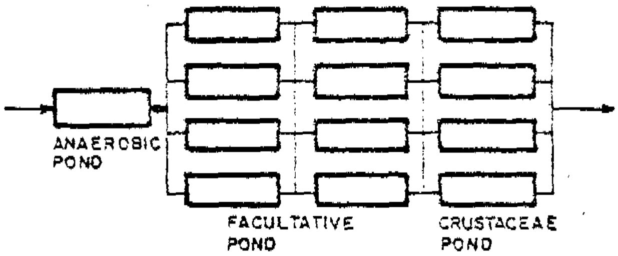
Based on the experiments carried out in the present study as well as on data collected in the literature, it is possible to state that the good performance of the zooplankton pond depends considerably on operational conditions of the facultative pond.

Small scale experimental tanks, as those used in this experiment, do not satisfactorily represent the full scale due to the limitations of the hydraulic factors, associated with the climatic conditions, which has hindered the evaluation of performance of the biological processes under study.

In the State of São Paulo, the ponds are the most used system of treating domestic wastes (there are around 100 ponds operating at present). According to the data obtained along several years, we could see that the facultative pond generally presents higher operational stability when it is operated with moderate loads (CETESB, 1979; Kawai et al., 1981).

Based on this practical information and assuming a detention time of five days for the zooplankton pond, it is possible to admit a polyculture system with a total detention time of approximately 30 days, out of which 5 days in the anaerobic pond, 20 days for the facultative pond and 5 days for the zooplankton pond. Notwithstanding the problem of building cost, the adoption of multicellular pond system as indicated in the following diagram, would provide more operational stability and management flexibility.

MULTICELLULAR SYSTEM



Out of the factors which limit the use of the polyculture system, we can point out the following as the most critical ones:

a) Reduction of Daphnia reproduction rate during winter (June, July and August). Although the average temperature in the coldest month ranges from 15 to 17°C, sudden drops often occur, causing sharp reduction in the reproduction rate of this kind of organisms, in the State of São Paulo;

b) Daphnia contamination of the facultative pond. During the operational period the passage of these organisms from one pond to another through animals and handling is almost inevitable. In this circumstance, the facultative pond would be temporarily transformed into another zooplankton pond, working favorably from Cladocera production point of view, yet it would be inoperative in terms of treatability in joint systems.

CONCLUSION

The experiments carried out in the present study suggest that the use of polyculture system as one of the alternatives for treating wastes in ponds still depends on future investigations. These investigations should be developed in full scale for a long period of time which will make possible to solve operational problems and to determine design parameters (including microcrustacean collection process and commercialization) for the practical application of the system under discussion.

ACKNOWLEDGEMENT

We thank SANASA for allowing us to use Cambuí Treatment Plant in order to carry out this experiment.

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IRRIGATION REUSE OF POND EFFLUENTS IN DEVELOPING COUNTRIES

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ABSTRACT

The large-scale reuse of sewage for irrigation, often without adequate safeguards, is commonplace in many arid and semiarid regions of the world. A UNDP/World Bank global research project has reviewed available epidemiological data and formulated a risk model to evaluate sanitary control options for effluent irrigation. The study concluded that wastewater treatment processes that effectively remove all or most of the pathogens in wastewater provide a major or total reduction in the negative health effects caused by raw wastewater reuse. Furthermore, the study found the recommended criteria for effective wastewater treatment for irrigation reuse in developing countries to be, in order of priority: (1) maximum removal of helminths; (2) effective reduction in bacterial and viral pathogens; and (3) freedom from odor and appearance nuisances (i.e., reduction of BOD). Multicell stabilization ponds are suited to meet all three criteria. Research sponsored by the UNDP/World Bank project has shown that well-designed and operated multicell stabilization ponds achieve virtually total removal of helminths and a greater than 99.99 percent reduction of enteric bacteria. Waste stabilization ponds can produce an odor-free effluent rich in nutrients and attractive for agricultural use. Most suitable in hot developing countries, ponds are a particularly robust, flexible, and almost fail-safe treatment system having low construction and operation costs. Research is now focusing on management and policy issues required to effectively achieve controlled irrigation reuse.

KEYWORDS

Wastewater treatment; waste stabilization ponds; reuse; irrigation; water quality guidelines; pathogen removal.

REUSE OF WASTEWATER IN AGRICULTURE

The large-scale reuse of domestic sewage for irrigation is commonplace in many arid and semiarid regions of the world as a result of increasing population pressures, water shortages, and agricultural demand. Twenty million mu (1.33 million ha) are irrigated with municipal wastewater in China alone (Wang, 1984). In the United States more than 3,400 reuse sites have been identified (Jewell and Seabrook, 1979). The data summarized in Table 1 show the extent of effluent irrigation in both developing and developed countries. These data indicate that sizeable agricultural activity based on wastewater reuse can be developed around major metropolitan areas. The relative importance of wastewater as a source of irrigation water in arid regions is illustrated by the data in Table 2, which gives sewage as a percentage of total irrigation water. These data also illustrate the extent to which land application can serve as a wastewater disposal option.

Although wastewater reuse in agriculture is commonplace, there are some crucial distinctions in how it is implemented. In the industrialized countries, reuse is planned, strict water quality and treatment standards are observed, and restrictions are made on crops allowed to be grown. Countries practicing controlled irrigation reuse are Germany, Israel, and the United States (especially the states of Arizona and California). In the developing countries,

TABLE 1 Data on Effluent Irrigation World-wide

Country and City	Irrigated Area (ha)
Argentina, Mendoza	5,700
Australia, Melbourne	10,000
Bahrain, Tubli	800
Chile, Santiago	16,000
China, all cities	1,330,000
Germany, Braunschweig	3,000
other cities	25,000
India, Calcutta	12,500
all cities	73,000
Israel, several cities	8,800
Kuwait, several cities	12,000 *
Mexico, Mexico City	90,000
all cities	250,000 *
Peru, Lima	6,800 *
Saudi Arabia, Riyadh	2,850
South Africa, Johannesburg	1,800
Sudan, Khartoum	2,800
Tunisia, Tunis	4,450 *
other cities	2,900 *
United States, Chandler, Arizona	2,800
Bakersfield, Calif.	2,250
Fresno, Calif.	1,625
Santa Rosa, Calif.	1,600
Lubbock, Texas	3,000
Muskegon, Michigan	2,200

* includes planned expansion of existing reuse

(Sources: Ayers and Wescott, 1985; Bartone, 1985; Cowan and Johnson, 1985; George et al., 1985; Kayser, 1985; Kalthem and Jamaan, 1985; Shende, 1985; Shuval et al., 1986; Wang, 1984)

TABLE 2 Data on Significance of Effluent Irrigation in Several Countries

Country (City)	Volume reused (MCM/Yr)	% of total sewage	% of total irrigation
Australia	149	11	-
China	10,000	27	-
Chile (Santiago)	190	100	100 **
Germany	100	3	10
India	730	55	-
Israel	152	85	18
Mexico (Mexico DF)	1,500	100	80
South Africa	70	16	-
Tunisia (Tunis)	68 *	75 *	-
USA (Arizona)	790 *	-	27 *

* planned expansion of existing reuse
** dry season conditions

(Sources: Ambrose and Lynn, 1986; Shuval et al., 1986; Strauss, 1986; Strom, 1985; Wang, 1984)

with a few notable exceptions, reuse occurs without effective controls and safeguards. In view of this, the global UNDP/World Bank Integrated Resource Recovery (Waste Recycling) Project included studies on wastewater reuse for agriculture and aquaculture in its research program. The project focused on the public health aspects of reuse and the sanitary control measures required for public health protection. In addition, an evaluation of policy frameworks and institutional arrangements for effective wastewater reclamation and reuse programs is now underway.

Benefits of Wastewater Reuse in Agriculture

Wastewater recycling by means of agricultural irrigation offers a number of potential benefits. The major direct benefits of effluent irrigation are (a) the increase in agricultural production due to increased irrigated area, multiple planting seasons, and/or increased productivity per unit area; (b) the environmental damages avoided due to irrigation reuse instead of other disposal options; and (c) the conservation of scarce water resources for other higher-value uses. Furthermore, many indirect benefits are realized since wastewater irrigation in developing countries contributes to an improved food supply, which is accompanied by nutritional and health benefits, and to a better ecological balance between the city and its rural hinterlands, resulting in rural jobs, settlement opportunities, and food production close to the city.

WATER QUALITY, HEALTH, AND ENVIRONMENTAL ISSUES

Some potential negative effects of wastewater irrigation merit attention. These include the risk of transmission of communicable diseases to sewage farm workers, persons consuming produce grown in wastewater-irrigated fields, nearby dwellers, livestock grazing on sewage-irrigated pasture land, and humans consuming meat or milk from such animals.

Toxic chemicals, including heavy metals, may be present in industrial and some municipal wastewaters. In addition to potential phytotoxic effects, there is concern that they might accumulate in food crops and endanger the consumer population. Industrial wastewaters should be isolated from the general municipal sewerage or receive appropriate pretreatment prior to being discharged to sewers where sewage farming is practiced. Other chemicals in wastewater can lead to salinization problems or can affect soil structure.

Consideration must also be given to the pollution (pathogenic microorganisms and chemicals carried by the wastewater stream) that effluent irrigation might contribute to surface water and groundwater adjacent to or underlying areas irrigated with wastewater. Finally, effluent quality may affect the choice of irrigation technology and practice.

Public Health Considerations

A comprehensive review of available epidemiological data was carried out as part of the UNDP/World Bank Resource Recovery Project. Based on the findings, a risk model was formulated to evaluate sanitary control options for effluent irrigation (Shuval *et al.*, 1986). The epidemiological model indicates that in the developing countries the highest risk of pathogen transmission, infection, and sickness is associated with the helminths, followed in order of importance by bacterial infections and viral infections. Although certain health risks are clearly associated with the use of raw wastewater in agriculture, as is evidenced by World Bank investigations of typhoid in Santiago, Chile, the epidemiological evidence compiled in the study suggests that the stringent wastewater irrigation standards applied in many of the industrialized countries are overly restrictive.

The UNDP/World Bank study by Shuval *et al.* (1986) suggests a guideline for unrestricted wastewater irrigation based on an effluent with less than one nematode egg (*Ascaris* or *Trichuris*) per liter and a geometric mean fecal coliform concentration of 1,000 per 100 ml. For restricted irrigation, such as for trees, industrial crops, fodder crops, fruit trees and pasture, less than one nematode egg per liter is recommended as a guideline. These recommended guidelines were endorsed by a group of experts at a meeting convened in Engelberg, Switzerland, in June 1985 by the World Bank, UNDP, WHO, UNEP, and the International Reference Centre for Waste Disposal (IRCWD, 1985). If these guidelines were routinely applied, no undue health risk of infectious disease transmission in effluent irrigation projects would arise.

Technical and Policy Options for Remedial Measures

A number of technological and policy options for reducing and controlling the health risks associated with wastewater reuse in agriculture were also evaluated by the UNDP/World Bank study (Shuval *et al.*, 1986). The following remedial measures were found to be the most effective for reducing possible negative health effects associated with raw wastewater irrigation: (a) wastewater treatment and/or storage practices aimed at effectively reducing the concentration of the priority pathogens to low levels (e.g. the above guidelines); (b) restrictions on the type of crops irrigated so as to prevent consumers from being exposed directly to vegetable or salad crops eaten raw; and (c) modifications of irrigation techniques and procedures so as to prevent or minimize direct contact between wastewater and crops.

Crop restrictions and modification of irrigation techniques have been highly effective in reducing infection from wastewater irrigation in developed countries with long traditions of civic discipline and effective methods of law enforcement. Such regulations work particularly well on centrally managed sewage farms or irrigation districts. However, they would be difficult to enforce among small subsistence sewage farms on the peripheries of developing country cities. Crop production on such farms is greatly influenced by nearby vegetable markets and depends heavily on sustained sources of inexpensive fertilizer, which can be provided by wastewater. This spontaneous economic demand makes it difficult to implement enforcement actions.

There are definite advantages to options that can be engineered and managed centrally and that do not require changing the life style, customs, and personal behavior of large populations to to be effective. The wastewater treatment option meets the following criteria: it is centrally engineered and managed and also provides protection for all exposed populations, from farmers to the public at large.

Only wastewater treatment that effectively removes helminths and also reduces pathogenic bacteria and viruses to an acceptable level can reduce the overall negative occupational and consumer health effects of wastewater irrigation. Clearly, the acceptability of treatment systems that can remove bacteria and viruses to some degree will depend on health profiles, immunities, and the priorities of the populations to be protected.

WASTE STABILIZATION PONDS AND EFFLUENT REUSE

Conventional wastewater treatment processes, with the exception of disinfection, are not effective for the removal of pathogens from sewage. Most conventional treatment plants are designed for maximum reduction of BOD (and in industrialized countries for maximum removal of nutrients!). Chlorination of sewage is not generally applicable in most developing countries because of operational and logistical problems, and it can only be applied effectively to secondary treated effluents. However, the UNDP/World Bank study has shown that waste stabilization ponds can be designed to remove virtually all helminths and a large number of bacteria and viruses. These ponds also produce a nuisance-free effluent rich in nutrients.

For developing countries, particularly in areas where vegetables are eaten raw, stabilization ponds are a preferred means of dealing with the health effects of wastewater irrigation. The highest level of overall protection is provided by multicell stabilization pond systems that can meet the effluent quality guidelines recommended for unrestricted irrigation. The data in Table 3 demonstrate the excellent fecal coliform removal performance of multicell pond systems having detention times greater than 25 days. Research at the San Juan ponds (Lima, Peru) supported by the UNDP/World Bank Project shows that the Engelberg guideline for unrestricted irrigation can be achieved in a five-pond system even with average daily water temperatures dropping to 18-19°C (Bartone et al., 1985). The San Juan pond research confirmed that fecal coliform removal efficiencies greater than 99.99 percent are obtained with appropriately designed pond systems, and that similar removal rates apply for Salmonella. In addition to the reuse of the San Juan pond effluents for irrigation, it was also demonstrated that fishculture for tilapia and carp is sanitarily acceptable in the advanced maturation ponds (Cointreau et al., 1987).

Waste stabilization ponds are particularly suitable for hot developing countries since ponds provide a robust, flexible, almost fail-safe treatment system having low construction and operation costs. In general, for hot climates a minimum 20-day, 4-cell stablization pond system should afford an adequate level of protection for unrestricted irrigation (Shuval et al., 1986). Although the land requirements for such multicell pond systems may be large, such systems are feasible, as is shown by the data in Table 4.

If restricted irrigation is being planned, then pond systems can be designed principally for helminth removal. An intermediate first-stage anaerobic pond with a detention time of 1-2 days followed by a facultative pond of 7-10 days detention will remove virtually all helminths (Shuval et al., 1986). At the San Juan ponds it was demonstrated that complete

TABLE 3 Reported Bacterial Removal Efficiencies of Multicell Waste Stabilization Ponds with Detention Times > 25 Days

Pond System	No. of cells	Effluent quality (F.C./100ml) *
Australia, Melbourne	8-11	100
Brazil, EXTRABES	5	30.
France, Cogolin	3	100
Jordan, Amman	9	30
Peru, Lima	5	100
Tunisia, Tunis	4	200

* F.C. = Fecal Coliforms

(Sources: Bartone et al., 1985; Cadilion and Tremea, 1985; Mara and Silva, 1986; Strauss, 1986; Water Science Labs, 1977)

TABLE 4 Data on Large-scale Pond Installations

City	Flow (m^3/s)	Pond area (ha)	Type *
Melbourne, Australia	2.2	1,499	AN+F+M
Auckland, New Zealand	2.4	530	F
Stockton, California, USA	2.9	250	-
Amman, Jordan	1.4	200	AN+F+M
Mexicali, Mexico	1.2	140	AN+F
Napa, California, USA	0.4	140	F+M

* AN = anaerobic, F = facultative, M = maturation

(Sources: Bartone, 1985b; Croxford, 1978; Mara, 1976; Thorton et al., 1985)

helminth removal was accomplished by a facultative pond in series with a maturation pond with a total of 5.5 days detention time (Yanez et al., 1980). The San Juan pond experience suggests that all human enteric parasites, both helminths and protozoa, can be effectively eliminated from effluents using a two pond series with 10 day detention time and baffled effluent weirs to prevent the breakthrough of solids (Bartone, 1985a). In Israel, initial anaerobic ponds in series with deep storage reservoirs (greater than 60-90 days detention time) have also proven very effective for pathogen control (Shuval et al., 1986). Finally, where raw sewage irrigation cannot be eliminated but must be controlled, first-stage construction of anaerobic ponds with 1-2 days detention will remove an estimated 90-95 percent of at least the exposure to, if not the incidence of, hookworm and other helminth infections (Shuval et al., 1986). The immediate health benefit-cost ratio of this option is high, even if such temporary plants are not completely free from nuisances.

Effluent Quality and Irrigation Practice

Wastewater irrigation will also supply almost all of the nitrogen and most of the phosphorus and potassium required by many crops, as well as important micronutrients. Pond effluents, because of their combined soluble BOD and algal biomass content, have high fertilizer value and the algae act as a slow-release fertilizer. These nutrients are important to the agricultural economy of developing countries where fertilizer costs are a major burden in cash outlay for farmers. The fertilizer value of wastewater has been estimated at 3-5 US cents per cubic meter in some cases (Shuval et al., 1986; Streit, 1986). Organic matter in the wastewater can also contribute to soil tilth and overall long-term fertility. Because of the amount of nitrogen in wastewater, however, problems could occur late in the growing season with certain crops such as cotton owing to excessive vegetative growth (Ayers and Wescott, 1985).

The total soluble salt content of municipal effluent is always higher than that of the supply water, and waste stabilization ponds in hot, dry climates may contribute substantially to this increase in salinity because of the large surface area for evaporation. The use of deeper ponds would help to reduce this impact. Excess irrigation water salinity may contribute to salinization problems and impair crop growth. In arid and semiarid zones, the lack of sufficient rainfall and the scarcity of irrigation water may lead to inadequate leaching and to the accumulation of excess salts in the crop root zone unless special steps are taken to avoid it.

In arid and semiarid areas, the low-rate application of effluent is recommended for irrigating crops. With highly efficient irrigation, the application rate could be about 2,500 m^3/ha/yr (that is, the effluent of about 75 persons at 90 l/c/d could irrigate one hectare). A range of 5,000-10,000 m^3/ha/yr is probably more typical for irrigation in developing countries where ridge and furrow irrigation is common. If greater irrigation efficiencies are to be achieved, then closed-conduit systems, such as sprinkler, trickle, drip, or root-zone irrigation systems, are needed which can deliver water to crops on demand. However, such pressurized systems are susceptible to clogging by suspended solids is wastewater. Algae present in stabilization pond effluents may contribute to clogging in cases where this irrigation technology is used. The clogging problem in pressurized systems can be resolved by appropriate orifice size selection, by the use of gravel filters to remove suspended solids from the effluents, or by screen filters of 60-mesh to 200-mesh inserted before manifolds and/or individual laterals (Shuval et al., 1986). New bubbler irrigation technology has recently been developed that avoids clogging problems and maintains the desirable features of other closed-conduit systems (Hillel, 1987). No emitters of any kind are used, and the water is simply allowed to "bubble out" of open vertical tubes of 1-3 cm in diameter. Bubbler irrigation is a low-cost, low-maintenance, high-efficiency system that may be very suitable for irrigation with pond effluents.

PLANNING AND MANAGEMENT ISSUES

While wastewater reclamation and reuse is becoming an increasingly important issue in arid and semiarid developing countries, it is necessary to consider it within an overall water resources development and management framework. Many irrigation reuse projects are not properly conceived and risk failure due to one or more of the following causes:

(a) lack of national policies and strategies for reuse;

(b) insufficient attention to social, institutional, and organizational aspects;

(c) inadequate financial basis to ensure long-term sustainability;

(d) unduly expensive technological choices; and

(e) failure to properly consider public health, and environmental issues.

Thus far the main emphasis has been on the technological, public health and environmental issues, which of course are essential considerations, but they can only be dealt with effectively within a broader framework of national reuse policy as part of water resources planning and management. Countries that overlook or ignore the reuse question are missing the point. In dry climates, if reuse is not planned and reuse policies not defined, reuse will almost surely take place anyway out of economic necessity--but without adequate sanitary controls. Examples abound of local farmers breaking into sewer interceptors both within and on the outskirts of urban areas to steal the effluents for watering their crops. These are often vegetable crops destined for local markets that will be consumed raw. In addition, indirect reuse is occurring everywhere. Highly polluted rivers serve as major water sources for large-scale irrigation projects.

While the composite benefits of using effluents for irrigation are obvious, institutional and organizational factors present certain difficulties. Lines of responsibility and cost allocation formula have to be worked out among the various sectors involved: the Municipality, which must otherwise pay for more costly or less effective treatment and disposal options; the Farmer, who receives additional irrigation water; and the State, which is concerned with alleviating the water scarcity and protecting environmental quality.

Mutually beneficial solutions have been found. A recent World Bank project evaluation recommended that policies be adopted for setting up regional organizations to operate reservoirs, pumping stations, and conveyance systems and for allocating treated effluent in accordance with agreed criteria (Streit, 1986). Regional organizations of this type, which function as closed systems, not only provide the framework and administrative mechanisms necessary for effective wastewater utilization but also permit effective monitoring and control of the crops irrigated, the quality of water, and the associated public health effects.

Although the responsibility for collecting, treating and disposing of urban sewage corresponds to the municipality, local farmers are often able and willing to pay for the effluents they use for irrigation (but not to subsidize all the disposal costs of the municipality). Such payments should cover incremental treatment required for reuse purposes plus additional conveyance costs. This may take the form of direct water use tariffs paid to the sewage entity or water resources management authority, or of a willingness to share in the investment in the treatment works which are a prerequisite to obtaining reuse permits. Cost sharing can be in the form of either cash payments or in-kind contributions such as land for siting treatment or storage facilities. Experiences in Peru also indicate that local farmers may be willing to perform operational and maintenance tasks associated with treatment, storage, and conveyance works as an in-kind contribution.

Benefit-cost studies made in Peru showed that the irrigation components in reuse schemes were feasible even if land and operation and maintenance for treatment was charged to farmers, but they were not economically viable if the full cost of investment in treatment facilities were charged against the agricultural component. In this case feasibility depends on the alternative minimum cost of treatment required for disposal without reuse.

Finally, the legislative framework for effluent reuse can also influence project feasibility. Authorities in Mexico are able to impose effective crop restrictions in Irrigation Districts there because they are empowered to withhold water from farmers not observing the regulations. In Chile, by comparison, the water law vests water rights in the farmers (landowners), and the sanitary authorities have little leverage. The authorities have never been successful in imposing crop restrictions even though lettuce and other vegetables irrigated by raw sewage have been implicated in annual typhoid epidemics in Santiago according to World Bank studies (Shuval et al., 1986). Rather than seeking an integrated solution, the Santiago water utility may have to assume the full cost of treatment and conveyance to a new discharge point, which will preclude farmers' access to the effluent--a nonoptimal solution for all concerned.

CONCLUSIONS

In both permanently and seasonally arid areas, wastewater irrigation has numerous benefits, most notably increased agricultural productivity, improved food supply, and reduced environmental pollution. In the final analysis, the total societal benefits of wastewater reuse in irrigation undoubtedly exceed the corresponding negative effects that can be easily controlled. To ensure that negative effects are successfully controlled, however, a number of policy issues must be addressed and resolved.

Technological Issues: The principal issues for developing country officials and their advisors are related to selection of sewage treatment technology and other remedial measures, choice of appropriate irrigation technology, and close matching of capacity with demand. Treatment choices should be based on the following criteria in order of priority: maximum helminth removal; effective reduction of bacteria and virus; and a nuisance-free effluent. In hot climates, waste stabilization ponds can efficiently meet all three criteria.

Environmental Issues: Environmental standards and criteria for wastewater reuse in industrial countries are often adopted by developing countries without critical review of their applicability. Standards and control policies should be established based on an appropriate risk-benefit analysis.

Economic and Financial Issues: The principal issues are complete economic costing, financial and economic benefit-cost analysis, and cost recovery within a dual-purpose project--wastewater disposal and agricultural productivity. The latter requires equitable allocation of costs among beneficiaries, namely municipalities and farmers.

Social Issues: Public acceptance of effluent irrigation may depend on cultural or religious attitudes toward excreta disposal and traditional recycling practices, issues that should be dealt with in the early stages of project identification. Health promotion and education components may have to be built into reuse projects. Farmers' willingness to pay for reclaimed water must be estimated if rational cost recovery policies are to be implemented.

Institutional and Organizational Issues: Effluent irrigation projects require an intersectoral institutional framework. Responsibilities must be assigned and capacity created for operation and maintenance of treatment systems and for monitoring and enforcement of effluent standards and crop restriction policies.

Water Planning and Management Issues: Reuse should fall within an integrated, multisectoral policy framework for water development in the country that includes the clear definition of reuse priorities and strategies as well as coherent laws and regulations.

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DISCLAIMER

The views and interpretations expressed are those of the authors and do not necessarily represent the views and policies of the UNDP or the World Bank.

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REUSE OF STABILIZATION POND EFFLUENT FOR AGRICULTURAL IRRIGATION IN ISRAEL

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ABSTRACT

Stabilization Pond (SP) effluent is the most immediately available additional water resource for irrigation in semiarid areas as Israel. Evolution of SP design in Israel has been directed by the needs of the irrigation reuse, which led to the enlargement of the storage capacity and improvement of treatment efficiency. The use of raw sewage for irrigation may endanger public health, but safe use of wastewater can be achieved by various degrees of treatment which can adequate effluents quality to sanitary standards. Algae and zooplankton in SP effluents may clog drip irrigation networks but there are several alternatives to overcome the problem. The future challange is to transform wastewater from a potential pollutant with disposal problems, into reclaimed effluent of high quality fit for unrestricted irrigation, by means of a cheap or even economically profitable system which should include biomass production.

KEYWORDS

Stabilization Ponds, Stabilization Reservoirs, Wastewater Irrigation, Wastewater Irrigation Standards, Biomass Production.

WHY STABILIZATION POND EFFLUENT FOR IRRIGATION

Israel is currently utilizing practically all of its conventional water resources of about 1600 million m^3 per year. Renovated or reclaimed wastewater constitutes the most immediately available additional water, particularly for agriculture which already utilizes 70 % of the Israeli water resources (Table I). Out of 260 million m^3 of potential municipal effluent only 170 will be treated and only 90 will be reused for irrigation in 1987. However, big efforts to reclaim and reuse the effluent are underway, and reuse of 430 million m^3 per year is planned for the year 2010 (Water Commission, 1985; Schwartz, 1986).

Solar energy is the sole free and abundant energy resource in the country. Thus, SP have been the natural option in contrast with the high energy consuming mechanical treatment systems. Moreover, since irrigation is carried out in Israel only in the dry summer while the need is for the year-long effluent flow, seasonal storage is unavoidable. The use of SP as storage devices started even at the earliest development of the country. Stabilization ponds and stabilization reservoirs are responsible for the total or partial treatment of most of the treated sewage in Israel (Table II).

EVOLUTION OF STABILIZATION PONDS DESIGN IN ISRAEL

Irrigation as the end-use of effluents determined the way SP design evolved in Israel (Fig.1). The need to rise storage capacity was one of the main acting forces. Improvement of treatment efficiency was also necessary to overcome the high land costs of big SP, and to allow massive reuse of effluents and unrestricted irrigation without affecting the environment quality and public health.

Table I. Water balance in Israel.*
Million m^3 / year.

Year	1985	1990	2010
RESOURCES			
Aquifer	1,210	980	980
Surface waters	700	680	725
Brackish waters	175	200	240
Wastewater	60	150	430
TOTAL	2,145	2,010	2,375
DEMAND			
Municipal	435	490	780
Industrial	100	115	135
Irrigation	1,445	1,230	1,250
Judea & Sammaria	105	125	180
Losses	60	50	30
TOTAL	2,145	2,010	2,375

* Source: Schwartz (1986).

Table II. Inventary of sewage treatment systems in Israel.*

Kind of treatment	No.of units	%	Conected settlements	%	Sewage volume m^3/year X 1000	%
Activated sludge	1	0.2	6	0.6	5,750	3.4
Trickling filters	1	0.2	1	0.1	1,095	0.6
Act. sludge & Tr. filters	1	0.2	9	0.8	21,324	12.7
Extended aeration	4	0.8	6	0.6	9,225	5.5
Aerated lagoons	24	4.9	46	4.3	29,813	17.7
Oxidation ditches	1	0.2	1	0.1	700	0.4
Recirculation ponds	1	0.2	14	1.3	25,000	14.8
Compact Act. sludge	21	4.3	21	1.9	318	0.2
Sediment. pond + Sta. pond	268	54.5	331	30.8	47,979	28.4
Stabilization ponds	33	6.7	42	3.9	1,563	0.9
Sed. pond + Sta. reservoir	23	4.7	26	2.4	5,136	3.0
Septic tank + Sta. reservoir	4	0.8	4	0.4	228	0.1
Stabilization reservoir	10	2.0	11	1.0	1,695	1.0
Sep. tank + Percolat.ditche	58	11.8	58	5.4	533	0.3
Sedimentation pond	13	2.6	14	1.3	796	0.5
Septic tank	29	5.9	29	2.7	703	0.4
Ground percolation tank	-	-	456	42.4	17,007	10.1
TOTAL	492	100	1,075	100	168,865	100

* Source: Water Commission (1985).

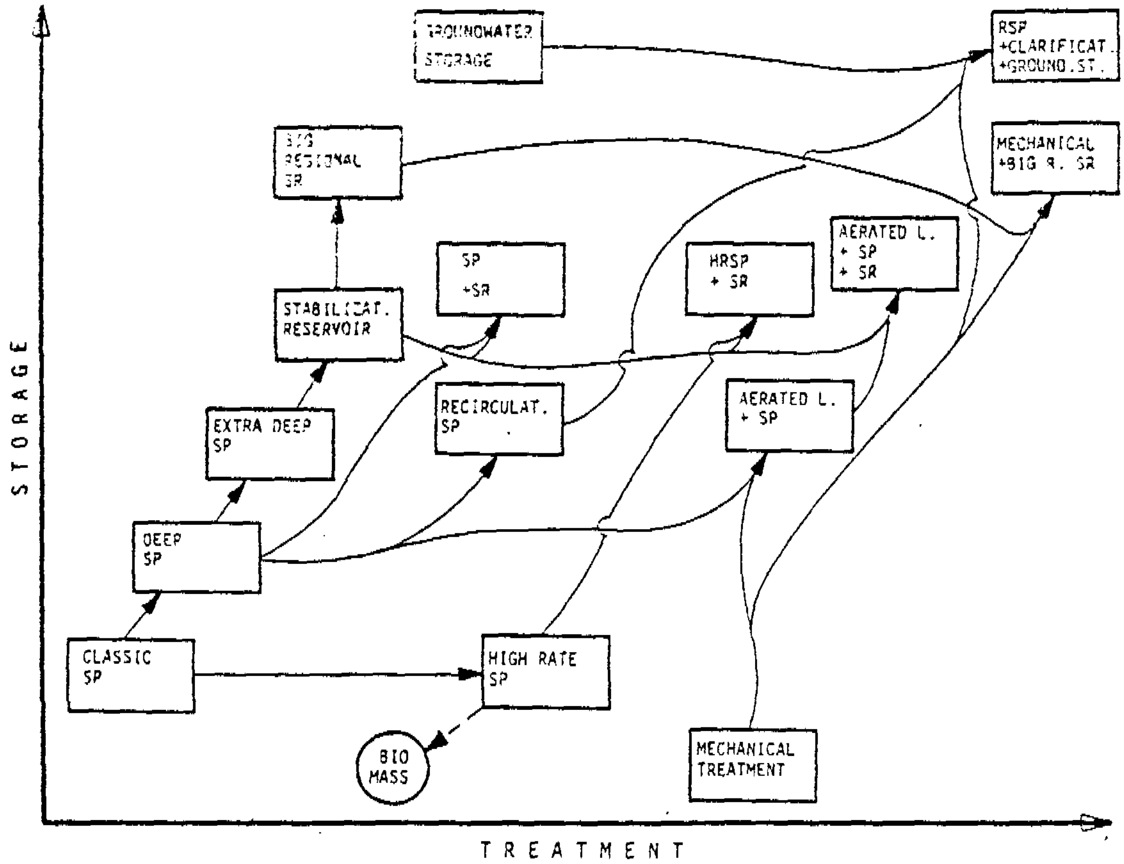

Fig. 1. Evolution of the design of SP in Israel. Main acting forces derived from irrigation reuse: increase of storage capacity and treatment efficiency. SP (Stabilization Pond), SR (Stabilization Reservoir), RSP (Recirculation SP), HRSP (High Rate SP). "+" refers to units working in series.

Rising Storage Capacity

"Classical SP" (60 cm deep, 5 days retention time) had two main problems: growing of vegetation from the bottom of the pond and low storage capacity. Deep SP (1.5 m deep) resolved these two problems and also increased the retention time. In Israel they are generally used following a small anaerobic sedimentation pond. Loading of the sedimentation pond is about 1500 kg BOD/ha/day and of the SP 150 kg BOD/ha/day. Extra Deep SP (2.5 to 5 m deep) represent a further step in increasing storage capacity, and are specially suitable to irregular terrains (Wachs & Berend, 1968).

Stabilization Reservoirs (SR) were introduced in the early '70s and more than 100 of them are in operation in Israel today. They were initially conceived just as storage reservoirs, but soon their high treatment capacity became evidence (Table III). Storage capacity vary from 50,000 m^3 to 1.5 million m^3 and depth from 6 to 12 m. SR may receive raw sewage, partially treated wastewater, floods, fish pond outflows and other marginal waters. However, the most common unit in small rural settlements is made of two parallel anaerobic sedimentation ponds, one SP and one SR (Fig. 2). SR are filled all year round and emptied in two or three summer months. Then water level decreases at almost 10 cm per day, leading to strong changes in the reservoir ecology and, as a consequence, in effluent quality. SR are relatively new and there are only few and inadequate criteria for their design. Shelef et al. (1977) developed a simple model based on four compartments (BOD, Sludge, Algae, Dissolved Oxygen) and showed that both loading and depth are key factors determining SR performance. A research program is presently underway to develop a new comprehensive model based on a deeper insight of SR limnology (Shelef and Juanico, 1987).

Table III. Stabilization Reservoir effect on effluent quality. As outflow quality is variable, given values are approximated averages.

RESERVOIR (source	VOLUME m^3X1000	DEPTH m	INFLOW BOD	INFLOW COD	OUTFLOW BOD	OUTFLOW COD	OXYGEN CONDITION
Hasidim(1	500	6.5	60	230	9	120	aerobic
Kishon (2	12,000	10	75	170	9	60	aerobic
Getahot(4	50	5.5	80	260	30	150	facultat.
Adashim(3	560	7	250	500	110	360	fac./anae
Genigar(4	900	8	600	1100	200	500	anaerobic

1) Eren (1978), 2) Rebhun (1986), 3) Shelef et al. (1977), 4) Shelef and Juanico (1987).

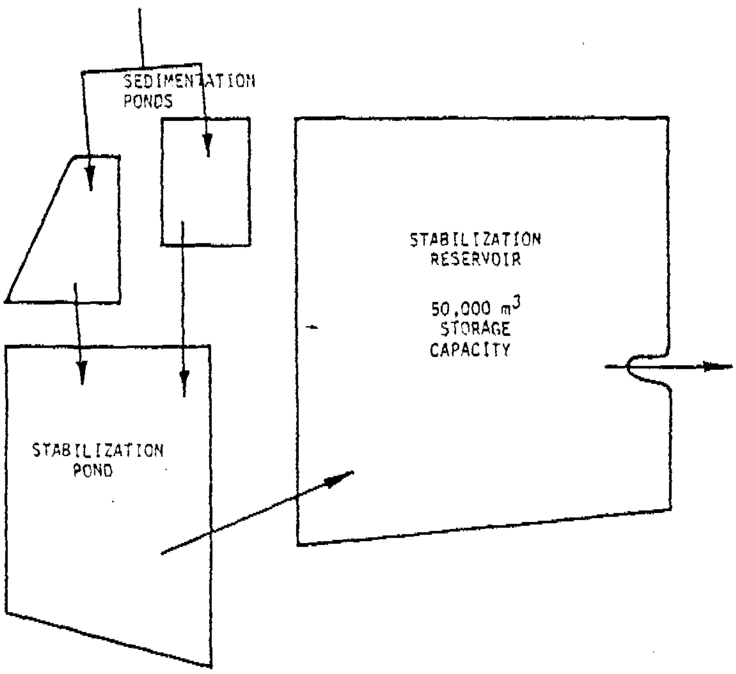

Fig. 2 . Scheme of L. Getahot Kibbutz treatment and storage system, typical of small rural settlements.

Big regional SR were the solution for seasonal storage of wastewater from big cities or wide areas. Einan (5 million m^3) and Maale Kishon (12 million m^3) were the first SR of this kind in the country. The Kishon Complex (Fig. 3) is the largest wastewater reclamation system for agriculture irrigation in Israel. The municipal sewage of Haifa (about 15 million cubic meters / year) is treated through parallel trickling filters and activated sludge. These effluents are chlorinated, and pumped as the only inflow to the Maale Kishon SR. This SR has a long retention time and is divided in two halves by a dam which prevents short-circuiting and facilitates operation. The outflow from this SR is again chlorinated and driven to an Operational Reservoir. This second reservoir receives waters from several sources, and supplies directly to the irrigation network.

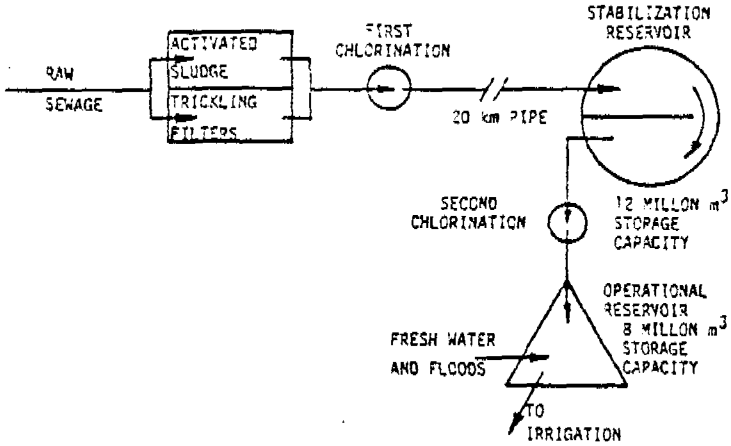

Fig. 3. Scheme of the Kishon Complex for the treatment and storage of effluents from the city of Haifa.

Improving Treatment Efficiency

The Recirculation SP system (RSP) (Shelef et al., 1978) is a series of SP where the first one receives not only the raw sewage but also effluents recirculated from the last pond in the series. Thus, the first pond can receive loadings up to 400 kg BOD/ha/day and a facultative regime instead of an anaerobic one is maintaned. A further advance in this direction was the step feeding. It was noted that in a series of five RP the largest part of the sewage treatment (e.g. BOD reduction) occurred in the first one or two ponds. Thus, the raw sewage was splited and part of it fed directly to the third pond. This allowed an additional load of the series without affecting the final outflow quality. RSP are presently used to treat the sewage from the Great Tel Aviv. Effluents are clarified, driven to polishing ponds and sent to groundwater recharge for later use in agricultural irrigation.

The High Rate SP (HRSP) (Shelef et al., 1978) are a still more intensive form of SP. Their shallow depth (0.2 to 0.5 m) makes a great part of the cross-section of the pond available for photosynthesis, and moderate mechanical agitation improves mass transfer among the components of the system. Retention time can be reduced to 2-7 days and organic loading increased to 400-800 kg BOD/ha/day. The high algal-bacterial biomass can be recovered as animal protein rich feed.

Some systems combine the high efficiency of mechanical treatment with the buffer and storage capacity of pond treatment, as in the above mentioned Kishon Complex. Another case is the Beersheva City Treatment Plant, composed of aerated lagoons whose effluents are driven to polishing SP and later storage in a SR.

PUBLIC HEALTH ASPECTS OF IRRIGATION WITH EFFLUENTS

The storage and reuse of SP effluents implies the potential or actual contact of part of the population with the effluents. Raw sewage is a pathway for water-borne diseases, but proper treatment overcomes this problem. Kott and Betzer (1972) found that Cholera bacteria died in SP after 24 h, and that chlorination tests performed in SP effluents inoculated with *Vibrio cholerae* showed that chlorine at 1 mg/l effectively killed bacteria after 30 min contact. Twenty years ago it was regarded as useless to chlorinate SP effluents because of the possible high chlorine demand of algae, but Kott (1973) showed that the algal cell wall is resistant to chlorine penetration for about 2 h. In SP effluents, 8 mg/l chlorine 1 h contact reduce *E. coli* B by three orders of magnitude. Kott (1975) also studied the storage of trickling filter effluents in a SR. Eggs of worms, cystes of amoeba and other relatively heavy organisms settle within the reservoir. Enteric viruses and bacteria die off in the SR but not enough to allow unrestricted irrigation. Enteric viruses are eliminated by 20 mg/l chlorine 2 h contact, or by 40 mg/l 1 h contact. Kott (op. cit.) recommends chlorination of trickling filter effluents with 20 mg/l chlorine 2 h contact, and storage in a SR up to the final elimination of viruses (about 2 months). Israel standards for wastewater irrigation (Shelef, 1977) include chlorination together with effluent quality parameters.(Table IV).

Table IV. Summary of Israeli Standards for Irrigation with Wastewater Effluents (Shelef, 1977).

Group	A	B	C	D
Total BOD_5 mg/l	60	45	40	25
Dissolved BOD_5 mg/l	-	-	20	15
Suspended Solids mg/l	50	40	35	20
Total Coliforms / 100 ml	-	-	100	12
Dissolved Oxygen mg/l	0.5	0.5	0.5	0.5
Cl^2 contact minutes	-	-	60	120
Cl^2 residual, mg/l after 60 min contact time	-	-	0.15	0.5
Minimum distance to residential area m	300	250	-	-
Minimum distance to paved road m	30	25	-	-

A:Cotton, sugar-beet, grains, seeds, dried fodder and other crops. Forest and wooded areas not used for recreation.
B:Green fodder, olives, dates, peanuts. Peeled fruits such as almonds, citrus, nuts, bananas. Ornamental trees and recreational forest.
C:Fruits (Irrigation under the trees and should be stopped two weeks before picking time). Vegetables with peels, eaten often after cooking or for conserving industry. Parks, golf courses, lawns (irrigation area closed to public 24 h after irrigation).
D:Unrestricted irrigation.

Fattal et al. (1983), Margalith et al. (1983) and others, performed a multidisciplinary research on populations of areas irrigated with effluents. Thirty kibbutzim with and without effluent irrigation systems totalizing 13,500 inhabitants were covered. The degree of exposure of residential areas in the proximity of sprinkler effluent irrigated fields to airborne aerosols was described by an Exposure Index which includes distance of irrigated fields, wind direction and velocity, size of irrigated field, and length of irrigation period. No correlation was found between enteric disease rates in the population and the aerosol Exposure Index. A seroepidemiological survey of antibodies of echoviruses showed no relation with the exposure of people to wastewater, even considering the persons encharged of the wastewater irrigation system.

EFFECTS ON THE IRRIGATION NETWORK

The high concentration of algae and zooplankton in SP and SR effluents may cause serious clogging problems in the drip irrigation systems. Farmers take SR effluents not from the surface of the reservoir but from the 0.7 to 1.5 m depth layer where suspended solids are fewer. Several kinds of filters are used, but experience recommends filters with automatic counter current cleaning mechanisms, together with appropriate pressure and periodical cleaning of all the irrigation network with chemicals (Thirosh, 1986).

THE FRONTIERS OF RESEARCH ON SP EFFLUENT REUSE

Although unrestricted irrigation with SP effluents is still not performed in Israel all efforts are oriented to do so. Algae and zooplankton recovered by physico-chemical and biological means would provide high quality effluents and a very valuable by-product (Shelef and Soeder, 1980). Fish culture in SP and SR would, too, work as a living filter for plankton and suspended solids recover, offering clean water and very valuable yields (Hepher and Schroeder, 1977; Milstein et al., 1985).

The challenge we face is to transform sewage from a potential environmental pollutant which creates disposal problems, into high quality effluent for unrestricted irrigation and biomass production, by means of a system not only cheap but even economically profitable. The solution should be a balanced combination of technologies where the Stabilization Ponds and

Stabilization Reservoirs play the central role. A comprehensive model of such a system is necessary in order to optimize it from both technical and economic points of view. Although this model may become a sophisticated one, the final design of the system may be very simple and of use even in rural areas of underdeveloped countries.

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REUSE OF STABILIZATION POND EFFLUENT FOR CITRUS RETICULATA (ORANGE), FOREST AND AVENUE PLANTS

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ABSTRACT

Utilisation of domestic sewage, treated in a stabilization pond, for growing orange (*Citrus reticulata* var - Nagpur Orange), forest and avenue plants was initiated at the National Environmental Engineering Research Institute, Nagpur in 1978. The irrigation quality of the stabilization pond effluent (SPE) was compared with that of well waters at NEERI and at the orange farms of Katol and Warud (35 to 55 km north of Nagpur). The SPE was found to be comparable with these well waters. Soils on NEERI fields were also found comparable with those of some of the orange farms at Warud and Katol. Irrigation scheduling was formulated for orange at NEERI. To study effects of SPE irrigation, some of the orange plants were irrigated with well water and others with the SPE. Fruiting on most of the plants was not regulated; effluent irrigated plant leaves exhibited similar osmotic concentration but higher succulence than well water irrigated plant leaves; there was an increase in potassium, decrease in sodium and no effect in calcium content in the water extracts of SPE irrigated leaves. Amongst the forest and avenue plants, *Cassia siamia*, *Peltophorum inerme*, *Enterolobium saman*, *Casuarina equisetifolia*, *Eucalyptus citrodora* *Lagerstroemia speciosa*, *Terminalia catappa* and *Leucaena glauca* responded well to SPE irrigation, though the response varied with the soil conditions. A garden developed around pilot sewage treatment plants clearly demonstrates scientific reuse of treated sewage for creating an appealing landscape and beautification of otherwise discarded areas.

KEYWORDS

Stabilization pond effluent, irrigation scheduling, evapotranspiration, leaf succulence, effluent irrigation .

INTRODUCTION

Water pollution control and environmental preservation are important research objectives of the National Environmental Engineering Research Institute (NEERI) Nagpur. In this context, the importance of developing controlled recycling was recognised and a project, on utilisation of city sewage, treated in a stabilization pond, was initiated in 1978 for irrigation of orange, forest, avenue and ornamental plants. Some important features of this work are presented in the present paper .

MATERIALS AND METHODS

Sewage treatment, sites and plant species for irrigation :

A part of the city sewage receives treatment in a stabilization pond; yet a smaller part of the sewage is treated in a number of pilot treatment plants. The effluents are used for irrigation of orange, a few other orchard plants, forest and avenue species and an ornamental garden. There are 3 wells for irrigation of the Institute's main garden and a number of orange and sapota (*Achras zapota*) plants. Figure I shows the layout of the area.

The work was initiated in June 1978 on field-I with plant spacings larger than the conventional for use of heavy machinery to remove bushy and wild growth after the rainy season. In June 1979 plantation was carried out on field-II using the conventional spacings (5.5 x 5.5 m). Plants on Field-I were irrigated with the stabilization pond effluent (SPE). Plants on Field-II were irrigated with well waters during the first year. In 1980, these plants were divided into 2 groups : (1) plants on the eastern side of the constructed drain receiving well water irrigation and (2) plants on the western side of the constructed drain receiving SPE irrigation .

During 1979-80 avenue plants were planted along the main road to the Institute and along the road from the main gate (MG) to the Guest House. During 1981, a number of forest and avenue plants were planted on Field-III for SPE irrigation. Ornamental flowering plants were planted on a small area surrounding the pilot sewage treatment plants, the effluent from which was used for irrigation of these garden plants. Plant species on various sites are listed in Table 1 .

Characteristics of SPE, well waters and soils :

The SPE, well waters at NEERI and those from orange farms at Warud and Katol were analysed for important irrigation quality parameters. The SPE was also analysed for other parameters such as algae, nutrients and some metals that are found in well waters only in insignificant amounts. Standard Methods (1975) were used for these determinations. Soil-water extracts (1:2) were analysed for field-I at NEERI and some of the orange farms at Warud and Katol. Soils on fields and other sites at NEERI were also analysed for other parameters such as texture, exchangeable calcium, cation exchange capacity (CEC), nutrients and organic matter. Soil properties of water retention against 5 lbs/in2 (5 psi) and 220 lbs/in2 (220 psi) corresponding to the soil water contents at field capacity and wilting respectively, were determined using pressure plate equipment (Richards,1954). After the irrigation, the loss of water from the soil through evapotranspiration was also monitored using 30 to 45 cms long tensiometers. Methods described by Black (1965) and Jackson (1968) were used for the soil analysis and tensiometer investigations.

Scheduling of irrigation for orange plants :

For scheduling of irrigation for orange plants, daily mean potential evapotranspiration (ET_o in mm/day) at Nagpur was computed for each month using the modified Penman Method (FAO,1977). Corresponding values for daily mean crop evapotranspiration (ET-crop) were computed using the relation ET-crop = ET_o x kc where 'kc' is the crop coefficient which is taken 0.6 for orange plants (Water Management Unit, 1978) .

Quantity of irrigation water to be given was determined on the basis of wetting 50 cms of the top soil to the field capacity. After this, next irrigation was given only when the water content in the top 50 cms soil was reduced to about wilting percentage through evapotranspiration. The quantity of water evapotranspired divided by the ET-crop (mm/day) during each month gave the irrigation interval during each month .

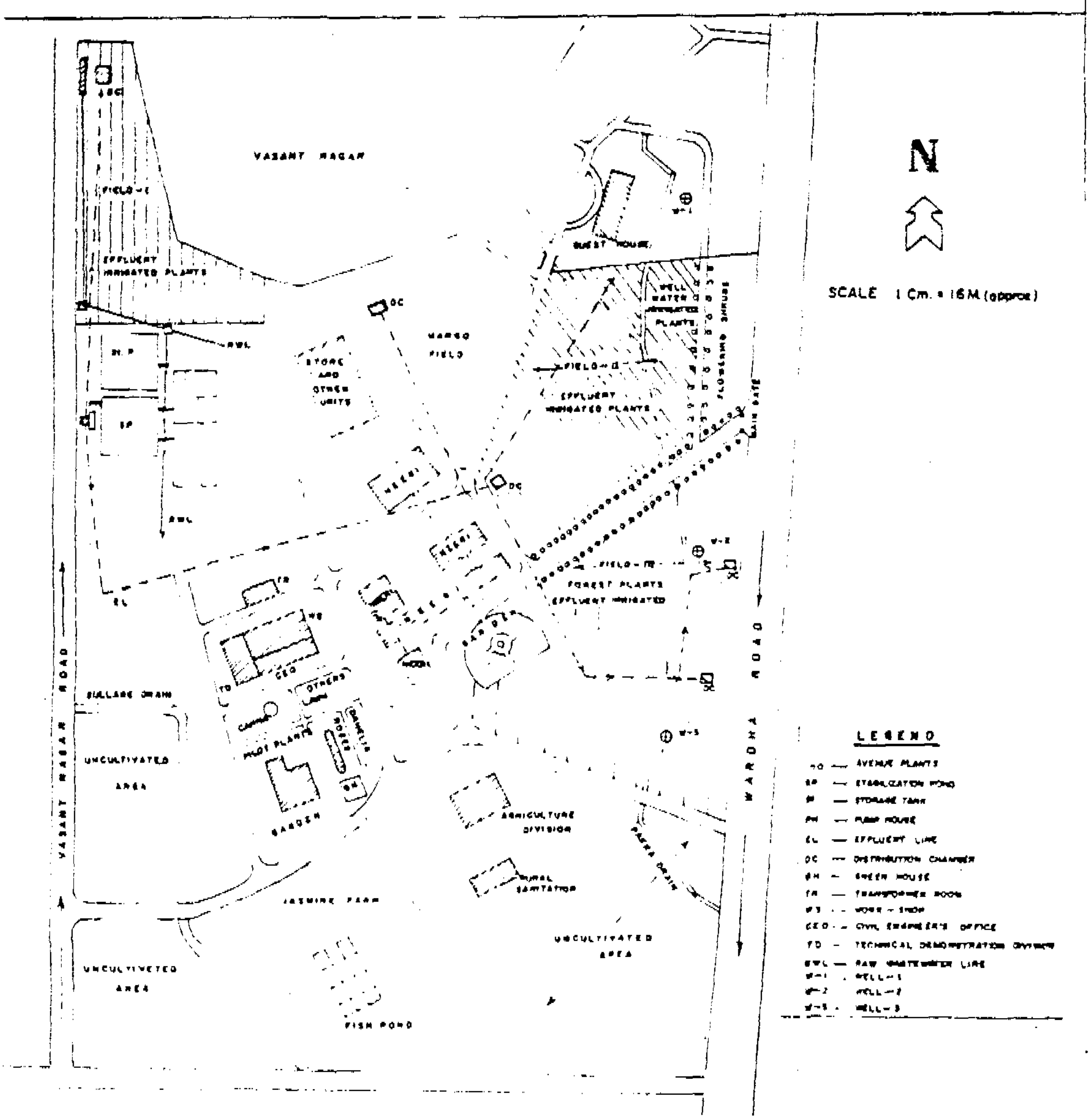

FIG 1 SITE PLAN FOR UTILIZATION OF STABILIZATION POND EFFLUENT FOR ORANGE, FOREST, AVENUE AND ORNAMENTAL PLANTS.

TABLE 1 List of Plant Species Planted on Various Fields, Roadsides and in the Garden around the Pilot Plants

Sl.No.	Scientific name of the plant species	Number of Plants Planted on different sites				
		Field I	Field II	Field III	Roadside	Garden around Pilot Plant
1.	*Achras Zapota* Linn.	3	30	-	-	-
2.	*Averrhoa carambola* Linn.	-	-	16	-	-
3.	*Bauhinia purpurea* Linn.	4	-	-	-	-
4.	*Cassia fistula* Linn.	-	-	4	-	-
5.	*Cassia siamea* Lam.	-	-	10	-	-
6.	*Casuarina equisetifolia* Linn.	5	-	-	-	2
7.	*Citrus aurantifolia* (Christm) Swingle	1	35	-	-	2
8.	*Citrus reticulata* Variety - Nagpur	106	550	-	-	-
9.	*Cocos nucifera* Linn.	10	-	-	-	-
10.	*Delonix regia* Rafin.	4	-	3	-	6
11.	*Emblica officinalis* Gartn.	-	-	16	-	-
12.	*Enterolobium saman* Prain.	-	-	16	-	-
13.	*Eucalyptus citriodora* Hook.	4	-	-	-	-
14.	*Lagerstroemia speciosa* pers.	2	-	16	-	1
15.	*Leucaena glauca* Benth.	-	-	30	-	-
16.	*Melia azadirachta* Linn.	4	-	14	-	-
17.	*Mimusops elengi* Linn.	-	-	-	-	4
18.	*Mimusops hexandra* Roxb.	3	2	-	-	-
19.	*Oreodoxa regia* H.B. & K.	-	-	-	18	5
20.	*Peltophorum inerme* Nives	-	-	20	-	-
21.	*Polyalthia longifolia* Thw.	-	-	-	18	12
22.	*Pongamia glabra* Vent.	-	-	16	-	-
23.	*Psidium guajava* Linn.	8	-	-	-	-
24.	*Putranjiva roxburghii* Wall.	-	-	6	-	-
25.	*Sapindus mukorossi* Gaertn.	-	-	16	-	-
26.	*Saraca indica* auct non Linn.	-	-	-	20	-
27.	*Syzygium cuminii* (Linn) Skeels	-	-	32	-	-
28.	*Terminalia catappa* Linn.	5	-	16	-	2

Effect of SPE irrigation on succulence and leaf-water extract characteristics

For these studies, 5 plants of the same age were selected from each of the two groups of plants on Field-II. From each of these plants, two sets of 5 full grown, green leaves were collected. One set was processed to make 100 ml of water extract of the leaves using, for grinding, acid washed sand and distilled water. The filtered leaf extract was analysed for EC, soluble Na^+, K^+ and Ca^{++}. The other set of leaves was processed for determination of leaf succulence gravimetrically (% water in fresh leaves).

Performance of avenue, forest and garden plants

Inferences on the suitability of the SPE irrigation for avenue, forest and garden plants were drawn from field observations supported by soil analysis. Suitability of some of these plants for SPE irrigation was evaluated under different soil conditions encountered on the Field-III .

Garden around the pilot sewage treatment plants

The pilot plants indicated in Fig. I are based on aerobic treatment of domestic sewage and include an oxidation ditch, a trickling filter unit, a rotating biological disc and a surface aerator. The area around these units has been converted into a small but planned ornamental garden developed since January 1980 .

RESULTS & DISCUSSION

Irrigation quality of SPE and well waters

Analysis of the SPE, well waters at NEERI and those from orange farms at Warud and Katol for important irrigation quality parameters is presented in Table 2 .

TABLE 2 Important Irrigation Quality Parameters of SPE and well waters at NEERI, Warud and Katol

Parameters*	SPE	Well Waters						
		N E E R I			Warud		Katol	
		1	2	3	1	2	1	2
pH	8.5	7.9	7.5	7.8	7.5	7.6	7.5	7.6
EC	0.70	0.94	1.1	1.05	0.75	0.72	0.56	0.56
Ca^{++} + Mg^{++}	6.5	5.84	4.25	3.99	4.7	4.4	4.8	4.7
K^+	0.25	0.015	0.009	0.005	0.01	0.001	0.005	0.007
Na^+	2.0	4.5	2.3	2.66	2.69	2.69	0.47	0.56
SAR**	1.10	2.63	5.14	5.78	1.75	1.80	0.30	0.36

* EC : Electrical conductivity in mmhos / cm .

** SAR : Sodium Adsorption Ratio = $\dfrac{Na^+}{\left\{(Ca^{++} + Mg^{++})/2\right\}^{\frac{1}{2}}}$

It is observed that the EC of well waters at NEERI and Warud is higher than that of the SPE. All the well waters contain only trace amounts of potassium, which is one of the essential plant nutrients. SPE contains moderate amounts of potassium. SPE was also analysed for other parameters that are found in well waters only in insignificant amounts. These parameters and their concentrations in SPE are : blue green algae - 2650 cells/ml, green algae 4250 cells/ml, total coliforms- 1.8×10^3/100 ml;

biochemical oxygen demand (BOD), total - N, -P, -Pb, -Cu, - Zn and - Ni - 105, 30, 14, 0.1, 0.2, 0.55 and 0.1 mg/l respectively. This means that SPE contains constituents such as nutrients, algae, organic matter and trace metals that may impart desirable biological properties to the soil (Elliott, 1977). The concentrations of BOD and some trace metals in the SPE are acceptable according to published guidelines (National Academy of Sciences, 1973). The SPE is thus comparable to or better than well waters at NEERI, Warud and Katol.

Soil Characteristics

Analysis of 1:2 soil-water extract, presented in Table 3 indicates that with regard to the EC, sodium, alkalinity and chloride contents, the soils on Field-I at NEERI are comparable with or better than some of the soils of orange farms at Warud and Katol. Additional characteristics of the soils on orange fields and other sites at NEERI were texture - silty loam to clay, exchangeable calcium - 52 meq/100 g, cation exchange capacity (CEC) - 55 meq/100 g, total N - 0.04 %, total P-0.08 %, organic matter - 1.9 % . The values for soil water contents at field capacity ranged from 17.4 to 31.8 % for Field-I, 30 to 33.5 % for Field II and 19.9 to 34.1 % for the road side soils. The values for water content in the soils at wilting ranged from 14.3 to 24.9 % for Field I, 15.3 to 30.1 % for Field-II and 13.1 to 22.4 % for the roadside soils. It is found that assessment of the soil water content at the stage of wilting simply by the 'feel' by hand as has been often done on many farms may not always be a correct indication of need for irrigation .

These results also indicate that soils on either side of the constructed drain are similar and amenable for studying effects of SPE irrigation and a general irrigation scheduling can be formulated.

TABLE 3 Characteristics of 1:2 soil-water extract for soils from fields at NEERI, Warud and Katol

Parameters*	Sites**				
	NEERI Field I	Warud Farms		Katol Farms	
		D-I	D-II	T	State Govt. Farm
pH	8.0	8.0	7.7	7.6	7.9
EC	0.25	0.34	0.24	0.22	0.25
Ca^{++} + Mg^{++}	0.25	0.26	0.21	0.32	0.35
Na^{+}	0.07	0.33	0.43	0.07	0.08
K^{+}	0.003	0.03	0.018	0.025	0.03
Alkalinity	0.47	0.70	0.40	0.50	0.55
Cl^{-}	0.06	0.035	0.05	0.035	0.05

* EC in mmhos / cm and other parameters in meq/100 g soil .

** D-I, D-II and T are used for identity of the farm owners .

Scheduling of irrigation for orange plants at NEERI, Nagpur .

Based on the water retention properties of the soils on various sites at NEERI, the water contents in the soil at the field capacity and wilting were taken as 30 and 22 % respectively for the purpose of irrigation scheduling. The corresponding values for water quantities work out to be 150 mm and 110 mm respectively for 50 cms of the soil depth, so that, every time after irrigation, 40 mm water is subjected to evapotranspiration. With these considerations, the scheduling of irrigation was designed for orange plants and presented in Table 4 .

TABLE 4 Scheduling of irrigation for orange plants ,NEERI,Nagpur.

Month	ETo ; mm/day	ET-crop mm/day (ETo x 0.6)	Total monthly ET-crop mm.	Irrigation interval in days 40÷ET-crop mm/day
January	4.9	2.94	91.14	13
February	5.98	3.58	103.82	11
March	8.67	5.20	161.20	8
April	9.8	5.88	176.40	7
May	12.86	7.71	239.01	5
June	8.26	4.95	148.5	8
July	4.26	2.55	79.05	15
August	4.05	2.43	75.33	16
September	4.45	2.90	87.00	14
October	4.84	2.90	89.9	14
November	4.47	2.68	80.40	11
December	4.35	2.61	80.91	15
			Total ...1412.66	

During the initial period of about one year, irrigation was given when the soil water content, as indicated by tensiometers and determined gravimetrically, was close to the wilting percentage. Though measurement of exact quantity of water given to each plant was not done, the pump and irrigation efficiency were considered to approximate the quantity in confirmation with designed irrigation scheduling. Gradually, the irrigation pattern was regulated with experience. It may be pointed out here that Nagpur receives moderate to heavy rains during the period July to October and irrigation is required only for the dry spell in these four months. Also, the root depth, which depends upon plant type, its age and the soil physical condition, has been assumed to be 50 cms in the present study. The evapotranspiration loss of 40 mm is based on individual discretion and does not indicate any generalization for Nagpur region.

<u>Leaf water extract and succulence characteristics</u>

Since fruiting on the orange plants was not regulated, physiological studies were restricted to the determination of leaf succulence and analysis of leaf extract. The results are presented in Table 5.

TABLE 5 Effect of SPE irrigation on the characteristics of the leaf-water extract and succulence

Type of irrigation PE (Pond effluent and WW(well-water)	Analysis of water extract of leaves: EC mmhos/cm	Na^+ (mg/g fresh wt.of leaves)	K^+ (mg/g fresh wt.of leaves)	Ca^{++} (mg/g fresh wt.of leaves)	Leaf succulence (% water in fresh leaves)
PE - 1 ...	0.66	0.18	2.52	0.20	63.9
PE - 2 ...	0.70	0.17	1.15	0.29	61.9
PE - 3 ...	0.84	0.17	2.65	0.72	61.8
PE - 4 ...	0.56	0.11	1.99	0.32	66.1
PE - 5 ...	0.85	0.21	2.46	0.53	60.8
WW - 1 ...	0.54	0.26	0.89	0.21	57.8
WW - 2 ...	0.57	0.48	1.6	0.21	59.3
WW - 3 ...	0.74	0.56	1.7	0.25	58.3
WW - 4 ...	0.64	0.32	0.89	0.29	60.7
WW - 5 ...	0.64	0.36	1.15	0.42	59.1

The SPE irrigated plants generally exhibit more succulence than the well water irrigated plants. This may be because of the higher EC of well waters resulting in higher osmotic concentration in the soil solution and reduced absorption of nutrients and water. The relative absorption of the individual ions is also influenced by SPE irrigation; sodium is considerably more in the leaf extracts of well water irrigated plants, potassium is more in those of the effluent irrigated plants, while calcium absorption is not affected.

Performance of the avenue, forest and garden plants

Based on the soil characteristics of the Field III and the area around the pilot wastewater treatment plants, the suitability of SPE irrigation for avenue, forest and garden plants was evaluated. The results are presented in Table 6 . With this experience, it should be possible to select a suitable plant species for effective SPE irrigation for a given site .

TABLE 6 Suitability of some of the avenue, forest and garden plants for SPE irrigation under specific soil conditions

Soil conditions; 1:2 Soil-Water extract	Plant species in descending order of suitability
pH : 7.5 to 8.4 EC (mmhos/cm): 0.3 to 0.6 Soluble sodium (meq/100 g soil): 0.2 to 1.0	*Cassia fistula* Linn, *Nyctanthes arbor-tristis* L., *Gardenia florida* L. *Citrus reticulata* Blanco *Citrus aurantifolia* (Christm) Swingle
pH : 8.5 to 9.0 EC : 0.51 to 1.0 Soluble sodium : 1.0 to 2.0	*Terminalia catappa* Linn, *Lagerstroemia indica* L . *Bauhinia purpurea* L . *Tabernaemontana coronaria* R.Br. *Jasminum sambac* (Linn.) Ait, *Mussaenda frondosa* Linn .
pH : 9.1 to 9.8 EC : 1.0 to 2.0 Soluble Sodium : 2.0 to 3.5	*Enterolobium sanam* Prain., *Syzygium cuminii* Linn. Skeels *Lagerstroemia speciosa* Pers *Cestrum diurnum* Linn . *Tecoma stans* (Linn.) H.B. & K. *Casuarina equisatifolia* Linn. *Mimusops elengi* Linn . *Leucaena glauca* Benth *Cassia siamea* Lam .

The area, around the pilot wastewater treatment plants has now developed into a good garden, there are suitable species of forest, avenue and ornamental flowering plants along the outer border and at the inside corners. In the interior, there are two rectangular lawns bordered by Roses and Dahelias in different colours and sizes with small narrow beds of seasonal flowering plants of attractive colours. Arches of *Bougainvelia*, *Polyalthia longifolia* along the small paths, additional circular beds of seasonals including *Canna*, with *Ixora*, *Lagerstroemia indica* and *Mussaenda* - all this is a treat and aesthetic tonic to the treatment plant operators and visitors .

CONCLUDING REMARKS

Considerable time was spent in making proper arrangements for on spot availability of the pond effluent before initiation of the detailed studies. The available well waters are neither good in quality nor adequate in quantity. In the initial stages, there were gaps in irrigation on account of practical problems. Therefore, the investigations have taken a natural course with emphasis given to the water retention properties of the soil and irrigation scheduling aimed at securing maximum plant survival. In most of the tropical countries, wastewater treatment in stabilization ponds followed by land application is likely to be practically possible. Being an economically feasible way of protecting the water supplies against pollution, detailed studies on utilization of pond effluent resulting in long-term benefits of conservation and recycling deservs to be carried out. The work contributed for this publication is expected to help in planning such studies.

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CLOGGING PHENOMENON IN IRRIGATION SYSTEMS REUSING POND EFFLUENTS AND ITS PREVENTION

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ABSTRACT

The clogging of emitters, orifices or laterals has turned out to be the main obstacle for further use of low rate applicators in wastewater irrigation. Granular filtration and screen filtration were studied and compared for particulate removal using small-scale pitot experiments. The removal ratio of particles larger than 10 μm in direct granular filtration was relatively large while smaller particles were hardly removed. Particles in the 10-60 μm size range were removed by 40-50 percent in depth and by 80 percent when surface filtration prevailed. Screen filters removed only 1-2 percent of the material but this was sufficient to cause their clogging. A release of particles from the screen into the effluent was observed, resulting in greater number of larger particles and thus greater clogging potential down the irrigation line. Deep-bed granular coarse media filters may serve for the control of particle size in effluents of oxidation ponds - deep reservoir systems while the conventional strainers are less recommended for this purpose.

KEYWORDS

Trickle irrigation; water reuse; filtration; oxidation ponds; particle analysis; wastewater treatment.

INTRODUCTION

The design and manufacture of filtration equipment for low rate application, and particularly for drip irrigation systems, has lately become an integral part of the water systems industry. The reason for this development is that the clogging of orifices, holes or lateral lines of the systems has turned out to be the main obstacle for further use of low rate applicators in irrigation. The problem is becoming even worse now, mainly in arid and semi-arid countries, as a result of increasing exploitation of marginal waters of secondary quality such as wastewater effluents from oxidation ponds and seasonal effluent reservoirs.

Research work and field observations concerning the performance of drip irrigation systems, utilizing wastewater effluents, indicate that the causes of clogging of low rate applicators may be divided into three main categories: a) suspended matter, b) chemical precipitation, and c) bacterial growth. Suspended matter seems, in most cases, to be the major cause of clogging (Adin, 1987).

Different types of filters (no chemical pretreatment) are currently being used to cope with the clogging problem. Most of them are based on straining mechanisms where filter (strainer) pores are smaller than most of the particles to be strained. This differentiates them from granular filters, which operate on the principle of in-depth filtration. The two types of filters were studied in this investigation.

This paper presents and discusses experimental results of granular filtration versus screen filtration of wastewater effluents used in an oxidation ponds - deep reservoir - trickle irrigation system.

EXPERIMENTAL SYSTEM

The experimental equipment for the pilot scale filtration studies was comprised of a feeding system and filter columns. The effluents were poured into a feeding tank in which they were pumped by a small centrifugal pump to a constant head tank to maintain an available head of 2.0 m above the filter beds. Three 55 mm diameter plexiglas columns were operated simultaneously so that studies could be carried out on the same feed. The transport columns allowed visual observation of suspended solids accumulation in the bed during filtration experiments. The filters were operated at a constant flow rate in the conventional downward direction. The flow rate, measured by a flowmeter, was adjusted manually by a valve connected to the outlet of each filter column. Head-loss development was measured using piesometer tubes connected to the outlet and inlet of each filter. The filter columns were 0.6 m long and the bed was restricted to 0.45 m to allow room for expansion during backwash. When screen filtration experiments were conducted, the filter columns were replaced by 45 mm diameter screen filter devices. The granular media were uniform local sands of 0.70, 0.84 and 1.20 mm effective grain size with uniformity coefficients of 1.28, 1.23 and 1.21, respectively. The screen filters were of common 80 and 130 μm polyester media.

HIAC/ROYCO PC-320, multi-channel particle size analyzer, was used for measuring particle size distribution, applying standard procedures for wastewater effluents previously developed in this laboratory. HIAC sensors operate on the light blockage principle and are commonly able to measure particles larger than 1 μm. Particles counts were carried out using 1-60 μm and occasionally 5-300 μm HIAC sensors. In the range of 1-26 μm measurements were taken in intervals of 1 um, while the rest up to 60 μm was taken as one interval. Influent and filtrate quality were also monitored by turbidity measurements with HACH 2100A turbidimeter and by gravimetric suspended solids determinations.

Effluents from an open deep reservoir (Naan Reservoir) storing stabilization ponds effluents were used in this investigation (effluent storing in open, deep reservoirs are commonly used in Israel to enable collection of effluents during the winter for reutilization in the dry summer season).

GRANULAR FILTRATION RESULTS

The effect of filtration rate on head-loss development for a 0.7 mm shallow (150 mm) sand bed is shown in Fig. 1. The head-loss curves show the characteristic exponential form, indicating that surface removal predominated. Water production per unit area decreased with the increase of the filtration rate, due to rapid head-loss build-up. The raw water contained a few very large particles, clearly observed through the transparent column, which quickly clogged the media surface forming a dark layer. Filter runs were short due to the rapid head-loss development: 80 minutes for the low filtration rate ($2.2\ l/m^2/s$) and only 30 minutes for the higher rate ($4.4\ l/m^2/s$).

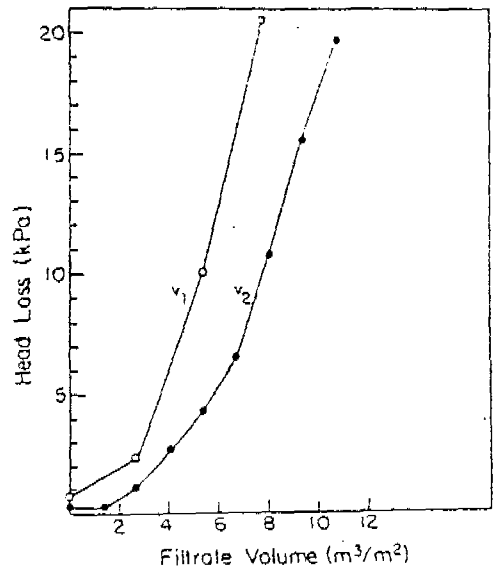

Fig. 1. Head-loss development during filtration of wastewater reservoir effluents for different rates.
v_1=2.2 l/m^2/s; v_2=4.4 l/m^2/s.
Effective grain size = 0.7 mm.
TSS of incoming effluent = 104 mg/l.

The effect of filtration rate on total suspended solids and particles removal was also studied, using gravimetric and PSD measurements in influent and filtrate. Fig. 2 presents particle removal efficiency as a function of particle size for the filtration rates mentioned above. The results show better removal efficiency of particles in the lower filtration rate for all particles sizes. The difference in percentage becomes smaller as particle size gets larger. A minimum removal efficiency was observed in the size range of 1-2 µm, while the removal efficiency of larger particles increased significantly up to 9-10 µm. An instantaneous phenomenon of more particles in the filtrate than in the effluent was observed in the higher filtration rate in the size range of 1-3 um. Particles larger than 10 µm were removed with a constant removal efficiency of about 80% most probably by the layer formed on the medium surface.

Although significant removal of large size particles was noted, the removal efficiency in terms of suspended solids concentration was very low, about 20% for 2.2 l/m^2/s and only 10% for 4.4 l/m^2/s filtration rate due to massive breakthrough of the fine particles.

From a certain date of the experiments the very large filter clogging particles in the reservoir effluent, i.e. the filter influent, were not observed. The filter runs were very long and no significant head-loss was developed across the filters. The removal efficiency in terms of TSS was very low, about 10%. No change in turbidity was observed pointing again at the colloidal nature of the raw water. The effect of filtration rate increase on particle removal as a function of particle size resulted in removal efficiency decrease. The effect of media grain size on particulate removal when no clogging phenomena were observed was interesting: larger grain sizes clearly showed better removal efficiency of very fine particles (1-10 µm). For particles larger than 10 µm (up to 60 um) there was no difference in the rate of particle removal. The percentages of particles removed in the above experiments were much lower than in the experiments where clogging of the media surface occurred throughout the whole size range suggesting that the surface filter cake increased the removal efficiency. On the other hand, filter runs here were longer and, therefore, more economical in this respect than those observed while surface removal predominated.

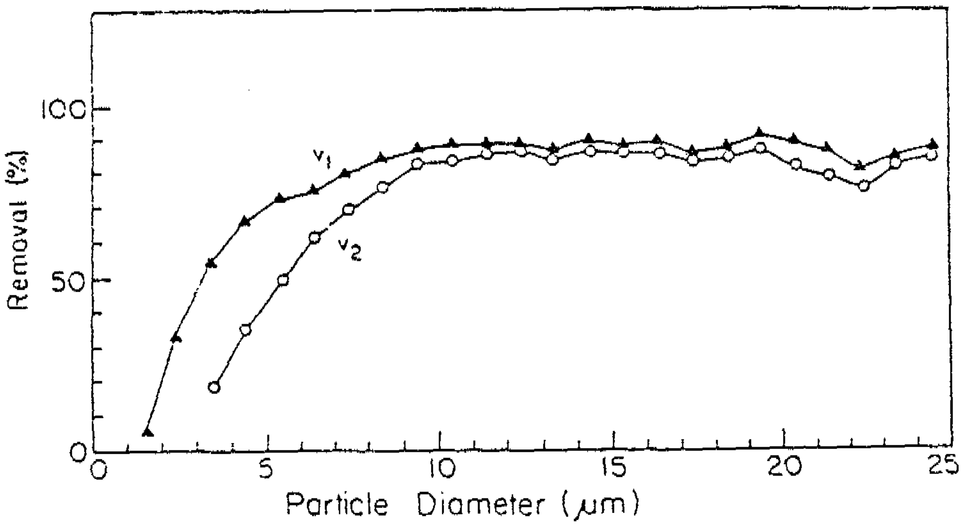

Fig. 2. Removal ratio versus particle size for different filtration rates.
Wastewater reservoir effluents:
v_1=2.2 l/m^2/s; v_2=4.4 l/m^2/s.
Effective grain size = 0.7 mm.
TSS of incoming effluent = 104 mg/l.

SCREEN FILTRATION RESULTS

Screen filters with 130 μm polyester media were clogged after a short period of time (1/2 hr) with a filtration rate of 6.6 l/m^2/s. The cause of the immediate clogging was the very large particles which formed a compressed cake on the screen. TSS removal by the 130 μm screen was no more than 2%. A phenomenon of more particles leaving the screen in the size range of 15-20 μm was observed, pointing out at a possible detachment mechanism as previously reported by Adin and Alon (1986). In the latter work, particles were counted in two separate analyses of 100 mg/l algal suspensions and the results were compared with counts of the particles which remained in the filtrates of two different screens. The particle dimensions ranged from less than 1 μm up to 60 μm. The results are depicted in Figures 3 and 4. For the 80 μm screen 1300 particles per mm^3 were counted before filtration and 1185 particles per mm^3 were counted in the filtrate. For the 130 μm screen, 1245 particles per mm^3 were counted before filtration, and 1710 particles per mm^3 were counted in the filtrate. As can be observed, the particles in the filtrate in certain size ranges outnumbered the particles in the influent.

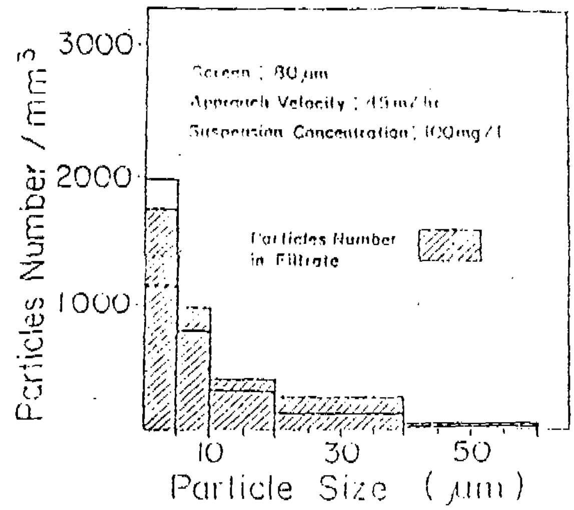

Fig. 3. Particle size range in influent and effluent - narrow screen aperture.

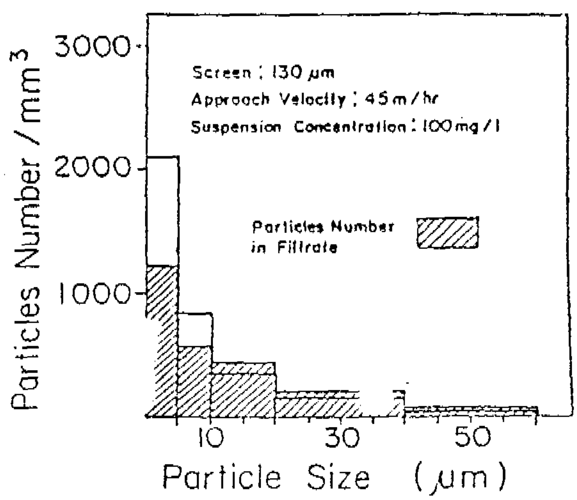

Fig. 4. Particle size range in influent and effluent - wide screen aperture.

DISCUSSION

The comparison of reservoir particle removal efficiency as a function of particle size for different filtration rates resulted in lower efficiency for the higher rate, probably due to the higher hydrodynamic shear forces developed in the bed, affecting more the smaller particle range. The tendency of the removal curves toward a minimum efficiency in the 1-2 µm range (Fig. 2) supports the experimental and theoretical studies of other investigators (Habibian and O'Melia, 1975; Fitzpatrick and Swanson, 1980) which point out a minimum particle transport efficiency in between the range where gravity mechanisms and diffusion mechanisms dominate. But, it is worthwhile to mention that no information based on direct particle size measurements was gathered about the behavior of submicron particles in the filter bed.

The instantaneous phenomenon occasionally observed of more particles in the filtrate than in the original filter influent may be also attributable to the sheer forces developed in the bed causing the breaking of aggregates and colonies of algae into single cells or the rupture of small particles from biological flocs. Sheer forces within the filter bed may also explain the clear observation of larger grain sizes removing fine particles better than small grain sizes for both types of effluents, although the larger specific surface of the latter. This may be attributed to the lower sheer forces due to lower interstitial velocity, caused by the smaller reduction of the larger pores, enabling more effective attachment even for very fine particles to the deposits accumulated around media grains.

Microscopic analysis confirms the formation of larger particles in the filtrate of screens. The phenomenon can be explained by recognizing the dominance of a detachment mechanism. As a result of the dominance of this mechanism, aggregates are forced through the screen to appear in the filtrate, thus forming pores in the filtrate cake.

It is worth noting that the detachment phenomenon may cause clogging in drip irrigation system components which are placed after the filters, and in the drippers themselves. Evidence for this was received from farmers and others who attempted to evaluate the reasons for clogging by field observations.

CONCLUSIONS

1. Deep-bed granular media filters may be used for the control of particle size in effluents of an oxidation ponds - deep reservoir system. The particulates removal efficiency through the beds increased with filter grain size and depth and decreased with filtration velocity, affecting more the lower particle size range.

2. The removal ratio of particles larger than 10 μm in direct granular effluent filtration is relatively large while smaller particles are hardly removed. A minimum removal efficiency exists in the 1-2 μm size range. Particles in the 10-60 μm size range were removed by 40-50 percent in "pure" in-depth filtration and by more than 80 percent when surface filtration conditions were developed.

3. Screen filters performed very poorly - about one percent removal efficiency. This one percent was sufficient to cause surface clogging. The release of particles from the screen into the effluent during the formation of a filter cake resulted in particle size distribution and counts different than those observed in the influent. The greater number of larger particles in the effluent might increase clogging down the irrigation line, and in the various components of the system that should be protected by the filters.

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POSTER PAPERS

FRENCH EXPERIENCE IN THE OPERATION AND MAINTENANCE
OF WASTEWATER TREATMENT LAGOONS

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ABSTRACT

A general survey of maintenance requirements and problems of wastewater lagoons has been conducted in France with a view to preparing an operation manual for plant owners and local operators. An outline of the pond maintenance situation in France is given, and the main information and recommandations contained in the manual are presented in this paper.

KEYWORDS

Wastewater, wastewater stabilization ponds, lagooning, operation, maintenance, operation manual.

PRESENTATION

Over 1400 lagoon systems are presently operated in France. Nearly all of them were built after 1978. 90 % of the plants in operation are used for the treatment of wastewater in small rural communities (less than 2000 inhabitants). Such a rapid development of lagooning among small communities is largely due to the relatively cheap and simple operating procedure involved : Basically, lagoons operate without any energy input, comprise no electromechanical devices and do not require specialized technicians for routine maintenance. The technique is therefore usually well suited to the educational level of locally available operators. For instance unskilled municipal workers assigned to the ponds maintenance, in addition to other tasks, would be suitable in the case of a small plant.

In this connection, it appeared important to provide plant owners, operators and local services in charge of technical assistance, with information on "good practice" in pond management. The preparation of an operation manual was undertaken in 1984, based on a general survey of the maintenance requirements and problems of lagoons. The manual was published in 1985 (Ministère de l'Agriculture, 1985). Information presented in the following chapters refers to this work. The proper maintenance of lagoons includes actions at various levels:

- appropriate design of ponds
- regular routine maintenance
- occasional desludging operations
- prevention and control of possible troubles in ponds.

POND DESIGN AND MAINTENANCE TASKS

Maintenance tasks should always be born in mind when planning and designing ponds : it is for instance most important that WSP projects should include wide enough access for engines around the ponds, easily get-at-able pretreatment units, flow measurement facilities for both influent and effluent, and so on... When the lagoon system includes a pond planted with rooted macrophytes, the pond shape and plants localization should be designed, taking into account maintenance constraints.

ROUTINE MAINTENANCE TASKS

These chiefly consist in cleaning the pretreatment units (usually reduced to a simple barscreen) and in the upkeep of the pond surroundings (control of vegetation on dike and accessways). It is recommended that pretreatment units be designed to need only one weekly cleaning operation. Routine maintenance also includes a control of water flow and water level in ponds, observations on the presence of floatting matter or vegetation, on dikes condition (notice local damages, presence of burrowing animals) and on abnormal odours.

Observations should be reported on standard forms and appropriate control actions should be undertaken, in connection, if needed, with the local technical assistance department (SATESE).

Routine maintenance tasks are summarized in table 1, which also indicates the approximate time needed, for an "average" French waste treatment lagoon, i.e serving a population of around 600 inhabitants.

TABLE 1 : Summary of routine maintenance tasks

Task	Frequency	Time needed	nb of operators	total nb of man-days/year
General checking and pretreatment cleaning	1/week	1 h.	1	7
Control of vegetation around ponds	4/year	1 d.	1	4
Control of dike vegetation (inner face)	2/year	1 d.	1	2
Various (small repairs, removal of floatting, materials,...)				5
Cutting of macrophytes in macrophytes ponds	1/year	1 d.	2	2

The use of macrophytes ponds entails additional maintenance (2 days per year). Including this, the total number of man days required amounts to approximately 20 days per year. The corresponding maintenance cost, including the small equipment needed is about 15FF/inhab. eq./year (1985 prices).

OCCASIONNAL DESLUDGING

Sludges accumulate in ponds : an average of 1 to 2 cm/year in the primary lagoon. No lagoon will operate indefinitly without occasional desludging. However, this can be done with very low frequency.

Usually, sludge management will consist in :

- Timely sounding of sludge thickness in the various parts of the ponds.

- Suppression of major sludge deposits, which usually accumulate near the inlet, or in some corner of the primary lagoon. This has to be done once every 1 to 5 years according to the plant operating conditions (shape of the ponds, type of network) and does not necessitate emptying the lagoons (simple pumping with a vacuum tank).

- General desludging with total emptying of the ponds. This is a heavy but very low frequency operation (not enough reference cases yet available in France ; perhaps once every 10 years). It is important to plan these works with consideration to possible sites for sludge disposal (application to agricultural land is recommended).

PREVENTION AND CONTROL OF TROUBLES IN PONDS

Disfunctionning may occur in ponds, and regular maintenance will contribute towards early detection, and consequently, easier control. This implies however that the operator is informed of how to detect disfunctioning, and what to do. Such information has therfore been included in the operation manual, for the most frequent troubles likely to impair pond efficiency, which are :

- Water level going down, due to defective tightness or over-sizing of ponds

- Development of anaerobic conditions in algal ponds resulting in odour problems

- Colonization of algal ponds by aquatic vegetation (either rooted or floating plants)

- Degradation of embankments by musk rats and other burrowing animals

- Mosquito breeding in ponds

- Algal blooms impairing the quality of the treated effluent

In most cases, these troubles can be avoided or minimized with appropriate preventive maintenance operations.

CONCLUSION

It clearly appears that pond maintenance should not be disregarded on the ground that it is a priori simple and cheap. Proper maintenance is essential to the permanent and reliable efficiency of lagoons. Therefore, both designers and plant owners should have a full knowledge of pond maintenance requirements.

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STABILIZATION PONDS IN SOUTH-WEST FRANCE: EFFLUENT QUALITY

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ABSTRACT

Long-term surveys were performed on three stabilization ponds. Experimental results concerning effluent quality are briefly presented and discussed.

KEYWORDS

Wastewater, biological treatment; wastewater stabilization ponds; lagooning; effluent quality; France.

PRESENTATION OF THE SURVEYS

In South-West France, long term surveys were rarely performed on small "natural" wastewater stabilization ponds. We present here data collected from three plants, typical of what is usually encountered in rural areas. Plant characteristics are summarized by Table 1.

TABLE 1: Plant Characteristics

Place	Department	Design capacity (pers.)	Sewerage system (un./sep.)	Imhoff tank (Yes/No)	Total area (sq.m)	Number of basins	Water depth (m)
COURLAY	2-Sèvres	500	Unitary	NO	5000	3	1-1.6
FENERY	2-Sèvres	200	Part. sep.	YES	1600	4	1.0
HOSTENS	Gironde	350	Separated	YES	2500	1	1.4

In Courlay and Fenery, the surveys were performed simultaneously. As they exceeded one year, the first 12 months only were considered for the statistical interpretation of results. On each plant, the organic load is about 40 kgBOD/ha.d (design load: 50 kg/ha.d).

EXPERIMEMENTAL RESULTS

Some representative environmental data and experimental results appear on figures 1 to 7 and on Table 2. For more consistency, they mainly concern one plant only (Courlay).

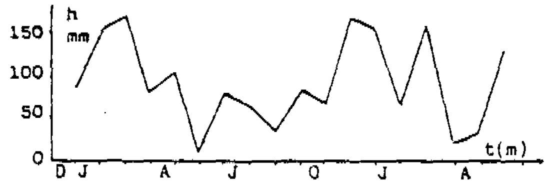

Fig. 1. Courlay: monthly rainfall

Fig 2. Courlay: Water temperature

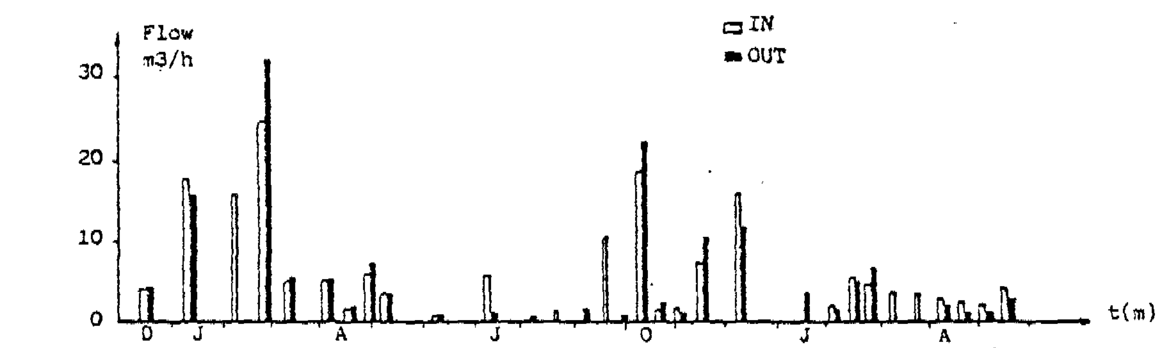

Fig. 3. Courlay: Measured flows of raw (in) and treated (out) effluent

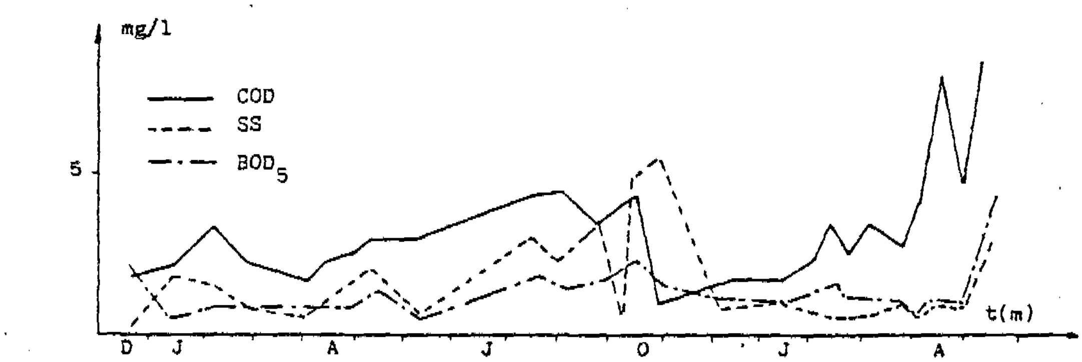

Fig. 4. Courlay: Treated effluent SS and (soluble) COD and BOD

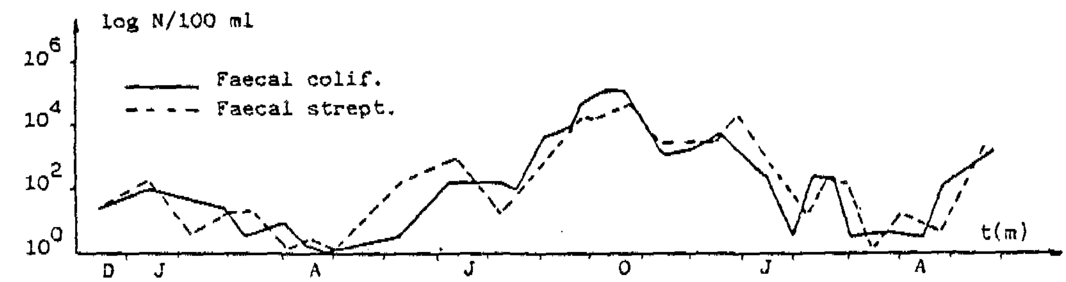

Fig.5. Courlay: Treated effluent "faecal" coliforms and streptococci (log. scale)

TABLE 2: Bacteriology of Effluents from Courlay and Fénery Ponds (nb. of organisms/100 ml, computed from a log-normal distribution)

	COURLAY						FENERY					
	SUMMER			WINTER			SUMMER			WINTER		
	C37	C44	SF	C37	C44	FS	C37	C44	SF	C37	C44	FS
Median value	250	30	30	22000	2500	1800	15	4	5	860	150	50
N_{90}	11000	1500	1900	1.6. 10	1.6. 10	4.3. 10	130	37	62	26000	6000	2100

C37 = "total" coliforms; C44 = "faecal" coliforms; FS = "faecal" streptococci.

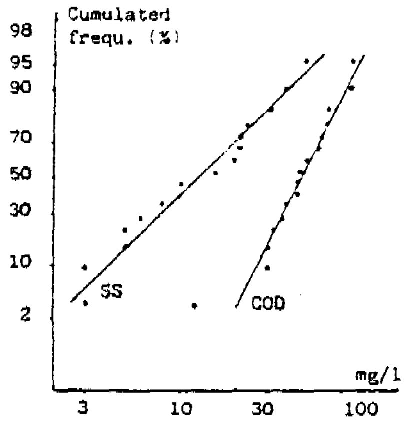

Fig.6. Courlay: Statistical distribution of effluent SS and COD

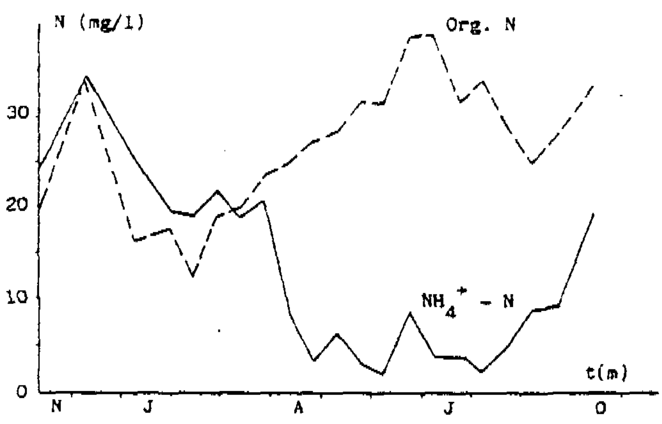

Fig. 7. Hostens: Unfiltered effluent ammonium nitrogen and organic nitrogen

MAIN CONCLUSIONS

1 - The poor results from 1-basin systems are confirmed (the Hostens pond is now completed to 3 basins).
2 - Hydraulical overloading may appear in Winter in the case of unitary or defective separated sewerage systems. It results in reduced residence times.
3 - Seasonal variations of effluent quality are mainly related here to algal pullulation. They concern above all suspended solids (higher in Summer) and nitrogen (inversion of the ammonium/org.N ratio in April-May and Sept.-Oct.). Effluent total N, soluble COD and BOD show less changes.
4 - Because of unwanted waters entering sewers, the flow of pollutants leaving the pond system is usually higher during the Winter period than in Summer.
5 - Effluent median values for 3-4 basin systems are around 15-20 mg/l for SS and 40-50 mg/l for soluble COD. SS and COD less than 50-60 and 80-90 mg/l respectively are found from 90% of samples. Values over 100 mg/l for both parameters are exceptional.
6 - Bacterial concentrations in the effluent widely vary all over the year, with a range of 5-6 decimal log-units (DLU) for coliforms in Courlay (3 ponds, hydraulic overload), 3-4 DLU in Fénery (4 ponds), 2 DLU in Hostens (1 pond). Seasonal differences are about 1.5-2 DLU for the 3-4 pond systems. Poor results concerning bacterial elimination are obtained from the 1 basin plant.
7 - In Summer, effluents from 3 and 4 basin plants meet the Engelberg criteria for an unrestricted reuse of water by irrigation. They are below limits for bathing waters.

ACKNOWLEDGEMENTS

The studies were granted by the **Etablissement Public Régional Poitou-Charentes** and the **Conseil Général de la Gironde.**

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Smith 1

SEWAGE STABILISATION PONDS IN
ARABIA AND KENYA

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ABSTRACT

Experience of pond design and operation in Arabia and Kenya is illustrated and described.

In Saudi Arabia major stabilisation pond installations were built for the Three Cities Project at Hofuf, Buraidah and Qatif. Features of the inlet works, embankment design and operational facilities are illustrated by photographs and drawings. Summaries of results, comment on performance achieved and implications for future stabilisation pond design are noted.

Recently commissioned ponds in Aden for both domestic and industrial sewages are featured and points of interest indicated.

The basis of the most recent designs now ready for tender for applications in Aden and Seiyun, P.D.R. Yemen are provided.

Conclusions are drawn regarding the future potential of pond treatment for disposal of waste water for re-use in irrigation.

KEYWORDS

Stabilisation ponds; anaerobic ponds; facultative ponds; maturation ponds; Arabia; Aden, Yemen; Kenya.

APPLICATION AND ADVANTAGES OF PONDS

Sewage stabilisation ponds are an appropriate form of sewage treatment in regions where:-

1. Large areas of land are available at low cost.
2. There is high incidence of sunlight.
3. There are minimal facilities for maintenance of complex mechanical equipment and process systems.

The particular advantages of pond systems in such situations are:-

1. Low capital and operating cost.
2. Simplicity of operation and maintenance.
3. Constant availability of treatment.
4. Good pathogen removal.

The pond systems illustrated by photographs and drawings and detailed below in Table 1 are designed for situations in which the above criteria apply and those plants which are in operation have fulfilled their objectives with the advantages enumerated.

TABLE 1 Stabilisation Pond Systems designed by John Taylor & Sons

LOCATION (date commissioned)	Population served x 1000	Design Daily Flow Ml/d	No. and Type of Ponds	Influent BOD kg/day	Total Volume of Ponds Ml	Total Area of Ponds ha.	Loading kg BOD /day	Retention days
SAUDI ARABIA								
Buraidah (1980)	60	10.9	2 No. Facult. Irregular - in parallel.	3240	155	11.5	282/ha	14.2
			2 No. Maturation Irregular - in parallel.	-	78	5.9	-	7.1
Hofuf (1978)	130	29.5	2 No. Facult. 640mx264mx1.25m	7020	422	33.8	208/ha	14.3
			2 No. Maturation 320mx264mx1.25m In parallel.	-	211	16.9	-	7.2
Qatif (1982)	20	4.5	2 No. Facult. 165mx158mx1.25m In parallel.	1080	65	5.2	208/ha	14.4
			2 No. Maturation 165mx80mx1.25m In parallel.	-	33	2.6	-	7.3
P.D.R. YEMEN								
Sheikh Othman, Aden. (1985)	139	17.4	2 No. Anaerobic Irregular - in parallel.	6255	23.4	0.78	0.27/m³	1.35
			4 No. Facult. Irregular - in parallel.	3753	119	9.5	395/ha	6.8
			4 No.Sec.Facult. Irregular - in parallel.		100	8.0	214/ha Total	5.7
Ga'ar (not yet constructed)	16	2.0	1 No. Anaerobic 40mx24mx3m	720	2.9	0.10	0.25/m³	1.45
			1 No. Facult. 130mx65mx1.25m	432	10.6	0.85	508/ha	5.3
			1 No.Sec.Facult. 130mx65mx1.25m		10.6	0.85	254/ha	5.3
Zingibar (not yet constructed)	12	1.5	1 No. Anaerobic 33mx19mx3m	540	1.9	0.06	0.28/m³	1.27
			1 No. Facult. 125mx55mx1.25m	324	8.6	0.69	470/ha	5.7
			1 No.Sec.Facult. 125mx55mx1.25m		8.6	0.69	235/ha Total	5.7
Seiyun (not yet constructed)	21	2.5	2 No. Anaerobic 32.5mx32.5mx3m	945	6.3	0.21	0.15/m³	2.5
			1 No. Facult. 164mx84mx1.25m	378	17.2	1.38	201/ha	6.9
			3 No. Maturation 61mx84mx1.25m	-	19.2	1.54	-	7.7

TABLE 1a Stabilisation Pond Systems designed by John Taylor & Sons

LOCATION (date commissioned)	Population served x 1000	Design Daily Flow Ml/d	No. and Type of Ponds	Influent BOD kg/day	Total Volume of Ponds Ml	Total Area of Ponds ha.	Loading kg BOD /day	Retention days
KENYA								
Nyahururu (1986)	31	2.6	2 No. Aerated 48mx48mx4m	1395	18.4	0.23	0.076/ m^3	7.1
			1 No. Facult. 288mx59mx1.75m	395	29.7	1.70	232/ha	11.4
			1 No.Maturation 302mx57mx1.75m	134	30.1	1.72		11.6
Bura (operating)	7	1.26	1 No. Facult. 172mx44mx1.75m	385	13.2	0.76	507/ha	10.5
			2 No.Maturation 87mx44mx1.75m	66	6.7	0.38		5.3
Kisii (not yet constructed)	32	2.9 to 4.2 varies season-ally	4 No. Anaerobic 36mx36mx4m	1440	20.7	0.52	0.07/m^3	5.0 min.
			1 No. Facult. 130mx130mx2m	432	26.0	1.30	332/ha	6.2 min.
			2 No.Maturation 60mx78mx3m 88mx53mx3m		28.0	0.93		6.7 min

Another 5 pond schemes have been designed for Kenya over the past 10 years, 4 of which are in operation. These are much smaller with 2 for military establishments.

SELECTIVE DEPLOYMENT

It is important to recognise those situations in which stabilisation ponds are not on appropriate form of treatment and particular disadvantages in addition to the obviously large land take are:-

1. Loss of water by evaporation or seepage.
2. High algal content of effluent.
3. Uncertain and variable effluent quality.

Where sewage effluent is seen as an important resource for irrigation or other re-use ponds may, therefore, be unsuitable as witness the adoption of more sophisticated and compact treatments in many places in Arabia.

OPERATING RESULTS

As one of the important objectives of pond treatment is operational simplicity it follows that a great deal of analytical data is not normally available unless special surveys are funded by outside agencies.

Few results are available from the plants cited and these are set out (on Poster) but must be regarded with due reservation.

ACKNOWLEDGEMENTS

The Author wishes to thank, on behalf of John Taylor & Sons, Clients in the regions concerned, and East African Engineering Consultants who had overall responsibility for the projects in Kenya, for access to information.

ALGAL-BACTERIAL PONDING SYSTEMS FOR MUNICIPAL WASTEWATER TREATMENT IN ARID REGIONS

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ABSTRACT

A pilot plant was constructed to investigate the use of algal-bacterial systems to treat municipal wastewaters in arid regions. The pilot plant consists of sand and oil traps, weir tanks, two facultative ponds (250 m^3 holding capacity each), two high-rate ponds (25 m^3 holding capacity each), four sedimentation tanks, a sludge digestor, and such auxiliary facilities as a pumping station for wastewater supply, drainage systems, and a site laboratory. The main objectives of the study are to determine the design parameters needed for large-scale algal-bacterial ponds in hot climates and to determine the degree of treatment and disinfection attained in the system by operating the pilot plant. After the algae is separated from the treated water, the final effluent will be used for irrigation, and algae will be used as a soil conditioner.

Investigations during start-up, test runs and experiments completed so far focused on the growth of algae and removal of biochemical oxygen demand, chemical oxygen demand, and suspended solids. During the 20-month experimental program, various operating patterns, including series operation of facultative and high-rate ponds, will be investigated. Results obtained so far indicate significant reductions in pollution concentrations.

KEYWORDS

Algal-bacterial systems, wastewater treatment, wastewater reuse, facultative ponds, high-rate pond, pilot plant, algae removal.

INTRODUCTION

Algal-bacterial ponds are now widely used in the Arabian Gulf countries to treat wastewater because these ponding system have proved to be efficient and popular in hot climates. But data on the actual design and operation of the ponds under local conditions (weather, composition of the sewage, microbial strains, operating patterns) were lacking. Therefore, research was started to determine the design and operational parameters needed for large-scale algal-bacterial ponds in a hot climate, and to determine if the degree of treatment and disinfection of the wastewater attained in the system is sufficient so the water can be used for irrigation. Properly controlled pilot plant experiments were used to obtain reliable results. Typical municipal wastewater from the Kuwait City sewage system was chosen for the pilot plant experiments.

The ponding system will be based on the results of the research; it is expected to help pollution control if established conventional treatment plants are shut down, and to be used in communities not connected to the sewage treatment system. The harvested algae will be used for soil conditioning after they are separated from the treated effluent.

The experimental program to study the continuous operation of the plant under various operating patterns and weather conditions will last 20 months. Prior to the continuous running experiments, test runs and plant start-up were completed. So far, the facultative and high-rate ponds have been operated individually for five months and three experimental runs have been completed.

METHODOLOGY

The pilot plant system consists of oil and sand traps (2.5 m x 2 m x 1 m deep), two weir tanks to measure the flows into the units, two facultative ponds (each 10 m x 10 m surface and with variable depth from 0.5-2.5 m), two oval high-rate ponds (each 10 m x 5 m with variable depth from 0.15-0.9 m) equipped with paddle wheels, a digestor (2.5 m^3 holding capacity), a flow mixing and division box, and four sedimentation tanks (each 6 m x 1 m x 1.8 m deep). The piping system allows the following operating patterns: (a) individual operation of the facultative ponds; (b) series operation of the facultative ponds; (c) individual operation of the high-rate ponds; (d) series operation of the facultative and high-rate ponds in different combinations; (e) series operation of the digestor and the high-rate ponds.

The pilot plant is supplied with typical municipal wastewater with the grit removed. Plant loading can vary up to 10 l/s.

Simultaneously with the operation of this system, experiments can be performed to study algae strains and conditions for continuous maintenance of the algae culture in four separate 400 l containers equipped with aeration.

Pond performance is monitored continually by wastewater analyses and microbiological examinations carried out at on-site and central laboratories.

The raw sewage and the effluents from the four ponds are analyzed twice a week to determine biochemical oxygen demand (BOD_5), filtered and unfiltered chemical oxygen demand (COD), dissolved organic carbon (DOC), suspended solids (SS), volatile suspended solids, phosphate, sulfate, ammonia nitrogen, organic nitrogen, chloride, and alkalinity. In addition to these analyses, in-situ measurements were made twice a day to determine flow rate, water depth, temperature, pH, dissolved oxygen, and several atmospheric parameters. The microbiological examinations included daily counts of algae, total coliform, and total bacteria.

A test run of the pilot plant to determined the hydraulic parameters of the system, such as flow rates, velocity, and hydraulic efficiency. The start-up experiments focused on building up the biological system to prepare the units for efficient treatment. Experiments were run under different operating conditions with variable detention time, flow rate, water depth in ponds, weather conditions (all seasons), and paddle wheel operation.

RESULTS AND OBSERVATIONS

In these experiments, water depth varied between 1.5 and 2.0 m in the facultative ponds and between 0.30 and 0.45 m in the high-rate ponds. The detention time varied between 8 and 16 days for the facultative ponds and between 6 and 16 days for the high-rate ponds. During start-up, the ponds were filled to half the required depth with fresh water, and then raw sewage was fed at the test value of the flow rate. For the high-rate ponds, some algae culture from the 400 l containers was added as inoculum. Stable conditions were reached fairly rapidly, and the biological system was built up within two weeks.

The strength of the sewage in Kuwait was very high, i.e., the raw sewage BOD varied between 300 and 600 mg/l. This caused the washout of algae in the high-rate ponds in the winter months when detention time was eight days or less. The results of the wastewater analyses revealed that more than 60% BOD and more than 50% COD were removed. Relatively high values of unfiltered COD and suspended solids in the high-rate ponds are caused by algae, which can be removed by physical treatment methods. Although the phosphate and chloride concentrations did not change significantly in any of the ponds, sulfate concentrations were reduced by 50% in the facultative ponds, but increased by about 30% in the high-rate ponds. This is due to the oxidation of sulfide ions in the high-rate ponds and the conversion of sulfate to sulfide because of anaerobic conditions in facultative ponds. High-rate ponds were more effective in reducing the concentration of nitrogen compounds than the facultative ponds. Alkalinity was increased in the facultative ponds, but decreased in the high-rate ponds.

The results of the microbiological experiments indicate that, in all ponds, total bacterial counts were about 10% of that of the raw sewage. Similarly, a reduction of more than 95% in total coliform counts was observed. The main algal population in the facultative ponds is scenedesmus and chlamydomonas; while in the high-rate ponds, it is only scenedesmus.

CONCLUSIONS

The results of the completed experiments indicate that a ponding system is a viable alternative for wastewater treatment in arid regions. Series operation of facultative and high-rate ponds looks promising since the organic loads on the high-rate ponds will be significantly reduced; a 50-60% reduction in BOD and COD should be expected through each pond.

ACKNOWLEDGMENT

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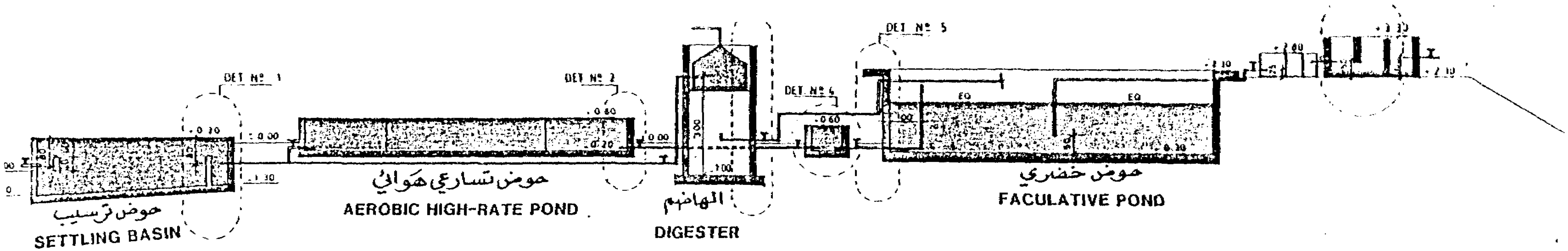

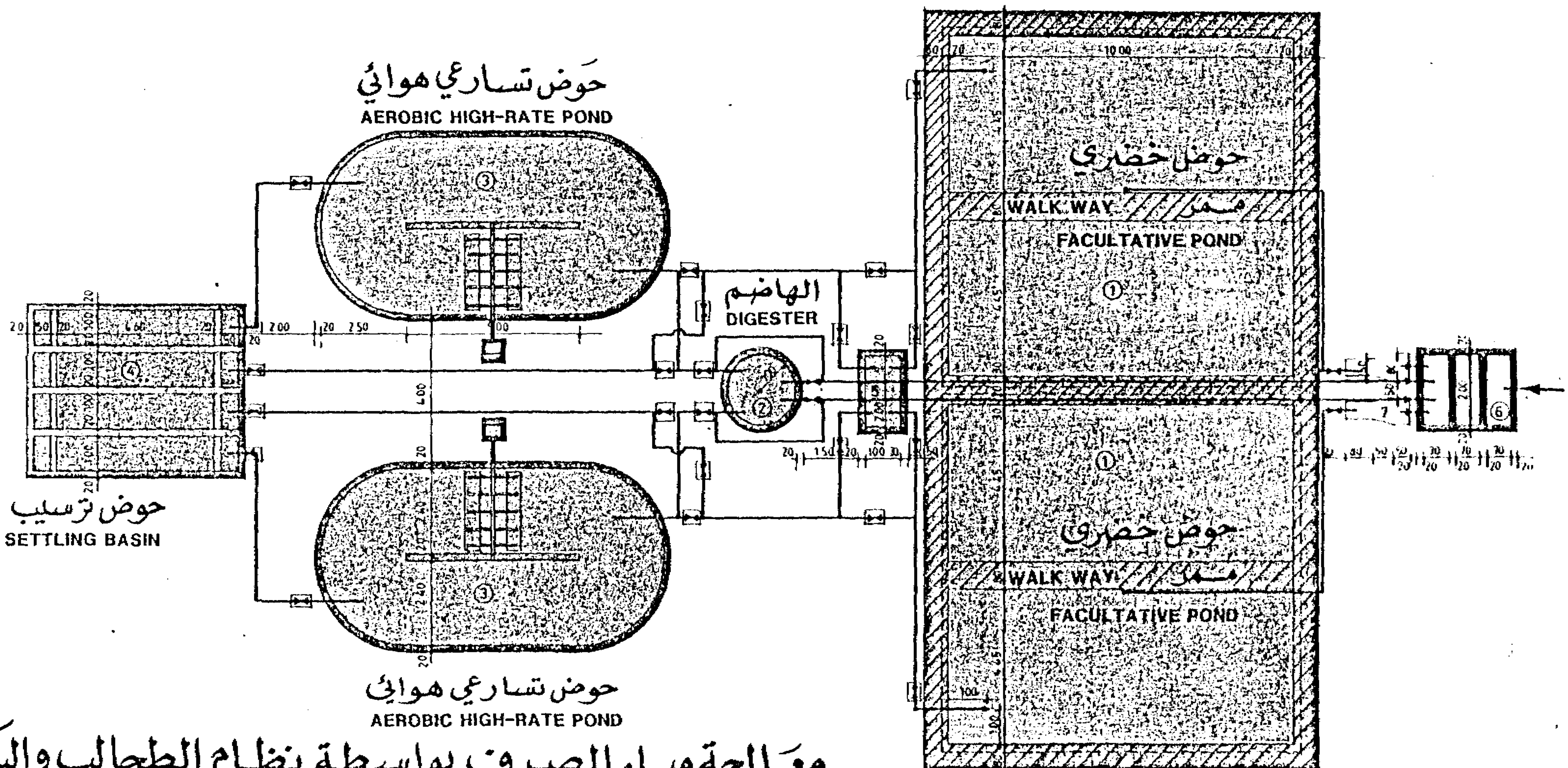

معالجة مياه الصرف بواسطة نظام الطحالب والبكتير

ASTE WATER

ENVIRONMENTAL IMPACT ASSESSMENT OF THE DISCHARGE OF THE EFFLUENT FROM FARO TREATMENT PLANT BY LAGOONING INTO RIA FORMOSA (ALGARVE)

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ABSTRACT

This paper intends to present the main problems studied in the environmental impact assessment of the discharge of the tertiary effluent from a stabilisation pond system to be built for the city of Faro (Algarve) into Ria Formosa, a shallow coastal water body of high ecological, economical and touristic value.
The environmental assessment emphasized the analysis of the impacts that may affect the shellfish beds as well as the flora and fauna, which are mainly the decrease of salinity, the increase of nutrients content and the afflux of pathogens.
Despite the impossibility of quantitative assessment, the study concluded that the effects of the variation of salinity and nutrients content will not be very significant, being the effect of the pathogens a possible source of contamination of the shellfish beds.
Recommendations are presented including an adequate construction and operation of the treatment plant, a tight sanitary control of the shellfish before human consumption and a monitoring of the future situation.

1. INTRODUCTION

The need to improve the water quality of Ria Formosa and to ameliorate the sanitary conditions of the population led the local authorities as well as the central departments concerned with these problems towards the development of a solution to the sewerage and sewage treatment works for the city of Faro.
The consequences of the discharge of the treated effluent into the Ria required special consideration, thus leading to the preparation of an environmental impact assessment together with the detailed design of the works, which include a stabilisation pond system for the sewage treatment.
This paper focuses on the main environmental problems that were analysed in that study, the conclusions of the impact assessment and the recommendations for future action.

2. GENERAL DESCRIPTION OF THE SEWAGE TREATMENT PLANT

The treatment plant under consideration is located 1.3 Km East of the city of Faro, in a wide flat area close to Ria Formosa.
The plant was designed to cope with a population of 87 145 inhabitants in peak season and 69 005 inhabitants in low season, at the year 2000, an expansion of it being foreseen at a later stage.
After preliminary treatment (screening and degritting) the works include three stabilisation ponds in series. The first pond is anaerobic, with an area of about 1 ha. The second pond is a facultative one and its area is about 14 ha. The last one is a maturation pond with 8 ha, being its main objective the reduction of faecal organisms of the sewage to ensure a safe discharge into the receiving water body. The final discarge to Ria Formosa is provided by a small outfall.
The effluent from the facultative pond is expected to have BOD_5 of 37 and 50 mg/L at peak season and low season, respectively, in the year 2000. The final effluent, in the

year, is expected to contain 495 and 5 000 faecal coliforms per 100 mL in the same seasons.

3. RELEVANT CHARACTERISTICS OF THE RECEIVING WATER BODY

Ria Formosa is a natural reserve enclosing an extensive shallow water body and a large area of marshes, both vital to several species of birds coming from the North of Europe in their migrations, having natural conditions to the spawning and breeding of fish and shellfish.

The water body is surrounded at the marine-side by a sand barrier with several permanent openings to the sea and is fed by very small watercourses, with flow only from October to April, the freshwater contribution being therefore insignificant.

The level of the tide and the direction and intensity of the currents vary very quickly, which leads to the absence of species with low resistance to environmental changes. The water of the Ria is entirely renewed in a tidal cycle, with pH and salinity identical to sea values, high dissolved oxygen concentration and very low levels of nutrients and heavy metals.

At inner zones of the Ria near Faro there are bad aesthetical and organoleptic conditions and the bacterial counts are extremely high, showing the influence of crude domestic sewage. The bacteriological quality of the water in several points of the Ria is clearly inadequate to safe shellfish breeding, activity which is very important both at regional and national level.

4. ASSESSMENT OF THE MAIN ENVIRONMENTAL IMPACTS ASSOCIATED WITH THE EFFLUENT DISCHARGE

The most important problems which may result from the discharge of the treated effluent into Ria Formosa and that deserved particular consideration in the environmental impact assessment were the following:

a) Decrease of salinity in the zone close to the discarge;
b) Increase of nutrients in the zone close to the discharge;
c) Influence of the microbial contamination of the Ria by the treated effluent.

We will now make a summary of the main conclusions of the environmental impact assessment in respect to each of these items.

a) Decrease of salinity in the zone close to the discharge

The effluent is expected to have a low concentration in chloride and therefore a low salinity and will be discharged into a water body with high salinity (about 36 ‰).

The volume of water of the Ria is completely changed during a tidal cycle and a great part of the flora and the fauna of the Ria (namely shellfish) is resistant to great salinity changes.

Therefore we believe that the effect of the effluent discharge, as far as salinity is concerned, will be limited to the zone adjacent to the outfall and to the initial period of operation of the treatment plant, being its consequences the migration or adaptation of some species less resistant to salinity changes.

b) Increase of nutrients in the zone close to the discharge

In estuarine ecosystems, nitrogen appears to be the limiting factor for the growth of algae. If the water has low organic pollution and high dissolved oxygen content the production of nitrates is the prevailing process. The phosphorus is mainly involved in processes of absorption and release into solution by the sediments, a buffer mechanism acting to control those processes.

In Ria Formosa the nutrients are present in very low levels; the prevailing mechanism for N compounds is nitrification and P contents of the water is very small.

Due to the high retention time at the maturation pond, it is expected a reduction of 80 to 90% of N and a decrease of 60 to 70% of phosphates.

These facts, as well as the intensive water renovation occuring during each tidal cycle, lead us to accept that any eventual impacts of the effluent discharge will not be significant and that they will be confined to the zone close to the outfall.

c) Influence of the microbial contamination of Ria Formosa by the treated effluent

Conventional sewage treatment systems do not eliminate completely faecal pathogens from the wastewater and although several biological mechanisms (like action of predators and antiseptic activity of certain algal secretions) as well as the sensitivity to physical and chemical changes (insolation, salinity, nutrients) contribute to their severe reduction after discharge, the effects of the presence of pathogens cannot be ignored in Ria Formosa where several shellfish beds are located only a few quilometers away from the outfall

discharge.
The lagooning system is expected to remove at least 99% of coliform bacteria and about 99% of viruses, these one being mainly concentrated on the sewage sludges by adsorption. The removal of cysts and parasite eggs by sedimentation in lagoons will be total, due to their high density.
Although this treatment plant will provide a global improvement of the sanitary conditions of the central area of the Ria it cannot be denied that some restrictions on the shellfish grown in this area must be implemented, like its compulsory purification at the plants already working at Faro and Olhão.

5. RECOMMENDATIONS FOR FURTHER ACTION

- The construction and operation of the future treatment plant according to the best techniques of engineering is undoubtly a precious instrument for the improvement of the aesthetical and sanitary conditions in Ria Formosa;
- A strong quality control of shellfish must be implemented, to ensure its suitability to human consumption, according to EEC Directive 79/923;
- The impossibility of quantitative prediction of the environmental impacts of the effluent discharge, due to the complex hydrodynamics of the Ria, strongly advises the monitoring and periodical assessment of the future situation. That monitoring should rely mostly on regular surveys already carried out by some oficial entities and must include additionally bacteriological control of the water and shellfish near the discharge and studies on the evolution of flora and fauna in the area.

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STEEL INDUSTRY WASTES TREATMENT IN
BUFFER PONDS

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ABSTRACT

The bioprocesses of steel industry wastes self-purification in buffer ponds have been studied. The representatives of 6-7 divisions of microalgae and many functional groups of bacteria have been found in the ponds ecosystems. The important role of a new functional group, of so called "lytic" microorganisms. in the regulation of microecosystems and in bacterial self-purification was established.
It was found out, that the activity of self-purification bioprocesses in the studied buffer ponds depends on variety of ecological factors. The main limiting factor has been shown to be high turbidity of water and deficit of solved phosphates and oxygen in it.
To intensify self-purification bioprocesses of steel industry wastes in buffer ponds the regulation of concentration organic and mineral forms of main biogenic elements (C,N,P) and their correlation, water clarification and aeration were recommended and effectively used.

KEYWORDS

Buffer ponds, steel industry wastes, microalgae, bacteria, oxygen, biogenic elements, self-purification, optimization.

INTRODUCTION

The buffer ponds are known to play an important role in the protection of natural environment from steel industry wastes pollution (De Falco, 1975; Bagnyuk, 1984). But, the qualitative and quantitative composition of these ponds microbiocenosis, their capability to self-purification and limiting factors were studied unsufficiently.
The objectives of our study were to carry out the complex (hydrochemical, algological and microbiological) research of steel industry buffer ponds, to clear up some aspects of the self-purification bioprocesses activity and to elaborate the methods of it's intensification.

RESULTS

Ecosystems of steel industry buffer ponds are determined by the structure peculiarities and wide variety of biotic and abiotic factors relationships. In these ponds phyto- and heterotrophic microorganisms populations of high adaptivity to chemical wastes ingredients and integral activity are formed on account of geno- and phenotypical variability on the one hand and spontaneous mechanisms of ecological selection on the other. Oxidative-reductive, hydrolytic and transferase fermentative reactions resulting in removing of certain quantity of organic, mineral and biological admixtures take place in these ponds alongwith dilution, sedimentation and other known physico-chemical processes determining matter transformation. The microalgae, the numbers and primary production of which are considerably lower than that in river, and many functional groups of bacteria such as ones utilizing oil products, phenols, cyanides, thiocyanides, iron, manganese inhabit in the ponds ecosystems (Table 1). Moreover, a new functional group of so called "lytic microorganisms" which takes part in the regulation of microecosystems and in bacterial self-purification of wastes has been found (Bagnyuk et al.,1983).

TABLE 1. The phytoplankton numbers (10^3 cells per 1 l) and primary production (the amount of O_2 mg per 1 l for 24h) in the steel industry buffer ponds (June - August)

	River-derived water	Pond I	Pond II	Place lower water discharge into river
The phytoplankton numbers	1842,0±45,0	44,0±9,5	283,0±49,6	516,8±63,1
The primary production	5,5±1,0	0,5±0,02	1,0±0,04	3,6±0,8

The abundance of filamentous and mycoplasma-like forms of microorganisms, capable of giving rise to biogenic precipitation of iron and manganese as their hydroxides has been detected in buffer ponds phytoplankton (Fig. 1).

Fig. 1. The filamentous ironbacteria with of iron oxides concretions (a) and saprophytic mycoplasma with iron-manganese oxides deposits (b) on the plasmalemma, X8000.
TI - trichomes of the ironbacteria;
CI - concretions of the ironoxides;
Mb - the mycoplasmas body;
MO - manganese oxides deposits.

In spite of different ecological conditions in buffer ponds, the qualitative and quantitative composition of their heterotrophic microbiocenoses had significant alikeness.

The general quantity of microorganisms, basic organic pollutants destructors in buffer ponds water-reached 12 millions cells per ml, heterotrophic microorganisms - 430 thousands cells per ml. The petroleum-, phenol- and thiocyanidutilized bacteria amounts reached tens thousands cells per 1 ml of water in the same ponds (Bagnyuk,1984).
As a result of our investigation of steelplants buffer ponds it was established, that LM were present both in river-derived water used technologicaly, and in the buffer ponds water. Moreover, parallel with LM lysing saprophytic bacteria (Micrococcus lysodeikticus 2665), microorganisms, which capable to lyse conditionally pathogenic bacterioflora (Staphylococcus aureus N 941 and Proteus vulgaris toxic species), have been found in water thickness (Table 2). The significant increase of LM observed in September was in agreement with the increase of heterotrophic microflora density giving rise by the blue-green algaes "water-bloom".

TABLE 2. The amount of lytic microorganisms (cells per ml) in the water from the steel plant buffer pond

Month of sampling	Places of sampling	L M, counted on:		
		M. lysodeikticus	S. aureus	P. vulgaris
January	river	7500±60	4360±100	-
	pond	4900±90	2600±53	-
July	river	70±10	550±50	44±2
	pond	128±5	120±4,1	38±2,5
September	river	10100±100	9500±60	1200±250
	pond	6636±90	3470±123	10530±217

Activity of self-purification bioprocesses in the studied ponds varies in large ranges according to ecological (season, degree and kind of pollution). Thus, it has been shown by the method of isolated water samples that every 24 hours 6,3-73,2 % of oil products, 6-80,5 % of phenols, 7-33 % of cyanides and thiocyanides were destroyed biochemically, and 13,3-70 % of ammonium nitrogen were utilized in the water mass of the ponds.
The main factors limiting bioprocesses of steel industry wastes self-purification are high turbidity, deficit of solfed phosphates and oxygen. It has been calculated that oxygen was supplied to water by photosynthetic (0,5-2,6 mg/1 per 24 hours) and atmospheric (0,5-4,0 mg/1 per 24 hours) aeration. For 24 hours the processes of water substances oxidation required 25,4-100 % of dissolved oxygen.
As in aerobic ponds oxygen, required for organic matters oxydation by microbiocenoses is produced by phytoplankton, self-purification bioprocesses calculations are based on the prognosis of phytoplankton growth (Voinich-Sjanokzentsky, Khosrovjants, 1984).
The microalgae reproduction prognosis is discribed as a power series of main dimensionless parameters values, determining phytoplankton evolution: limiting biogenic element concentration, temperature, water clarity, time of process.
However, the calculations of steel industry buffer ponds self-purification bioprocesses is advisable to base not on BOD, but on one of the limiting pollutants. We consider, that in the integral process of self-purification both direct and indirect oxydation reactions, hydrolisis and transformation of organic, as well as mineral admixtures, may proceed even without free oxygen. Thus, nitrates, sulphates, phosphates, carbonates, chlorides and ions of metals may be used as electrons, acceptors in organic matters oxidative destruction by microorganisms.
It has been shown on the example of two buffer ponds in which 105 and 1100 mln.m^3 of respectively treated wastes were accumulated for cooling and pretreatment than in order to oxidize residual quantity of bivalent ferrum, oil and phenols about 1340 tons of dissolved oxygen for the first ponds and more than 9 thousands tons for the second are required. But the real level of

primary production and atmospheric aeration in the ponds, provides only 8-35 % of total oxygen demand. The mentioned above oxygen deficit may be eliminated either by pneumatic aeration which is expensive or by special measures providing intensification of self-purification bioprocesses. The latter is more advisable from the ecological and economical points of view. The addition of soluble phosphates to wastewater leads to increase of phyto- and bacterioplankton numbers and vital activity, hence about 25-50 % added phosphate was eliminated. At the same time the acceleration of self-purification, water softing effect and its saturation by oxygen were observed when compared with control.
The generalized mathematical model of final biological purifications management of steel industry total wastes, primary purified on the local sewage plants, has been developed. The model is based on the point systems of matrix differential equations:

$$\frac{dv}{dt} = AD\ (u)$$

$$\frac{dT}{dt} = B_1D\ (u) + B_2\ (x) + B_3T;$$

$$\frac{dZ}{dt} = C_1D\ (u) + C_2\ (u_o)X + C_3\ (u_o)Z;$$

$$\frac{dx}{dt} = M\ (u_o)X - F(X)D(u);$$

where:

V = water volume
T = water temperature
D (u) = entrance and exit water flows vector
u = flows of water management (fresh water addition, discharge into river, recirculation, removing of sludge);
z = pollutants, biogenic elements and soluble oxygen vector A,B,C,D,F - water and warm and matters balances matrixes
u_o = phosphatization of buffer ponds water
x^o = bacteria-destructors of pollutants and phytoplankton biomass vector.

In the model the meteorological and hydrological data, horizontal and vertical pond structure, balance of solved oxygen and main biogenic elements, the growth equation of phytoplankton microalgae and bacteria, which are organic matter destructors, are taking into consideration. The calculations based on the many factors correlation-regressive analysis permit to identify and choose the most advisable measures to intensify pretreatment of wastes from mechanical, organic and mineral admixtures and its stabilization by 1,3-2,0 times. Among the most reliable are follows: regulation of concentration between organic or mineral forms of biogenic elements (C,N,P), water clarification and aeration.

CONCLUSION

The adaptated algo-bacterial communities of organisms purifying water from organic and mineral pollutants inhabit in steel industry buffer ponds used for dilution, cooling, sedimentation and natural stabilization of industry wastes. The regulation both content and correlation of biogenic elements (C,N,P) in water allows to intensify the biochemical processes of buffer ponds phytoheterotrophic biocenoses and hence to improve the water purification quality.
For purpose of control and management of self-purification in buffer ponds automatic monitoring system is recommended. The minimal set of parameters which is necessary to establish the optimal regime of water quality is chosen on the base of the mentioned above model and they are transfered to the manager block. Thus the possibility of reiterative water use in the recirculated technological cycles is ensured.
An ecological and technological analysis of causal and resulting relationships between technogenic systems and environment made on the base of the principle of cost minimizing contribute to creating, the modern self-regulated ecological and industrial complexes and to maintenance its balance.

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THE ATTENUATION OF UNDERGROUND WATER CONTAMINATION IN A VINASSE CLAY-LINED POND

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ABSTRACT

Vinasse, the liquid residue from sugar cane alcohol destillation, is a major industrial pollutant in developing countries. The use of ethanol as an alternative automotive fuel in Brazil led to the construction of large number of stabilization ponds for the treatment of this material prior to river disposal. But, because of the infiltration of this high BOD residue into the underground, the water for human consuption has been highly contaminated. The objective of this work is to study a Brazilian clay which can be used both as an impermeable layer and also adsorb the organic components of vinasse. The results show that both the organic components of vinasse and the vinasse itself are adsorbed by the tetramethylammonium (TMA) derivative of a Brazilian sodium bentonite. Similar experiments performed with Wyoming bentonite show that the Brazilian clay is less efective than the American one because of its lower content on the clay mineral smectite. A computer simulation shows that the clay liner do not have to be replaced before one year of operation. This is very convenient because the alcohol industry works for 8 months approximately. Therefore when the distillery is not in operation the clay liner can be replaced.

KEYWORDS

Vinasse; organic load; bentonite; smectite; clay liner; underground infiltration; cane sugar alcohol.

INTRODUCTION

Vinasse is the residue from sugar cane alcohol destillation. It is a high BOD effluent because of its organic content (50,000 mg/l). Before the oil embargo of 1973 most of the vinasse produced in Brazil was used as a weak fertilizer. But because of the high cost of imported oil the automobile factories were encouraged to produce cars and trucks running on alcohol. The amount of vinasse produced increased so much that the excess had to be treated in stabilization ponds prior to river disposal. But the infiltration to underground water and leakage of these ponds is causing death of fishes and contamination of drinkig water. Experiments performed by the author (Büchler and Perry, 1986) had shown that the Wyoming bentonite saturated with TMA can adsorb most organic components of vinasse. The objective of the present study is to show that the Brazilian bentonite tested, in spite of its lower purity, can replace the Wyoming bentonite at a lower cost.

LITERATURE REVIEW

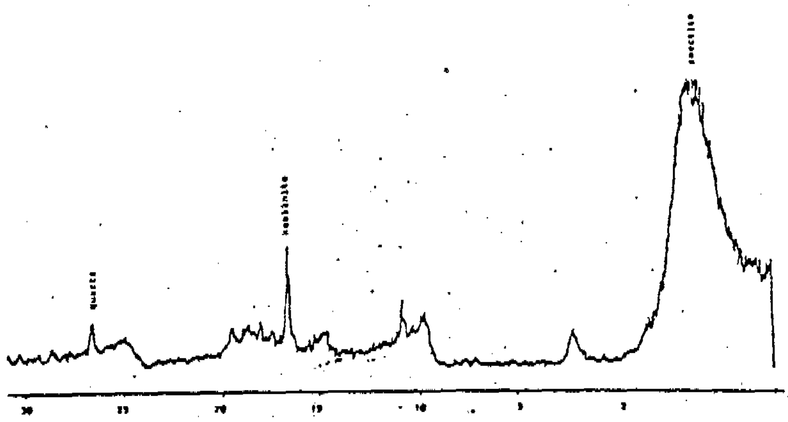
There is no information in the literature about the adsorption of vinasse on bentonites. But a substancial amount of data is available on the adsorption of the organic components of vinasse on bentonites reacted with several different cations (Theng, 1974). The TMA derivatives of sodium bentonites are specialy active on the adsorption of polar organic molecules (McBride, 1985). Therefore the adsorption is high on the surface of this clay mineral. TMA is a hydrophobic cation therefore is more likely to atract organic molecules which are not too miscible with water. This is specialy true in the adsorption of alcohols (German and Harding, 1969) and amino acids on bentonites (Sieskind, 1985 and Talibudeen, 1954). The adsorption of glucose is not so intense (Greenland, 1956) because it is a highly hydrated molecule as compared with the methylated glucose which is more intensively adsorbed by calcium bentonite. Dextran, which is the gum most frequently found in sugar cane, follows the "chain lenght rule" which means that the high molecular weight dextran found in vinasse is well adsorbed (Olness, 1975). Studies with glycerol (Brindley, 1966) show that the problems of its adsorption comes from the presence of 3 OH groups which make the molecule too bulky to fit into the interbasal space.

MATERIALS AND METHODS

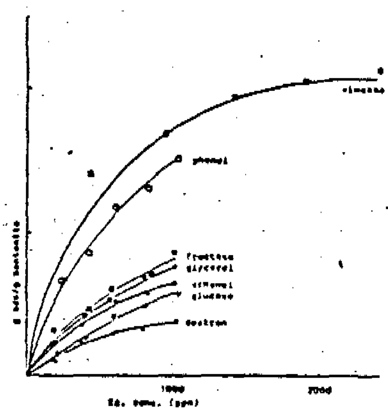
The materials in the laboratory experiments are: chemical grade glycerol, ethanol, glucose, fructose and phenol; clinical grade dextran and glycine and also vinasse from a local destillery. The sodim bentonite is reacted with TMA for 24 hours under agitation. The suspension is centrifuged and washed several times until no excess of TMA is present. The procedure to adsorb the organic components and the vinasse is similar to the one outlined above. The concentration of organics before and after adsorption on the clay surface is measured in a fully automatic digital total organic carbon analyser.

RESULTS

Figure 1 shows the X-ray difratogram of the Brazilian bentonite. The peak at 15.22 Å is caracteristic of the interbasal spacing of smectite. The peaks at 7.13 Å and 4.25 Å show kaolinite and quartz as impurities. The content of sodium smectite is 70% approximately. Figure 2 shows the adsorption isotherms of the pure components and also the isotherm of vinasse.



X-ray difratogram of the Brazilian bentonite



DISCUSSION

The temperature used in the present experiments is 30°C which is the average temperature in a tropical country where most cane sugar plantations are located. Lower temperatures do occur but they are the exception rather than the general rule. Besides, since the adsorption capacities are lower for higher temperatures the present results are conservative. The phenolic bodies present in the vinasse (as simulated by phenol) are strongly adsorbed by the TMA derivative of the Brazilian bentonite. Other components are adsorbed to a lesser extent. The results for vinasse are surprisingly high since we would expect an average value for the adsorption capacity as compared with the components. The suspended organic solids in the vinasse are responsible for the higher value of the adsorption. Actually the suspended solids are filterd by the clay liner and not adsorbed.

CONCLUSIONS

The impermeability of the clay and its capacity to adsorb organics makes it useful as a liner. Besides, as the computer simulation shows, the clay liner can be removed after one year of operation and therefore the system looks economical for a developing country.

ACKNOWLEDGMENTS

The author wants to thank the British Council for financial support and Professor Roger Perry from the Imperial College for the Research Proposal and permission to use the Public Health Laboratory.

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A CIVIL ENGINNEER'S POINT OF VIEW ON WATERTIGHTNESS AND CLOGGING OF WASTE STABILIZATION PONDS

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ABSTRACT

A survey on the watertightness of waste stabilization ponds in France and specific field investigations on clogging have lead to some recommendations for the designer. On average, 25 % of the French lagoons have leaking problems. The main reasons for this situation are an insuffisant preliminary geotechnical study and insuffisant initial influent flow. In some cases, clogging may occur and reduce the leaks after a few months.

KEYWORDS

Waste stabilization ponds, leaks, clogging, civil enginneers.

INTRODUCTION

Watertightness is generally a civil engineer's main concern when he has to deal with waste stabilization ponds. Leakage may pollute the groundwater and even hamper the filling of the bassins. Technical solutions do exist (soil compaction, soil improvement, synthetic membrane...), but they generally increase costs. Thus, an attractive possibility to reduce these expenses could seemingly be found in clogging the bassins after the filling. Sludge deposits bring about physical, chemical and biological clogging (Kristiansen R., 1981-Chang <u>et al.</u>, 1974) and create a low-permeability layer which can have a major effect on the infiltration rate (Gril, 1982).

Recommendations on watertightness with clogging are given for the designer on the basis of both an inquiry into the situation in France and specific field investigations.

SURVEY ON THE WATERTIGHTNESS OF WASTE STABILIZATION PONDS IN FRANCE

We questioned the SATESE (the French service departements for the sewage works) about the amount, the causes and the localisations of the leaks in the 612 waste stabilization ponds built between 1983 and 1986.

Figure 1 indicates the geographical distribution of these lagoons. On average, 25 % of them had watertightness problems. This high mean value does however considerably vary according to the regions (figure 2). A part of these regional fluctuactions can be related to the soil types. So, in the Landes (SW of France) where soils are mainly sandy, the percentage of leaking problems amounts to 40 %.

The answers on the survey indicate that leaks can be accounted for by :
- an insuffisant preliminary geotechnical investigation : 35 %
- ordered works which had been badly carried out: 28 %
- an insuffisant initial influent flow or a delayed filling of the bassins : 25 %
- various causes (rodent holes...) : 12 %

The leaks are mainly located at the bottom of the bassins (47 %) and at the dikes (42 %), rarely at the contact between the dike and the bottom (11 %).

FIELD STUDIES

The in-situ approach is composed of two parts :
- the observation of the clogging of lagoon bassins as soon as they are getting filled
- the study a posteriori of already clogged wastewater ponds.

* Observation of the clogging : it consits in following the evolution in time of the infiltration rate in the first bassin as soon as filled. We have selected ten lagoons, built up in summer-autumn 1896 and which presented a priori high leaking risks. They were characterized by a soil prospection. Just before filling the bassins with wastewater, we tried to measure the soil permeability in the bottom with clear water, by using the in-situ percolation test (W.S. Department of Health, 1959).

But this method presents a lot of difficulties (destructuration of the compacted layer, necessity of a large number of measures, long saturation time). Better initial values were obtained by drawing up the input-output balance of the water fluxes for a bassin just after its filling with clear water. The input flow was cut so that the infiltration rate could be estimated by measuring the decrease in the bassin water level related to time (rain and evaporation are taken into account). This method is also valid to follow the evolution of the infiltration rate periodically when clogging occurs.

The principal restriction comes from the difficulties to have a precise measure of the decrease in the water level with low soil permeabilities. The practical limit is $K=10^{-8}$m/s. In our cases, the initial permeabilities varied beween $K=5x10^{-8}$ m/s and $K=10^{-5}$ m/s with a mean value of $K=10^{-7}$m/s. Only one of the ten initially considered lagoons had finally the adequate characteristics to study the clogging (in particular large initial leaks and groundwater table far under the bassin's bottom).

It may be too early to give here any indication on how permeability has evolved since the filling of this bassin in october 1986.

* Study of a clogged lagoon : the influence of clogging may be illustrated by the example of HARSKIRCHEN's lagoon (NE of France), whose main characteristics are :
- date when it was brought into service : 1983
- number of equivalent inhabitants : 1300 at the filling, 2100 today among whom 500 seasonal
- waste and rainfallwater are collected in the same pipes
- mean flow by dry weather : 400 m^3/day in summer ; 300 m^3/day in winter
- total suspended solids in the influents : 160 mg/liter
- soil characteristics : sand and silt (heterogenous composition) with a 30 cm added on clay layer (theorically "imperveous")
- surface of the bassins : bassin 1 : 6000 m^2 ; bassin 2 : 6000 m^2 ; bassin 3 : 2000 m^2
- water table level : 1 meter under the bottom of the bassins.

For more than a year, the first pond could not be filled. At the moment however, all of the three bassins are full and don't leak. The measured sediment thickness in the first bassin varies from a few centimeters in the middle, to more than 40 cm at the inflow and the outflow. The mean permeability of the deposits is near 10^{-9} m/s. This low value can be related to the small particle size and to the high ratio of organic matter (15 %).

The example that we gave here shows that deposits may clogg a lagoon even with a considerable initial leak. But the forming of the clogged layer needs a period during which the groundwater can be highly polluted.

CONCLUSIONS

The survey carried out at the SATESE in France showed that watertightness problems of natural lagoon bassins do exist.

One lagoon out of four is leaking when getting filled and this, mainly at the bottom of the bassins. The main reasons for this situation as indicated in the survey are an insuffisant preliminary geotechnical study, badly carried out works and an insuffisant initial influent flow or a delayed filling of the bassins.

Considering also the results of the field studies and the CEMAGREF's experience in these civil engineering problems, following recommendations can be given :

* Necessity of a preliminary geotechnical study : pits for soil investigations should at least be dug and in-situ permeability tests be carried out (CTGREF, 1980). When the soil permeability K is higher than 10^{-7} m/s, watertightness must be improved by soil compaction, soil improvement or a synthetic membrane. For $10^{-8} < K < 10^{-7}$ m/s, it seems that clogging may generally occur and rapidly decrease the initial leaks. However, the optimal situation remains when $K < 10^{-8}$ m/s. Furthermore, knowing the groundwater table level is very important in order to evaluate the contamination risks.

* Necessity of an immediate filling of the bassins after their achievement : if this condition is not respected, shrinkage fissures may occur in sun exposed soils and the vegetation will perforate the impervious layer. Often, only a part of the inhabitants are initially linked up to the stabilization pond so that the sewage inflow is smaller than the leak. In such situations, the bassins must be filled with clear water provided for example by a nearby river.

* Clogging effects : first, there will be no clogging without any sewage inflow. Secondly, the sedimentation rate is difficult to predict in a general case. It seems that for the first bassin of a 500 equivalent inhabitant lagoon receiving only domestic sewage, the mean value is close to a 5 cm/year deposit. However, there can be a ratio of ten between the sediment thickness in the middle of a bassin and the sides. The sludge deposit and the clogged soil under it have a low permeability (we measured $K < 10^{-9}$ m/s).

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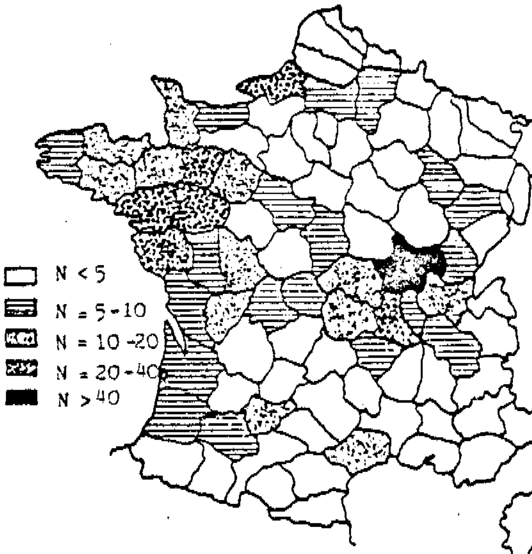

Fig. 1 : Number of lagoons built between 1983 and 1986

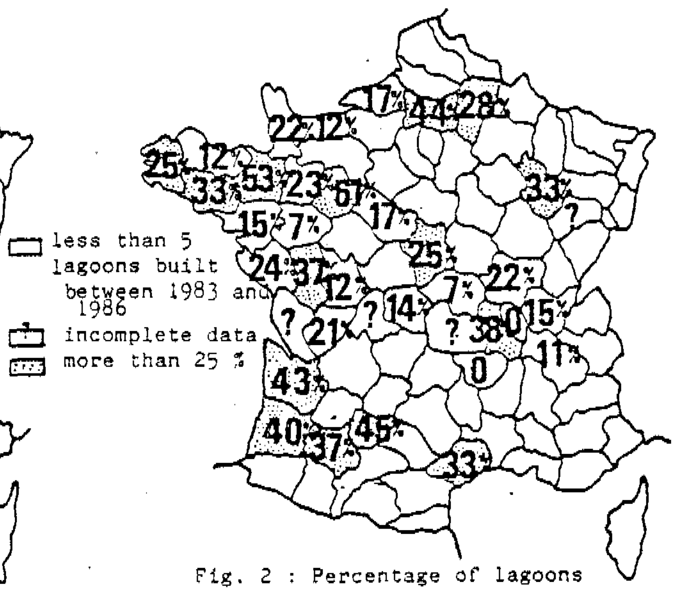

Fig. 2 : Percentage of lagoons with leaking problems

THE EFFECT OF pH ON THE PERFORMANCE OF HIGH RATE OXIDATION PONDS

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ABSTRACT

A common observation in high rate oxidation ponds (HROP) is that pH rise will usually be followed by decrease in algal production and in overall pond performance. It was found that the pH has minor direct effects on algal production but has major indirect effects as it determines the ratios of the carbonate system species and the dissociation of ammonia in the pond. Thus, increased pH values may cause carbon limitation of algal production or free ammonia inhibition of algal photosynthesis in HROP.

KEYWORDS

Oxidation ponds; pH; photosynthesis; biomass production; carbon dioxide; ammonia.

INTRODUCTION

High rate oxidation pond (HROP) is an intensive biological wastewater treatment process which combines both water reclamation and algal biomass production (Oswald,1972, Shelef et al., 1980). Among the many parameters which determine the performance of HROP, the pH is one of the most complicated. It has indirect effects which are not easily defined. One can include among the pH effects the availability of inorganic carbon to the algae (Azov et al.,1982), toxicity of ammonia to the living biomass (Azov and Goldman,1982), availability of phosphorus to the algae (Bogan et al.,1960), precipitation of calcium and magnesia salts and in some cases leads to the determination of algal species dominating the pond (Azov et al.,1980). The pH of the water determines the ratios of the carbonate system species in the pond as well as the ratios of free ammonia (NH_3) to ammonium ion (NH_4^+). We shall demonstrate the effects of these processes on HROP performance.

MATERIALS AND METHODS

Laboratory experiments were conducted using algae grown in fully controlled chemostates. The basic experimental procedure envolved algal photosynthesis rate measurements using the labeled carbon method. A detailed description of the continuous culture apparatus, the culturing protocols and the experimental analyses are given elsewhere (Azov and Goldman, 1982, Azov, 1982).

Outdoor experiments were conducted in pilot plant HROP (120 m^2 each) and in some of 36 mini-ponds 0.35 m^2 each. A detailed description of the outdoor facilities and the experimental procedures and analyses is given elsewhere (Azov et al., 1982).

pH AND CARBON AVAILABILITY

Laboratory experiments indicated decrease in algal photosyntheses at elevated pH values when total dissolved inorganic carbon (DIC) concentration in the media was 40 mg/1 (Fig. 1). However, it was found that actually, pH affected algal growth by determining the concentration of free CO_2 in the water, hence, determining the availability of carbon to algal photosynthesis. When total DIC concentrations in the experiments were calculated to produce equal amounts of free CO_2 at different pH values, no pH effects on algal photosynthesis could be observed (Fig. 1). Outdoor experiments confirmed the laboratory results. Algal cultures grown outdoors at constant pH value of 7.5 using HCl to suppress pH elevation by photosynthesis, could not maintain further growth after water alkalinity decreased below 0.2mM (on the 5th day). On the other hand, using CO_2 as pH regulator brought about further growth of the algae over control ponds (Fig. 2).

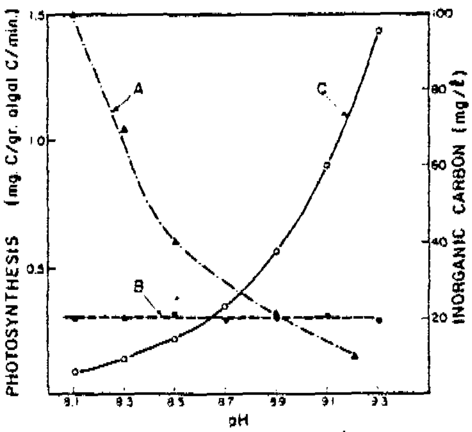

Fig. 1. Effects of pH and inorganic carbon on algal photosynthesis.

A. Increasing pH values at constant inorganic carbon concentration of 20 mg/1.

B. Increasing pH values at increasing inorganic carbon concentrations calculated to produce 0.1 mg C/1 of free CO_2 at each pH value.

C. Concentrations of inorganic carbon in experiment B.

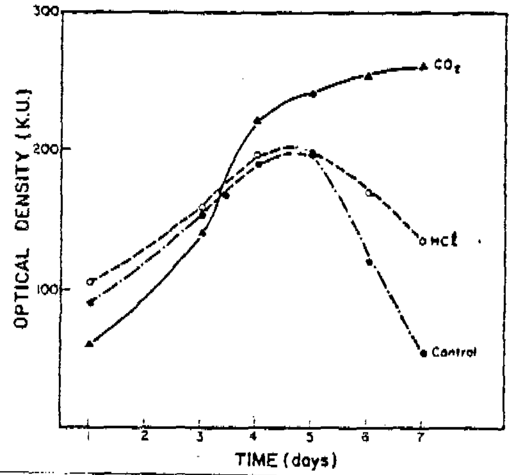

Fig. 2. Algal concentration in batch cultures having constant pH value (7.5) controlled by the addition of either CO_2 or HCl.

AMMONIA TOXICITY

Numerous experiments with different algal species indicated a general phenomenon of ammonia toxicity to algal photosynthesis. At constant total ammonia concentration ($NH_4^+ + NH_3$) of 5mM, a dramatic effect of pH on algal photosynthesis was observed. Carbon assimilation was reduced by 90% between pH values of 8.2 to 8.7 (Fig. 3). However, when free ammonia (NH_3) concentrations were calculated for each pH value it could be shown that pH had no other effect than determining the relative concentration of the toxic agent - NH_3, in the media (Fig. 3). The possible effects of ammonia toxicity on HROP performance is demonstrated in Figure 4. In batch cultures, algal photosynthesis will cause a pH rise up to values of 10 - 11. When high concentration of ammonia is present in the sewage feed, inhibition of photosynthesis by free ammonia will occur, following pH rise. At certain pH values, photosynthesis is inhibited almost completely concomitant with no further pH rise. After a few days when ammonia concentration is reduced due to evaporation and algal consumption, photosynthesis will be renewed causing further pH rise.

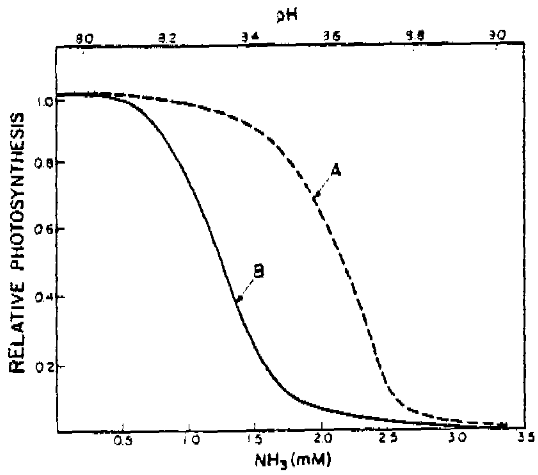

Fig. 3. Effect of pH and ammonia on algal photosynthesis.

A. Increasing pH values at constant total ammonia concentration ($NH_4^+ + NH_3$) of 5 mM.

B. Increasing free ammonia (NH_3) concentration (calculated from numerous experiments of different total ammonia and pH values).

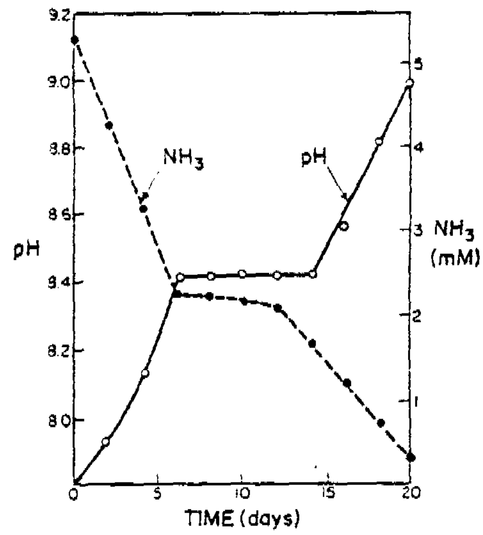

Fig. 4. Combined effects of pH and ammonia on batch culture of algae.

CONCLUSIONS

The high photosynthetic activity of algae in HROP increases pH to values that may affect the performance of the pond. Carbon limitation may occur in long retention times when organic carbon supply is low. In regular operation, the additional carbon which derive from biodegradation of the organic matter could prevent carbon limitation even at increased pH values. The high ammonia concentrations in certain domestic wastewater feed into HROP may cause algal photosynthesis inhibition at elevated pH values.

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THE IMPROVEMENT OF THE QUALITY OF OXIDATION POND EFFLUENT BY SAND FILTRATION TREATMENT
- Analysis of the accumulation and decomposition mechanisms by a mathematical model -

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ABSTRACT

In this research, the sand filtration process is focused on as one of the techniques for raising the quality of oxidation pond effluent. Based on the data of down-flow filtration experiments, a mathematical model for DO and deposit is developed, and the behavior of the deposit in the sand filter is analyzed. The decomposition of the deposit follows a first-order reaction at a filtration rate higher than 5 m/day, while it follows a zero-order reaction at a rate lower than 1 m/day. From the relationship between deposit and head loss, the maximum quantity of the deposit is determined to be around 0.3 kg/m^2 at the rate of 0.5-1 m/day and 0.2 kg/m^2 at the rate of 5-10 m/day.

KEY WORDS

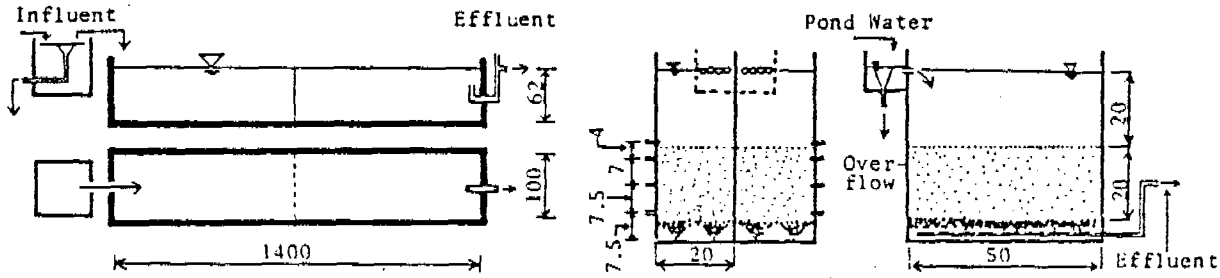
Sand filtration, mathematical model, oxidation pond, deposit, head loss, decomposition

INTRODUCTION

Sand filtration is available for the treatment of oxidation pond water because of its high efficiency of removal of SS material. To evaluate this treatment, information on the deposit and head loss is required. Deposit is decomposed to some extent during the filtration period before clogging. Then, mathematical models for the behavior of the deposit are developed and analysed.

EXPERIMENTAL PROCEDURES AND DEVELOPMENT OF MATHEMATICAL MODEL

Experimental procedures The experiments used for model verification are the same as the paper titled by "Water Quality Improvement of Secondary Effluent by an Oxidation Pond with Subsequent Sand Filtration Treatment"(Fujii *et al.*,1987). The layout of experimental equipment is shown in Fig.1. The



(a) oxidation pond (c m) (b) sand filtration equipment (c m)
Fig. 1 Layout of the test oxidation pond and sand filtration

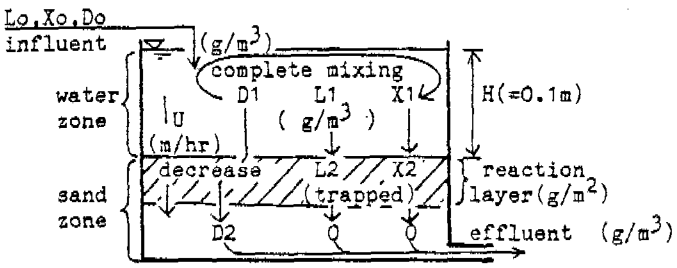
detention time of the oxidation pond was 3.3 days, and the effluent was filtered through the sand filter at the rate of 0.5 to 10 m/day. The conditions and results of each run are summarized in Table 1. SS, DO and head loss were measured and the final deposit mass was obtained in every run.

Model and differential equations For the estimation of deposit behavior, a mathematical model based on material balance is developed as shown in Table 2. In the model, SS is divided into VSS and ash. The ratio of VSS to SS in influent is determined to be 0.913 from the average value measured in pond water. From tracer experiments, the flow pattern is determined to be complete mixing in the water zone and piston flow in the sand layer. Since the deposit is almost completely concentrated near the sand surface, the very thin layer just below the surface is assumed to be a reaction layer where all reactions occur. The sedimentation of VSS and ash, the decomposition of VSS, the consumption of DO and the gasification of supersaturated DO are considered to be reactions in this model. The VSS decomposition rate is proportional to the VSS concentration.

Table 1 Condition and results of each filtration experiment

	Run 1	Run 2	Run 3	Run 4	Run 5	Run 6	Run 7
Filtration rate (m/day)	10	5	3	1	1	0.5	0.5
Period	Jul. 9 -Jul.10	Aug. 3 -Aug. 6	Aug.28 -Sep. 6	Sep.28 -Oct.22	Sep.28 -Oct.27	Sep.28 -Nov.18	Jun. 11 -Nov.10
Run length (days)	1.0	2.3	8.7	24.	29.	63.	152.
SS loading (g/m^2)	223	313	549	515	640	640	1684
Final Deposit(measured)(g/m^2)	205	288	462	354	401	354	793
(simulated)(g/m^2)	211	282	455	348	440	357	1066
Remarks					fish		fish

Table 2 Model for sand filtration and fundamental differential equations



$$dL1/dt=(Lo-L1)/T-sL1/H-rL1 \quad (1)$$

$$dX1/dt=(Xo-X1)/T-sX1/H \quad (2)$$

$$dD1/dt=(Do-D1)/T-k-arL1 \quad (3)$$

$$k=ko((D1-Ds)/Ds)^n(U/U^*)^m \quad (4)$$

$$dL2/dt=UL1+sL1-rL2 \quad (5)$$

$$dX2/dt=UX1+sX1 \quad (6)$$

$$dD2/dt=dD1/dt-ar(dL2/dt)/U \quad (7)$$

Lo,Xo,Do = VSS, ash & DO in influent (g/m^3)	H = depth of water zone (0.2 m)
L1,X1,D1 = VSS,ash & DO in water zone (g/m^3)	T = detention time of water zone (hr)
D2 = DO concentration in effluent (g/m^3)	a = (DO consumed)/(VSS decomposed) (=1.28)
L2,X2 = VSS & ash storage in sand layer (g/m^2)	s = settling coefficient (=0.01 m/hr)
Ds = saturation value of DO (g/m^3)	r = decomposition rate constant (1/hr)
U = filtration velocity (m/hr), U*= 0.042 m/hr	ko,n,m = parameter for gasification of DO

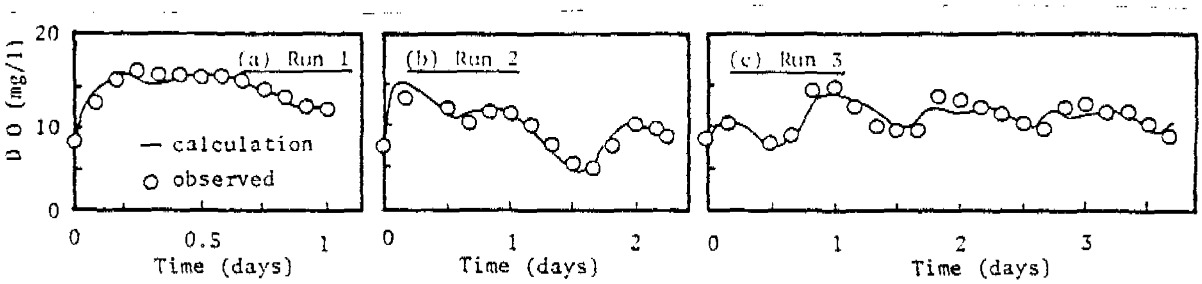


Fig. 2 The results of simulation in the effluent DO concentration

Numerical calculation and verification The data on Run 1, Run 2 and Run 3 (0–88 hrs) are used for the verification of the model. The differential equations are solved using the RKG method. The values of r, ko, n and m are determined to minimize the sum of the squares in calculation errors. Then, ko of 0.1 1/hr, n of 2 and m of 1 are determined in these three cases. r is 0.0034, 0.0032 and 0.0027 1/hr in Runs 1 to 3, respectively. The calculation results(Fig.2) show that a time-series variation of DO is precisely simulated with this model. The final calculated deposits are 211 and 282 g/m^2 for Runs 1 and 2, while the measured ones are 205 and 288 g/m^2 for Runs 1 and 2, respectively. The errors are less than 3 %, and thus the reliability of the model is demonstrated in this experiment.

Since the modeling of decomposition is difficult at a low rate, the deposit variation in Run 4-7 is obtained from the following procedure . First, the DO concentration for the case without biological reactions is calculated from modified differential equations, in which the decomposition rate constant r is given as zero. The difference between the measured DO concentration and the calculated one is considered to be the oxygen consumption in the sand filter. The decomposed quantity is calculated from this DO consumption. As shown in Table 1, the final simulated deposit quantity is coincident with measured one. The reliability of this model is also demonstrated.

DISCUSSION

The behavior of the deposit The variation of the influent SS loading, deposit mass (=VSS+ash) and cumulative decomposed mass are shown in Fig.3. The ratio of the decomposed material increases with the decrease in the filtration rate. In Run 6, the deposit appears to be approaching the same level as the decomposed mass, while it is almost the same as influent loading in Run 1. The decomposed mass seems to increase progressively in Runs 1 and 3, but linearly in Run 6. It seems that the decomposition reaction is of the zero-order at a low rate and is of the first-order at a high rate. At a low rate, the reaction may be controlled by oxygen supply rate. In Fig. 4, the VSS decomposition rate (Run 6) is plotted against temperature. In Run 6, this rate is expressed as $5*1.036^{t-25}$ g/(m^2day).

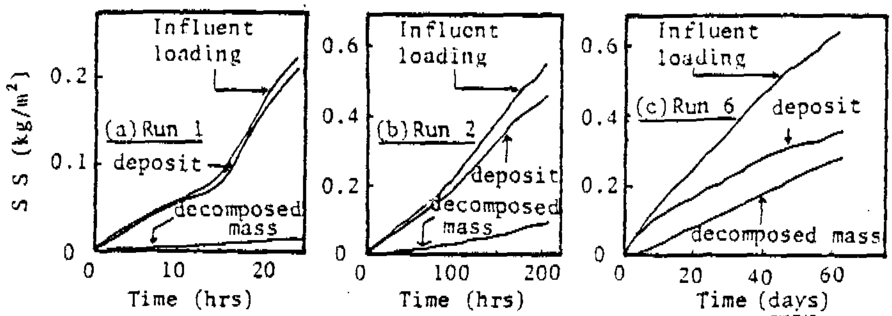


Fig. 3 Variation on influent SS loading, deposit mass and cumulative decomposed mass

Deposit and head loss Fig. 5 shows the relationship between the head loss and the simulated deposit. In Runs 1- 3, head loss increases with the increase in the deposit, while head loss in Runs 4 and 6 is very low before deposit becomes to a certain quantity, and then increases quickly. The final quantity of the deposit seems to be about 0.3 kg/m^2 at the rate of 0.5-1 m/day, but 0.2 kg/m^2 at 5-10 m/day.

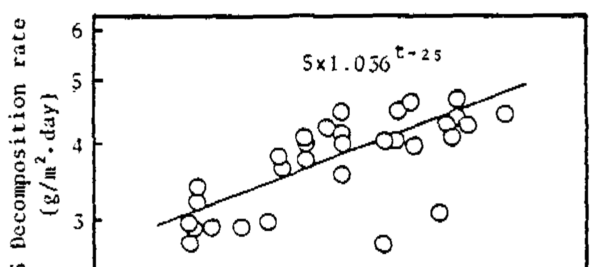


Fig. 4 Decomposition rate and temperature(Run 6)

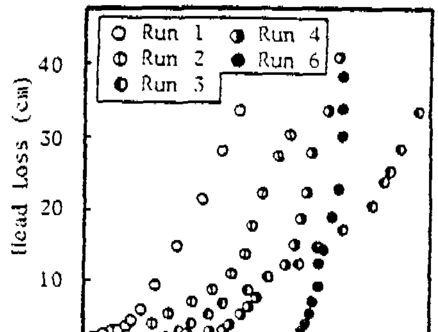


Fig. 5 Head loss and deposit mass

SUMMARY

In this research, the behavior of the deposit in a sand filter was discussed with the help of a mathematical model. The degradation of the deposit follows a first-order reaction at a filtration rate higher than 5 m/day, while it follows a zero-order reaction at a rate lower than 1 m/day. The maximum quantity of the deposit in the sand filtration bed is determined to be around 0.3 kg/m^2 of the bed surface area at the rate of 0.5 to 1 m/day and 0.2 kg/m^2 at 5 to 10 m/day. Since deposit behavior is successfully simulated with this model, the length of run until clogging occurs estimated from the time when the simulated deposit becomes this quantity.

MICRO ALGAE HARVESTING BY IN-SITU AUTOFLOTATION

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ABSTRACT

Dissolved Air Flotation (DAF) principles were applied to harvest microalgae from high rate algae pond (HRAP) effluents in a highly simplified system which consists of an open channel and a dosing pump only. Polyelectrolyte solution is dosed into the channel through a diffuser in the vecinity of a paddle wheel, where rapid mixing occurs. Slow mixing takes place in the channel. While flocs grow, oxygen bubbles resulting from algae photosynthetic activity are entrapped. Floating flocs are removed manually at the end of the channel. Removal efficiencies of 90% were obtained at a 10 mg polyelectrolyte dose. Float had 4 to 6% solids which dewatered easily to 10% solids.

KEYWORDS

Microalgae, Polycationic flocculant, Harvesting, Autoflotation, Oxygen.

INTRODUCTION

Production of algae on a continuous basis, at high yield, has been demonstrated to be possible when treating wastewater. The main problem in algae production in wastewater treatment is the high cost of separating the algae from the effluent. Preconcentration by sedimentation followed by centrifugation has been proposed as a means of reducing costs (Mohn 1978), but it would seem to be impractical to couple stabilization ponds, which are relatively inexpensive to operate, with centrifuges which are expensive pieces of equipment and are costly to operate (Middlebrooks et al. 1974). Flocculation followed by flotation seems to be the most accepted and reliable method for algae removal from pond effluents. However, the combination of flocculation and DAF is still too costly. The capital outlay for equipment is too high for the application of these methods in small communities and developing areas.

This paper is a continuation of work done on algae harvesting by flotation (Sandbank, et al, 1987), It introduces a procedure for harvesting microalgae, called in-situ autoflotation, which uses excess photosynthetic oxygen for the flotation of algal flocs in a channel using a simple technology, similarly to autoflotation performed by Koopman et al. (1983), for harvesting microalgae from HRAP treating animal wastes.

MATERIALS AND METHODS

A DAF unit was further simplified by eliminating recyrculation and performing flotation only at hours of photosynthetic oxygen supersaturation. A fast acting polyelectrolyte, (Zetag 57), was used in a 0.1% to 0.2% solution and injected in-line. Since the polyelectrolyte is not pH dependent at the pH range of HRAP, the pH controller and acid pump are not needed. Compared to conventional DAF, the rapid mixing tank, slow mixing tank, pressurization tank, pressure pump and the air compressor have been all eliminated. A further simplification is the replacement of the flotation tank and screpper by an open channel figs (1 and 2).

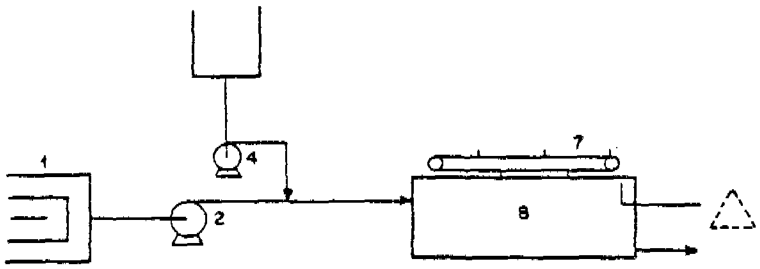


FIG. 1 Autoflotation unit or "Dissolved oxygen flotation" (Without recirculation pump, pH meter, pH control, pressure vessel and Venturi)

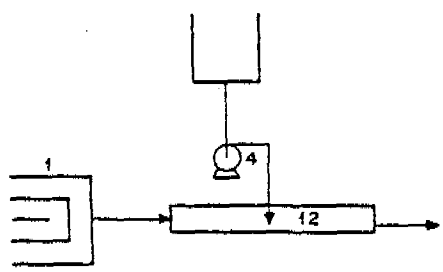


FIG. 2 In-situ autoflotation.

1. H.R. Pond
2. Pump
3. Rapid mixing
4. Dosing pump flocculant
5. Dosing pump acid
6. pH meter & Controller
7. Scrapper
8. Flotation unit
9. Pressurization pump
10. Air compressor
11. Pressurization tank
12. Flotation channel

In-situ-Autoflotation

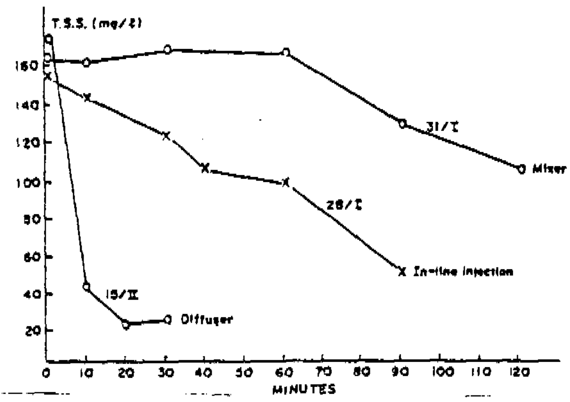
The overflow of a 100 m² high-rate pond operated at a retention time of 5 days was fed to a 17 m length channel. This channel was used as a flotation unit (Fig. 2). Harvesting was performed on a batch basis operation when the channel was full and when the dissolved oxygen level was above saturation. At this stage the inflow was discontinued. Flocculation of the algae and flotation were performed in-situ. A paddle wheel kept the channel mixed. Flocculation was performed by using the polyelectrolyte Zetag 57 in a 0,1 to 0,2% solution, at doses varying from 3 to 10 mg/l. Different ways of distributing and mixing the flocculants were tested, namely: a) addition through the inflow of a submerged pump which was pumping the water of the channel before the paddle wheel, b) addition near a flash-mixer placed before the paddle wheel and c) addition through a diffuser submerged in the channel. The diffuser consisted of three perforated pipes, as wide as the section of the channel, connected to a manifold and submerged at different depths, thus ensuring a good distribution of the flocculant. The flocs that are formed after the slow mixing action of the paddle wheel float downstream. They are intercepted by an inclined plate at the end of the channel.

RESULTS

The best method for introducing the polyelectrolyte solution is by in-line injection into the suction side of a pump, pumping effluent from the channel and spreading it through a diffuser (Fig. 3). The effect of incremental doses of Zetag 57 on algae removal by autoflotation showed that 10 mg/l remove 90% of the TSS (Fig. 4).

The collection of the algal float in in-situ-autoflotation was performed manually, in order to reduce equipment costs and maintenance. If required, a mechanical collecting device can easily be installed. The algal flocs produced with the polyelectrolyte, contained from 4 to 6% solids and dewatered easily on a screen to solids content of 8% to 10%.

One disadvantage of in-situ autoflotation is that its operation is limited to a few hours per day. However, considering the amount of equipment saved, together with the lower amortization and maintenance costs, this drawback is fully compensated. In situ autoflotation satisfies the requirement of simplicity and ease of operation which is necessary for small rural communities.



Fif. 3. In-situ autoflotation of microalgae. Effect of 5 mg/l polyelectrolyte on TSS removal.

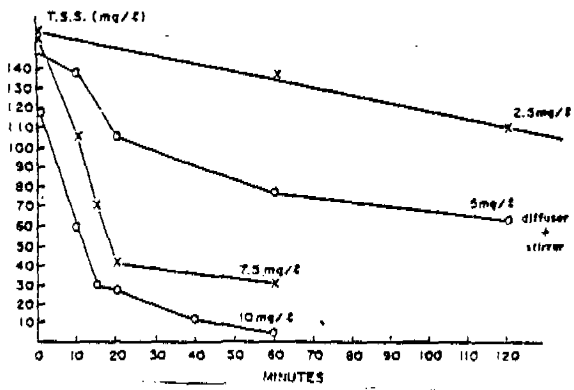


Fig. 4. In-situ autoflotation of algae. Effect of polyelectrlyte (Zetag 57) dose on the removal of TSS.

ACKNOWLEDGEMENTS

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Ultraviolet Radiation/Sedimentation of Wastewater Lagoon Effluents

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ABSTRACT

A process consisting of ultraviolet radiation followed by sedimentation was used to remove suspended solids from a wastewater lagoon effluent. An average of 33 percent of the suspended solids, 45 percent of the volatile suspended solids, and 54 percent of the BOD_5 were removed by this process. The process also produced a microbiologically acceptable effluent by removing 66 percent of the total coliform organisms, 78 percent of the fecal coliforms and 49 percent of the fecal streptococci organisms.

KEYWORDS

Wastewater lagoons, ultraviolet disinfection, suspended solids, bacteria, algae, discharge, standards, effluent polishing.

INTRODUCTION

Many methods of suspended solids removal from lagoon effluents have been used with varying success, including land application systems, intermittent sand filtration, dissolved air flotation, controlled discharge, in-pond removal by chemical addition, biological harvesting, autoflocculation, rock filters, coagulation-flocculation, microstraining, and centrifugation.

In addition to these processes, it has been suggested that UV disinfection followed by sedimentation (UV/sedimentation) may remove algal cells from suspension in lagoon effluents (Borup, 1982; Borup and Adams, 1985; Nieminska, 1985). In this process the germicidal effect of UV radiation is used to inactivate the algal cells in the effluent. UV radiation of wastewater streams has several advantages as a destructive agent. Effluents treated with UV light have fewer adverse effects on aquatic life than chlorinated effluents. UV radiation has little effect on both nonvolatile oxidizable organic constituents and nonvolatile UV absorbing components of a secondary effluent. UV radiation is effective against pathogenic bacteria and vegetative bacteria, and can effectively inactivate algal cells. A portion of the inactive algal cells can then be removed from the wastestream in a sedimentation basin. Because of concern in recent years over the possible harmful effects of wastewater effluent chlorination, UV radiation has also been investigated as an alternate disinfection method. Ultraviolet radiation followed by sedimentation was thus investigated as a method for polishing lagoon effluent.

MATERIALS AND METHODS

A pilot plant was assembled at the Logan City wastewater lagoons, Logan, Utah, (USA) to treat a portion of Logan City municipal wastewater after it has been biologically treated in a sequence of seven lagoons. The pilot plant used a Model SPF teflon-tube ultraviolet

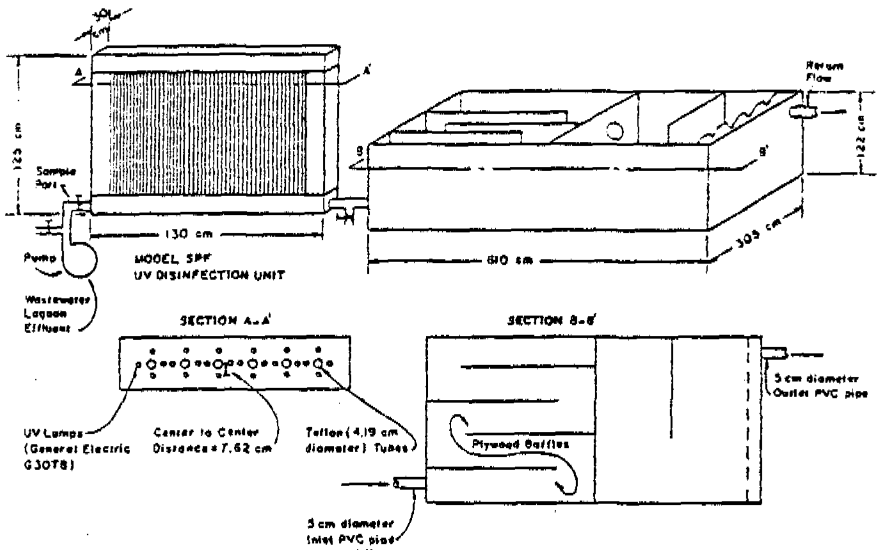

Fig. 1. UV/sedimentation pilot plant

device manufactured by Ultraviolet Technology Inc. (El Toro, California).

The unit was placed in a small utility building to give it protection and to attempt to keep the lamp skin temperature less dependent on the ambient temperature. Wastewater was taken from the point of discharge from the last lagoon and pumped through the UV reactor into a sedimentation basin (Figure 1). The sedimentation basin was divided into two compartments. Baffles in the first compartment created a serpentine flow of the wastewater. One baffle was situated across the second compartment to diminish short circuiting and increase hydraulic efficiency. Mean and median detention time was determined by the use of fluorescein dye concentration changes with time. The pilot plant was operated from August 5 until December 10, 1984. The wastewater flow rate was varied over the period of the study from about 37 to 190 l/min (10 to 50 gpm). Grab samples were taken daily from sample ports before and after the ultraviolet unit, and from the end of the pipe discharging the wastewater from the sedimentation basin. The system was operated alternatively with the wastewater in the basin exposed to sunlight, or with the tank covered with a non-transparent plastic tarp.

Grab samples were collected daily, taken to the Utah Water Research Laboratory and examined for total and volatile suspended solids, total and fecal coliforms, and fecal streptococci (APHA, 1980). Twice a week, during the pilot plant study, major wastewater quality parameters were examined, i.e., suspended, volatile and dissolved solids, ammonia, nitrites, nitrates, total phosphorus, ortho-phosphate, BOD_5, pH, and turbidity were determined (APHA, 1980).

RESULTS AND CONCLUSIONS

Among wastewater quality parameters only suspended solids and volatile solids BOD_5, coliform, bacteria, and fecal streptococci played an important role in the performance of UV radiation followed by sedimentation. (Table 1)

Assessment of the feasibility of removing suspended solids and water-borne bacteria from wastewater lagoon effluents by UV light disinfection followed by sedimentation resulted in quantitative isolation of major factors involved in the overall treatment efficiency. Changes in ultraviolet disinfection efficiency were monitored and related to changes in wastewater flow rate changes, in wastewater quality, and changes in environmental conditions such as temperature, precipitation, and sunlight intensity.

Analysis of the results of the tests performed lead to the following conclusions concerning UV/sedimentation performance:

1. The UV/sedimentation process removed an average of 33 percent total suspended solids (to 4 mg/l) in lagoon effluents having an average 6 mg/l suspended solids.

2. The UV/sedimentation inactivated 66 percent of the total coliforms, 78 percent fecal

coliforms, and 49 percent of the fecal streptococci from lagoon effluent, resulting in discharging less than 30 total coliform bacteria/100 ml and less than 60 fecal streptococci/100 ml.

3. An increase in overall solids and bacteria removal efficiency was observed when higher than average concentrations of solids and bacteria were found in lagoon effluent.

4. A decrease in wastewater BOD_5 by an average 54 percent was achieved in the pilot plant application at the Logan lagoons. There was no significant change in pH, turbidity, nitrogen and phosphoraus compounds, and in metals concentrations in wastewater as it passed through the process.

5. An increase in wastewater flow rate from 37.8 to 189 l/min resulted in a decrease in disinfection efficiency from 97 to 49 percent expressed as removal of fecal streptococci and other bacteria growing on KF streptococcal agar. A non-linear relationship between UV dose and wastewater flow rate was found.

6. No significant influence of wastewater quality parameters on disinfection efficiency was observed at the concentration levels found in the wastewater lagoon effluent.

7. Changes in air and wastewater temperature had no significant effect on the UV/sedimentation performance.

8. Photoreactivation of bacteria in wastewater reduced treatment efficiency.

TABLE 1. Physical, Chemical, and Bacteriological Data Summarizing the Pilot Studies at Logan Wastewater Lagoons from August 6, 1984, until December 12, 1984

	No. of Samples	Lagoon Effluent			After UV disinfection			After sedimentation			Overall Percent Removal
		Mean	Range	S.D.	Mean	Range	S.D.	Mean	Range	S.D	
Water Temp.(C)	91	12.5	2.0-23.5	7.7	12.5	2.0-23.5	7.7	12.5	2.0-12.5	7.7	
pH	17	8.0	7.4-8.4	0.3	8.0	7.5-8.3	0.2	8.0	7.5-8.3	0.2	
Turbidity (NTU)	17	5.6	3.6-7.5	1.2	5.6	3.6-7.5	1.2	4.9	3.3-6.0	1.1	12
Total Suspended Solids (mg/l)	91	6.0	2.4-14.2	2.2	5.2	1.8-13.8	2.2	4.0	0.4-8.8	1.8	33
Volatile Suspended Solids (mg/l)	91	3.0	0.4-7.2	1.3	2.4	0-4.8	1.0	1.7	0-4.6	0.9	45
BOD_5 (mg/l)	18	12.0	6-18	3.0	6.4	2-16	4.0	5.5	2-14	3	54
Total Coliforms per 100 ml	75	89	0-883	121	7	0-153	21	30	0-175	31	66
Fecal Coliforms per 100 ml	75	18	0-113	20	1	1-18	3	4	0-30	8	78
Fecal Streptococci per 100 ml	39	119	16-299	80	44	5-114	3	61	9-180	46	49

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EFFECTS OF LAGOON TREATMENT ON HELMINTH EGGS

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ABSTRACT

In this study, we have investigated the effect of lagoon treatment on the elimination of helminth eggs. The Marrakech treatment lagoon receiving only part of the city sewage, consists of two connected basins (total retention time from 8 days to 30 days). Water samples (surface and deep) and sediment samples were collected. In the surface water samples collected at the lagoon entrance there was an average of 11.7 eggs/l while no helminth eggs could be recovered at the exit from the lagoon. In the two basins very few eggs were found, both in surface and deep water samples (≈ 3/1). Helminth eggs were mostly found only in the sediment samples 275 eggs/100 g sediment from the first basin and 158 eggs/100 g sediment from the second, on the average. In conclusion, this study demonstrates that lagoon treatment is efficacious in removing helminth eggs. Nonetheless sediments from such treatment present a problem as they remain heavily contaminated with helminth eggs.

KEYWORDS

Sewage treatment - Helminth eggs - Lagoon treatment - Teleman-Rivas technique modified according to Bailenger.

INTRODUCTION

Sewage treatment is a fundamental problem in countries with limited water resources. The ideal solution would be a reasonably priced, simple and reliable method of purification, which produced a water both chemically and biologically pure enough to permit reutilization for agricultural uses. Lagoon treatment could be such a method. In this study, we have investigated the effect of lagoon treatment on the elimination of helminth eggs.

MATERIALS AND METHODS

The Marrakech treatment lagoon consists of two connected basins with the following characteristics : 2,400 m^2 in area, 1 m in depth and 2,400 m^2 in volume. The bottom is covered with a plastic film, rendering it impermeable. This lagoon receives only part of the city's sewage. Two different flow strategies have been tested : First period (12/7/1985-28/11/1985) : 6 l per second, 4 d retention in each basin giving an 8 d total retention time ; Second period (12/12/1985-6/12/1986) : 1.5 l per second, 15 d retention in each basin giving a 30 d total retention time. Four sampling stations were selected : the entry canal into the lagoon (E) ; the exit from the first basin (I) ; the exit from the second basin (II) ; the effluent canal from the lagoon (S).
On liter samples were collected, from the surface at all 4 stations and from 80 cm down at

the basin exits (If and IIf). Sediment samples were collected in plastic traps placed at the bottom of the basins (Is and IIs). Surface water samples were collected twice monthly (92 samples), deep water samples every two months (16 samples) and sediment ones every three months (6 samples).
An enrichment technique was necessary as the concentration of parasites was very low. Preliminary studies (Stien and Schwartzbrod, 1986) indicated that the Teleman-Rivas technique modified according to Bailenger would be the most useful for this study. Results were quantified using a MacMaster cell.

RESULTS

Quantitative results were analyzed according to sample type, water or sediment.

Surface water samples. At the lagoon entrance (E), 20 out of 23 samples (86.9 %) contained from 1 to 36 helminth eggs per liter (an average of 11.7 eggs/l). At the exit of the first basin (I), 4 out of 23 samples (17.4 %) were positive with from 1 to 6 eggs per liter. At the exit of the second basin (II), no eggs were isolated from any of the 23 samples. At the lagoon exit (E), no helminth eggs were found in any of the 23 samples analyzed.

Deep water samples. Only 2 out of 16 samples analyzed were positive, one at the exit of the first basin and the other at the exit of the second one. The concentration of eggs in these positive samples was very low, from 1 to 3 eggs per liter.

Sediment samples. Helminth eggs were found in all the sediment samples tested. From 225 to 325 eggs/100 g wet matter were recovered from the samples at the first basin exit while the second basin exit samples yielded smaller quntities, from 75 to 200 eggs/100 g wet matter. Nonetheless, it is difficult to draw definitive conclusions from such a small number of samples.
No difference was found in helminth egg recovery as a function of flow rate.

Distribution of Helminth eggs recovered. Three types of helminth eggs were recovered from the samples tested : Nematodes (*Trichuris*, *Ascaris*), Cestodes (*Tenia*, *Hymenolepis*) and many free-living Nematodes or ankylostomides-type eggs. This last group is not considered an animal parasite. Samples from the lagoon entrance contained both Cestode eggs (*Hymenolepis*, *Tenia*) and Nematodes eggs (*Trichuris*, *Ascaris*). *Hymenolepis* and *Trichuris* eggs predominated in their respective families. Surface water samples from the basins contained only *Nematode Trichuris* eggs while the deep water samples from the basins had only Cestode eggs (*Hymenolepis* and *Tenia*). The sediment samples from the first basin had both Cestode eggs (53.4 %) and Nematode eggs (46.6 %). *Hymenolepis* and *Ascaris* eggs predominated slightly in their respective families.

CONCLUSION - DISCUSSION

Overall, there was an average of 11.7 eggs/l at the entrance to the lagoon, while no helminth eggs could be recovered at the exit from the lagoon. In the two basins, very few eggs were found, both in surface and deep water samples (≈ 3/l). Helminth eggs were mostly found only in the sediment samples : 275 eggs/100 g sediment from the first basin and 158 eggs/100 g sediment from the second, on the average. There is considerable variation in the numbers of helminth eggs recovered from sewage water samples reported in the literature. Reported values range from 27 eggs/l (Knaack and Ritschel, 1975) to 2,330 eggs/l (Fitzgerad, 1977) with intermediate values such as 219/l (Grabow and Nupen, 1972) and 450/l (Panicker and Krishnamoorthi, 1981). The sediment samples tested here must be compared to values given for sludge samples in the literature. These are also quite varied, from 10 (Kabrick and Jewell, 1982) to 1,440 eggs/100 g sludge (Reimers *et al.*, 1981) with intermediate values of 54 (Graham, 1981) and 460 eggs/100 g sludge (Arther *et al.*, 1981). Species distributions in helminth eggs recovered from sewage water samples reported in the literature generally agree with our findings. Eggs from *Ascaris*, *Trichuris*, *Tenia* and *Hymenolepis* are all identified although *Toxocara* (Fitzgerald, 1977) and *Enterobius* (Knaack and Ritschel, 1975) were also reported. Most authors also found *Toxocara* eggs in sludge samples, (Kabrick and Jewell, 1982 ; Fitzgerald, 1977 ; Graham, 1981 ; Schwartzbrod *et al.*, 1986).

In conclusion, this study demonstrates that lagoon treatment of sewage is efficacious in removing helminth eggs, even after only short retention times. No eggs could be recovered from the lagoon exit samples using the method described by Teleman-Rivas and modified by Bailenger. Nonetheless, sediments from such treatment present a problem as they remain heavily contaminated with helminth eggs. The next step should be the investigation of helminth egg survival time in such sediments.

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WASTE STABILIZATION PONDS AS TEACHING AND RESEARCH TOOLS

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ABSTRACT

In any campus where there is an excess of land or where a decorative pond is available, it is often possible to establish a system of waste stabilization ponds (WSP) to be used as an open air research laboratory, as source of water for watering the campus greenery and as demonstration units for the community at large. One such system, consisting one photosynthetic and one macrophyte ponds in parallel, followed by one fish and one irrigation pond in series, was built at the Faro Polythecnic in Portugal. The ponds are preceeded by another underground unit formed by three septic tanks in series followed by two upflow anaerobic filters in parallel. This system is expected to deal with $120m^3$/day of a mixed effluent coming from the sanitary facilities, the refectory and the fish processing laboratory, with concentrations of about 600mg/l in both BOD and SS.

KEYWORDS

Waste stabilization, demonstration units, water reuse, photosynthetic ponds, macrophyte ponds, fish ponds, anaerobic upflow filters

PONDS IN THE CAMPUS: THE NEED AND THE FEASIBILITY

Institutions of higher education, such as universities and polytechnics, which offer courses suitable for the training of water scientists and engineers, are always faced with the difficulty of finding appropriate facilities for performing experiments that may be comparable to real world situations. Such a problem comes to the fore when dealing with wastewater treatment processes which require large expanses of land. This is often circunvented by resorting to bench scale experiments and leaving aside the processes that are less amenable to miniaturization. In such cases ponds are always found at a loss.

However, it does not need to be so. Many a campus is provided with decorative or landscaping pools that can easily be converted to stabilization ponds which alone or in association with compact or underground units, may be organized into effective wastewater treatment systems.

THE EXPERIMENTAL DIMENSION

A large number of experiments of varying complexity can be carried out in such a WSP system. They are ideal to determine the rate constants for BOD and coliform decay, to study inhibitory effects to treatment processes caused by metals, pesticides or other industrially originated toxic materials, to test the efficiency of fixed or floating macrophytes on the removal of contaminants and to study the adaptation of fish and molluscs to pond environments and to measure their rate of growth, their most convenient population densities and their suitability for the uptake of nutrients.

Another set of experiments can also be performed outside the pond environment using the pond water, either before or after the treatment is completed. Examples of such experiments are studies on hydroponics, using both recirculating and "once through" flow arrangements, the measurement of biomass buildup by irrigated plants, the efficiency of nutrient removal by plant roots, the use of unsaturated soil as a wastewater treatment media and the modelling of groundwater contamination.

THE DEMONSTRATION DIMENSION

The water exiting from the pond system may be distributed by an underground pipe network and used for watering the grass, flower beds and trees existing in the campus and peripheral fields, which may bring in considerable savings in waters bills and pumping costs from alternative boreholes. This is particularly important in climates of low and erratic rainfall where the peak demand for water by other users coincides with the dry periods. Wastewater treated by a pond system, with its normally excelent coliform quality, is well suited to be used for satisfying the irrigation needs, leaving aside the underground water resources to meet other needs.

In the course of their professional lives, the students, that were exposed to the merits of the pond system while at college, will tend to adopt it whenever suitable, thus transfering the technology into the community at large. Besides, the system may be used as a direct demonstration facility for the benefit of water managers and the general public.

THE FARO POLYTECHNIC EXPERIMENT

The System

A sewerage network serving the academic buildings, the catering block and the sports center empties into the sump of the pumping station that feeds the wastewater to an underground, three compartment septic tank; from there on the flow is gravity driven. The upstream compartment has 2/3of the total capacity to accommodate the build up of sediment. The anaerobic effluent from the tank enters two parallel anaerobic, fixed film, upflow filters packed with high specific area plastic media.

The effluent from the filters may be split between two aerobic ponds and fed near their bottom; one of them is a photosynthetic pond and the other will be populated with a choice of fixed or floating macrophytes, as required by the experiments. Those two ponds empty into a fish pond where selected populations of both fish and shellfish will be tested. Recirculation water is pumped from this pond to a manhole some 10m upstream from the pond system where it joins the effluent from the anaerobic filter and satisfies its immediate oxygen demand. The effluent from this pond runs into a retaining tank from where it is pumped into a pressurized underground grass irrigation network. Excess water is wasted into the town sewer.

The water level in the system may be varied by adjustable weirs which allow the fish free movement. Shallow wells dug into the ponds bottom provide shelter for the fish when the ponds need emptying for cleaning or mosquito control.

The Design Criteria

The fish processing laboratory will discharge $60m^3$/day with 1000mg/l of both BOD and SS. Full capacity student and staff population is 1200 and wastewater discharge is taken as 50ℓ/hd.day. A mixed effluent of $120m^3$/day and 650mg/l of both BOD and SS was assumed. A coliform count of less than 10/100ml was taken as the limiting design parameter.

A septic tank hydraulic retention time (HRT) of 18 hours was chosen in order to achieve a removal of 40% BOD, 60% SS and 60% coliform count. Allowing for the long night and weekend stoppages the HRT based criteria was dropped in favour of a BOD based one to design the anaerobic filter and a conservative value of $1.3kg/m^3$day was assumed. A removal of 85% BOD and 60%coliform count is expected in the filter. The depth of all ponds is 0.75m. The macrophyte pond, with an area of 0.14ha, an HRT of 14 days at full flow and an organic load of 50kg BOD/ha, day, is expected to give removals of 80% BOD and 96% coliform count. The aquaculture pond has an area of 0.05ha and receives 1.4kg BOD/day. It should give removals of 95% BOD and 99.9% coliform count. Therefore the final effluent should have ≈1mg/l BOD and ≈6 colif./100ml.

ACKNOWLEDGEMENTS

We are grateful to Prof. J. M. Novais for his contribution on establishing the design criteria for the system.

DOMESTIC WASTEWATER TREATMENT IN TANKS WITH EMERGENT HYDROPHYTES : LATEST RESULTS OF A RECENT PLANT IN FRANCE

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KEYWORDS

Wastewater treatment, domestic wastewater, rooted macrophytes, emergent hydrophytes, full scale case study.

INTRODUCTION

Since 1979, CEMAGREF has been studying the potential purifying action of aquatic vegetation in two small wastewater treatment units (Boutin, 1986) built under the direct supervision of Dr. K. SEIDEL to ensure that they are identical to "MPIP Process or Krefeld system" plants.

In 1985, based on these results, CEMAGREF designed a similar pilot plant for the Village of Pont Remy (Somme), 500 inhab. eq. capacity.

OBJECTIVES OF THE PILOT PLANT

The Saint Bohaire EHTS plants (Boutin, 1986) are of a small size (approximately 30 inhab. eq.) and, irrespective of the high quality of their construction, they are nonetheless man-made, which makes it difficult to estimate their real construction cost. Interesting treatment results have already been obtained. Nevertheless, these promising results still remain to be confirmed by advanced investigations on a larger plant receiving polluting loads of domestic origin such as a permanently resident population would produce.

PLANT DESCRIPTION

The operating principle is identical to the one used at Saint Bohaire. The useful surface area of the plant (1250 m^2) was calculated on the basis of the Saint Bohaire results, according to which a specific surface area of 2.5 m^2/inh. eq. should be sufficient. The treatment is carried out in 5 stages :

1st and 2nd stages (percolation flow).
The first stage consists of 8 parallel tanks with a surface area of 80 m^2 each, (length 16m, breadth 5m). Only four of them are presently operated so as to rapidly collect information at a significant load rate, based on users' sewerage connections.

In order to observe the relative treatment performance of each hydrophytes specie represented, the retained configurations are as follows :
- 1 tank planted with Phragmites communis,
- 2 tanks planted with Glyceria aquatica,

- 1 tank without plants.

The second stage is composed of 4 tanks, 2 of which are now operational. They have been planted with Phragmites only, and covered on the surface with a 2.5cm sand layer of 0 to 5mm grain size. After completion, when the whole area is put in operation, this type of tanks will represent almost 77 % of the total surface area.

3rd, 4th and 5th stages. These tanks, which are planted with Scirpus (3rd and 4th) and Iris (5th) are arranged in a series. In the initial design, the water level is kept constant by a sill, thus providing a mainly horizontal component path, known as translation flow. The surface area of each tank is 96m² (17.5m X 5.5m).

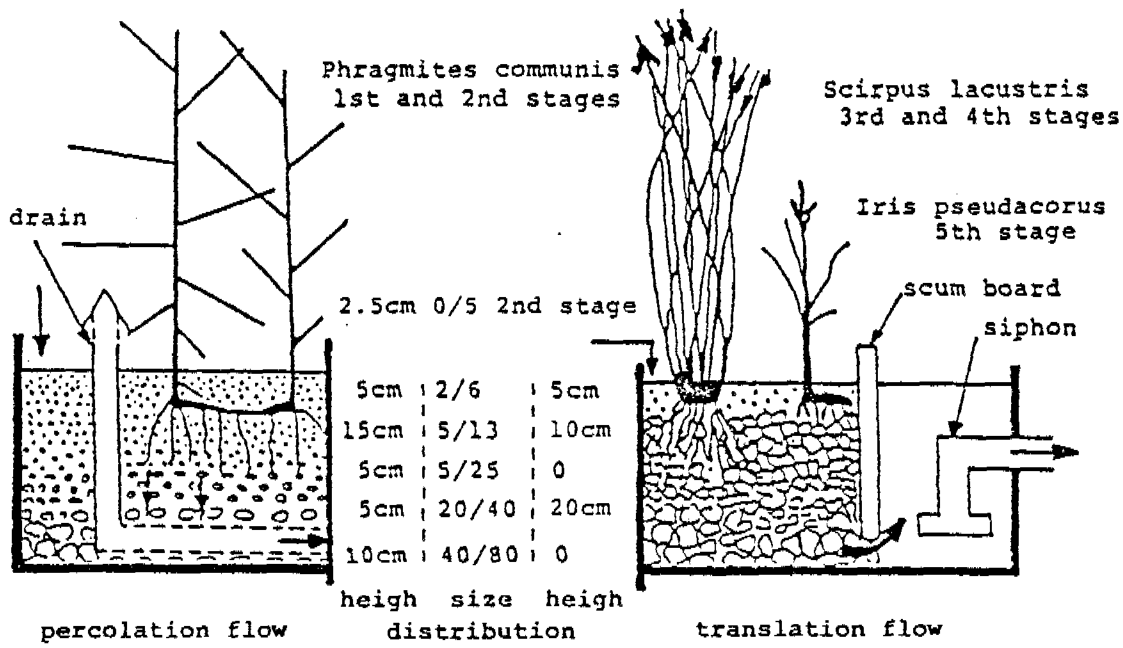

Fig.1. Longitudinal section of the tanks

Auxiliary works. The plant is fed from a recently built separate sewerage system, by means of a two-pumps prefabricated unit. The inlet pipework is connected to a static grease trap which also operates as a distributor at the top of the 1st stage tanks.
A measuring channel with a triangular adjustable weir is placed at the outlet before discharge into the Somme River.

CONSTRUCTION TECHNIQUE USED

Concrete was immediately selected as construction material. However, because of local problems of loadbearing capacity of the ground, it was necessary to substantially reinforce it. For the same reason, the outer walls are made of "STEPOC" reinforced hollow construction blocks, then lined with a water repellent coating, for increased watertightness. As a result, this strong construction was found to be expensive. The minimum cost (end of 1986 value) was estimated to be 1650 F.F/inh.eq.

OBSERVATIONS ON THE TRANSPLANTATION AND SECOND GROWTH OF VARIOUS PLANT SPECIES

General condition. All the young shoots were bought and planted by a specialized horticulturist. The purpose of this was to identify the quality of the market available plants and to evaluate the cost of the total service, including one-year growth guarantee and maintenance.

The 2 year old seedlings had grown in a tight pool filled with earth and watered with river water. The offshoots were picked the day before their transplantation and stored in water-saturated atmosphere inside tight containers during their transfer. Prior to the actual transplantation, the tanks were filled with clear water up to a level slightly above the gravel surface. This immersion was constantly maintained for 4 weeks, followed by the normal introduction of wastewater with periodic dilution using water from the Somme River.

The planting was done in 2 stages :
- September 1985 - Phragmites, Scirpus and Iris
- End of June 1986 - new planting of Phragmites and Glyceria

The plantation density was always maintained at 5 units per m^2.

Phragmites. This specie was characterized by a very low second growth rate on two attemps (10 % and 15 % respectively). This may be due to a number of reasons (heterogeneity and size of the mineral substrate - lack of fertilizer), the short size of the picked rhizomes may also be a determining factor.

Glyceria, Scirpus and Iris. Scirpus and Iris have remarkably taken root again (100 %) right from the first planting. A similar result was achieved with the Glyceria used to replace part of the Phragmites in the second planting attempt.

REMARKS CONCERNING THE FUNCTIONING OF TRANSLATION FLOW TANKS

The experience acquired in Saint Bohaire shows that the design of those tanks needs perfecting. The medium is insufficiently aerated, the low mineralization of the organic matter is responsible for an excessively rapid clogging and an insuficient nitrification prior to denitrification. The oxidoreduction potentials observed, never exceeded +150 mV.EHN.

At Pont Rémy, in spite of the low quantity of treated load, signs of septicity in the medium (substrate and water) appeared shortly after the plant was put into operation (unpleasant odours, whitish bacterial colonies). This condition shows that species presumably capable of transfering oxygen to their roots such as Scirpus, in actual fact, only disperse a small quantity of that oxygen in their immediate root environment which is just sufficient for their growth but not sufficient to achieve a satisfactory treatment. CEMAGREF is presently studying the installation of additional devices which might remedy this situation, such as an auto-starting siphon. A syphon prototype with a mobile arm has been tested at the outlet of the 5th stage at the Pont-Rémy plant since October 1986. Two similar devices are being installed at the outlet of the 3rd and 4th stages.

Numerous tests still remain to be made on these devices to ascertain their efficiency and to determine the optimum operational frequency and amplitude of level variations.

IMPROVING THE EFFLUENT DISTRIBUTION

Presently, the infiltration of wastewaters on the tanks of the first two stages takes place through one inlet only. This deliberate choice was made for the sake of simplicity and to follow the Saint Bohaire example. However, it appears insufficient for tanks with a large surface area. A simple distribution system using overflow gutters is also being tested.

CONCLUSION

An important effort in applied research still remains to be made on the Pont-Rémy site which is representative of many small communities in Europe, particularly in respect of its operating cost and constraints compared with natural lagooning. This is essential in order to determine if the specific surface area could be reduced 4 times as it is hoped.

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ADVANCE AND APPLICATION OF ENERGY-UTILIZATION SYSTEM OF ORGANIC WASTEWATER

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ABSTRACT

The main character of Energy-Utilization System of Organic Wastewater is to combine treatment structures with oxidation ponds, making up their deficiencies. Based on different environmental and economic conditions, various types of treatment forms can be set and work well. The main contents of the system are as follows: 1. to change the microbe phase by using high-energy aquatic plants in place of algal-bactetial system. 2. to make the ponds become a part of afforesting areas of a city; to increase the size of treatment sites by designing the ponds like a tower or steps; If land is limited, pretreatment should be enhanced to reduce the size of ponds. 3. to utilize solar energy to carry out the energy conversion of chemical Exergy, avoiding the conversion of Exergy into Anergy. Then, energy can be got, at the same time, the problem of aquatic plants in the ponds during winter season can be partly solved.

KEYWORDS

Organic wastewater; natural purification; energy conversion; Exergy; solar energy; high-energy aquatic plants; photosynthesis.

INTRODUCTION

It is well known that in urban ecosystem mankind get materials and energy by way of resources from the environment through production and return new materials and energy to the environment by way of products and wastes. Wastes can result in pollution and influence mankind. Then, mankind may connect itself with the environment through material flow, and energy flow(and its information flow) to form an organized urban ecosystem. One of the tasks for optimization of the ecosystem is to give a reasonable distribution to the physical, chemical and biological dissipative forms in the ecosystem, regulate and control material flow and circulation of energy flow so that the urban environment can be in the condition of least-consumption of energy and suitable comfortably to mankind's life, work and production.

Using carious types of natural purifications and energy conversions effectively in wastewater treatment is one of the feasible methods, which gives full play to the self-organizing action of complex systems in nonbalance and nonlinear parts and regulates and controls material flow and circulation of energy flow in the ecosysten. The oxidation pond and treatment structure system is a powerful method mankind use to utilize natural purification capacity and energy conversion capacity of organisms. We call it Energy-Utilization System of Organic Wastewater.

ADVANCE OF THE SYSTEM

In China, oxidation ponds and other treatmant structures were used to treat wastewater of organic materials, but they were always considered contridictive. Since ponds generally use algal-bacterial system, the problems of large size, bad view and hard to pass winter exist. Conventional treatment structures need high construction and performance cost. Therefore the applicable range of ponds and structures is restricted. If they can cooperate to form a complete energy-utilization system of wastewater, making up their deficiencies, based on different environmental and economic conditions, virous types of treatment systems can be arranged, then the systems can work in an optimic condition, which is significant to the organization of urban ecosystem.

In the view of energy conversion, any kind of energy(En) consists of Exergy(Ex) and Anergy(An), i.e. En=Ex+An. The more Ex in energy, the higher the power value that can be used, the higher the energy quality, $\lambda = Ex/En$.

It is obvious that organic material in wastewater is a kind of chemical Exergy. It results from conversion of organism Exergy formed after plants' absorption of solar energy or from conversion of organism Exergy produced by various energy sources in industrial production. But chemical Exergy is difficult to use directly. Without use of it, the chemical Exergy will convert into Anergy. The key point of Energy-Utilization System is to import high-energy aquatic plants to realize the conversion of chemical Exergy and a serious of structures. Then energy conversion of wastewater will be fulfilled under the action of solar light and thermal energy, Exergy can be used effectively instead of conversion into Anergy.

MAIN CHARACTERS OF THE SYSTEM

Being directed against the defects of ponds and conventional treatment structures, we put forward some solutions, which form the main characters of the system, as follows: 1. to change the microbe phase in ponds by using high-energy aquatic plants of strong treating capacity and decorative value in place of algal-bacterial system, then the defects of microbe's death to result in the deterioration of environment in ponds can be overcome, and also the treatment sites become the decorative spots to add additional lustre to the urban environment. 2. to solve the size problem of ponds from three aspects: Decorative aquatic plants are used and ponds are associated with afforesting, then the rate of plant cover increases and ponds become a part of afforesting areas of a city or town; To utilize hillsides and azimuth and altitude degree of the sun, we design a pond like a tower with many stories in it (Upflow Anaerobic Sludge Blanket reactor is installed where sunlight can not reach all the day) and ponds like steps to gain more space for treatment and more energy from the sun , reducing the occupation of land; If land for treatment is very limited, pretreatment should be enhanced(generally with the anaerobic process, aerobic process is also effective) to reduce the load of ponds and the occupation of land. 3. to utilize solar energy for the realization of energy conversion of chemical Exergy, avoiding its conversion into Anergy. The process is as follow : high-energy aquatic plants utilize the nutritional elements in wastewater and convert solar energy into biomass energy through photosynthesis; the plants can be used in anaerobic processes after crushed to change the biomass energy into chemical and heat energy, further, electric energy and mechanical energy can be got; the temperature of wastewater may be raisen with the energy. If the system can be associated with urban planning, we can not only solve the problem of environmental pollution, but also beautify the environment of cities or towns. When the system cooperates with no-soil growing of three dimensions, the land occupation for vegetable growing will decrease.

APPLICATION RESULTS OF THE SYSTEM

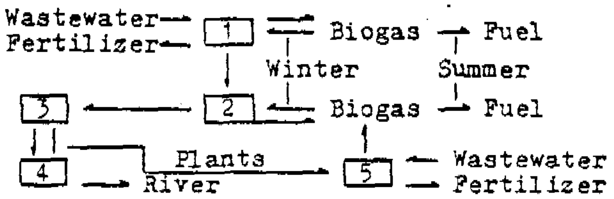
From 1964 on, we often propagated the system so that the department concerned began to accept the plot of the system. Since 1981 we have designed five full-scale practical projects, some of which have been in operation for two years. The area of the test pond in a project is 709 m^2, the flow of wastewater is 150 m^3d^{-1}, then the retention time is 3.8 days, BOD load is 14 $gm^{-2}d^{-1}$. All the data are shown in Table 1.

TABLE 1. Effect Analyses of Wastewater in the Test Pond

	Algal-bacterial Pond Jan. 1984			Water Hyacinth Pond July-Aug. 1985			Water Hyacinth Pond Nov. 1985		
	Raw	Treated	Eff. %	Raw	Treated	Eff. %	Raw	Treated	Eff. %
BOD_5 mgl^{-1}	29.50	23.30	21.0	56.54	11.36	83.0	86.25	4.75	94.5
COD mgl^{-1}	9.88	7.14	27.0	109.78	20.45	81.4	170.72	14.95	91.2
ss mgl^{-1}	12.90	9.25	28.0	44.68	10.53	76.4	59.00	4.00	93.22
Total Count ml^{-1}	7.8×10^5	7.7×10^5	1.5	1.9×10^7	8.4×10^5	95.6	3.2×10^6	8.1×10^4	97.47

* Total Count — Total Count of Bacteria Colonies

The designed flow of the project in a animal products factory is 200 m^3d^{-1}. Two workshops drain off higher organic wastewater. COD may be up to 1420 mgl^{-1} and the flow is from 80 to 100 m^3d^{-1}. As the height in the factory differs much and the site for treatment is very small, we designed a many-storied pond like a tower (see Fig. 2) The flow sheet is shown in Fig. 1.



1— Anaerobic regulating tank.
2— UASB reactor. 3— Ponds in the tower.
4— Ponds. 5— Anaerobic digestion tank.

Fig.1 The flow sheet

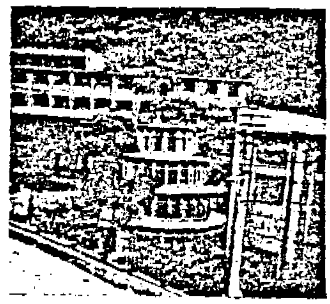


Fig.2 Ponds and the tower

In the middle of the tower is an UASB reactor (soft filler is fixed in it). The tower has four-story ponds and its height is 10 m. The area of the ponds is 108 m^2. If the ponds around the tower are included (180 m^2), the total area will be 300 m^2. Then we get the BOD load of 20 $gm^{-2}d^{-1}$, the retention time of 1 day. The data of the results are shown in Table 2.

TABLE 2 Performance data

	Raw	Treated	Efficiency (%)
COD mgl^{-1}	630.7	49.0	92.2
BOD_5 mgl^{-1}	440.0	35.0	92.0

We are regret to say we have not got data of the biogas tank because the factory is not willing to increase manpower and the structure has not been constructed.

CONCLUSION

It is proved theoritically and practically that the construction and performance cost of the system is lower. The system is easy to run and has good results, therefore it can be used well in many places.

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