

**REPORT OF NATIONAL COURSES
ON WASTE STABILIZATION PONDS DESIGN
AND OPERATION**

Alexandria , Egypt 12 - 21 September 1981

Khartoum , Sudan 12 - 21 December 1981



**WORLD HEALTH ORGANIZATION
EASTERN MEDITERRANEAN REGION**

1985

EM/ES/312

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Alexandria, 12-21 September 1981

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The texts of the lectures presented during the National Courses on Waste Stabilization Ponds Design and Operation, as Part II of this document as submitted by the authors, do not necessarily represent the views or the stated policy of the World Health Organization.

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29 December 1979 - 5 January 1980
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IN DEVELOPING COUNTRIES

PREFACE

In pursuance of a recommendation, made by the participants in the Regional Seminar on Waste Stabilization Ponds Design and Operation held in 1980 in Lahore, WHO/EMRO took steps to promote holding of national courses on the subject in the Member Countries by extending its technical cooperation. Two courses have already been held in Egypt and Sudan. It is proposed to offer technical assistance during 1982-1983 in holding more national courses in four of the Member States. The teaching material evolved during the two courses held in Egypt and Sudan is sufficiently flexible for use for other courses and is therefore issued as Part II of the report.

It is hoped that this publication will further the objective of disseminating information to professionals directly concerned with problems of wastewater disposal and treatment.

PART I
SUMMARY REPORT
of
NATIONAL COURSES

I INTRODUCTION

This report concerns the National Courses on Waste Stabilization Ponds Design and Operation, one of which was held from 12 to 21 September 1981 in Egypt at the High Institute of Public Health, Alexandria, while the second was organized from 12 to 21 December 1981 in Sudan at the Faculty of Engineering, University of Khartoum. In each of the two courses about twenty participants, drawn from different departments of the respective Governments dealing with wastewater systems, attended.

All engineering, scientific and economic aspects were covered in the lectures delivered, mainly by two WHO Temporary Consultants. These were supplemented by seminars, design exercises and field visits.

The participants displayed keen interest throughout and appreciated that stabilization ponds represent a complete system of treatment, for domestic as well as industrial wastewater.

The need for disseminating further information concerning pond design, construction and maintenance was highlighted. The participants were informed of WHO/EMRO's programme of bringing out a publication in the form of a Manual on the subject which would emphasize the design and operational aspects of the system.

II PURPOSE OF THE NATIONAL COURSES

In pursuance of the recommendations on the subject made at the WHO Regional Seminar on Waste Stabilization Ponds Design and Operation held in Lahore, Pakistan, from 29 December 1979 to 5 January 1980, EMRO initiated the conduct of the national courses in Member States. Two such courses have already been held while an additional four are being planned. The purpose of the courses was to acquaint national senior engineers and scientists concerned with the design and operation of waste stabilization ponds with essential background information on the chemistry and microbiology of wastewaters and the conventional techniques employed for wastewater treatment.

III COURSE ACTIVITIES

Preparatory work preceding the Courses was undertaken by WHO Consultant Mr K.V. Ellis by visiting Ministries and institutions dealing with wastewater systems in the respective countries. This resulted in selecting the institution having the necessary facilities and sites for the field visits, and deciding on the number and qualifications of the participants.

The national course in Egypt was held at the High Institute of Public Health, University of Alexandria, for nine working days from 12 to 21 September 1981, while the one in Sudan took place at the premises of the Faculty of Engineering of the University of Khartoum, from 12 to 21 December 1981. About twenty participants attended in each of the countries (See Annexes I and II).

Each course consisted of lectures and seminars and included a design exercise. The early lectures provided background information on the microbiology and chemistry of polluted water and reviewed conventional practices in wastewater treatment. Following this, the theory, operation and design of all types of Waste Stabilization Ponds were considered in some detail. The seminars gave the students opportunities to discuss with course leaders not only the information imparted during the course, but the whole concept of wastewater treatment and the associated objective of disease prevention.

The design exercises involved the wastewater treatment facilities required for a hypothetical new town in an environment identical to that of the country concerned and provided the participants with the opportunity to consider in some detail the actual design of a waste stabilization pond system. For this exercise the participants operated in groups of four or five; the results were very encouraging, demonstrating that all participants had understood the information imparted and would be able to initiate suitable designs for treatment ponds.

The programme of the courses, which were conducted by the two WHO consultants namely Mr Ken Ellis of University of Technology, Loughborough, and Mr J. Lumbers of Imperial College of Science and Technology, London, is given in Annex III.

The teaching material consisted of lecture handouts prepared beforehand for each of the sessions. Full texts of this material are given in Part II of this report.

IV RECOMMENDATIONS

The two lecturers, on the basis of discussions held with participants, made the following recommendations.

1. That a follow-up seminar of three to four days should be arranged for the same participants and perhaps others as well, to discuss the development of stabilization ponds planning in Egypt and Sudan and to consider results of pond operation, as well as reviewing recent publications and discussing the areas in which further research is most required.
- 2 Organization of national courses solely concerned with appropriate methods for treatment of industrial wastewater. These should include recovery of valuable materials, treatment of toxic and intractable wastes, prevention of excessive water and waste discharge and, possibly, single cell protein development from wastewaters with high carbohydrate contents.

PAPER I
WASTEWATER TREATMENT PRACTICES

by
Mr K.V. Ellis
WHO Short Term Consultant

PART II
TEACHING MATERIAL
(HANDOUTS)
COMPLETE TEXTS

Surface water pollution

By pollution we usually mean the addition of harmful (disease producing) organisms to the water or the addition of 'biodegradable' organic material to the water. Wastewater treatment processes greatly reduce the numbers of harmful organisms discharged to rivers, but wastewater treatment is mainly concerned with reducing the quantity of 'biodegradable' material that is discharged.

The main purpose of wastewater treatment is that the bacterial stabilization processes should take place in wastewater treatment works in a rapid concentrated process, rather than more slowly in the river. If the process of stabilization takes place in the river all the oxygen which is normally dissolved in the river water can be used up and then all the normal life in the river (including fish) will die. This must be avoided.

Most wastewater treatment processes are 'aerobic' in that they make use of the same types of aerobic bacteria that exist naturally in the rivers. However, the aerobic processes in the wastewater treatment processes are usually far faster and more concentrated than in the river, and artificial means must be employed (mechanical aeration) to add a lot of oxygen to the wastewater being treated.

The processes of wastewater treatment can be listed as

- Stabilization ponds - anaerobic ponds
facultative ponds
aerobic (maturation) ponds
- Aerated lagoons
Oxidation ditches
Percolating (trickling, biological) filters
Rotating disc treatment units
Activated sludge
Anaerobic digestion

Approximately, these processes are listed in an order of increasing sophistication or complexity. They progress from low technology (ponds) to high technology (activated sludge etc). Percolating filters, rotating disc units and activated sludge processes are called secondary, biological treatment processes as they require the prior removal of the suspended-solids from the wastewater by separate, settlement processes (primary treatment). Most of the processes also require the removal of grit and the screening of the wastewaters (preliminary treatment processes)

Oxidation ditches are a simpler form of the activated-sludge process. Percolating filters and rotating disc treatment units are called 'fixed-film' treatment processes as the aerobic bacteria are held in a fixed position as the wastewater flows past.

Stabilization ponds are the simplest form of treatment and require no skilled maintenance, but use a large area of land. The treated water discharged at the end of the process (the effluent) can be of a high quality but is often contaminated by the presence of large numbers of 'algae'. All the oxygen required by stabilization ponds is provided either by the 'photosynthetic' action of algae or by direct absorption of atmospheric oxygen from the air.

Often the large land area required by stabilization ponds forces designers to use aerated-lagoons which are a lot like the ponds but use mechanical means for dissolving the atmospheric oxygen into the wastewater. Aerated-lagoons require much less land area than stabilization ponds but require power and mechanical equipment and some skilled maintenance.

The activated sludge process requires the development of a large, active mass of aerobic bacteria and other microorganisms (the activated-sludge), which is mixed with the wastewater being treated and later separated from the treated effluent and then used again and again. Activated sludge units need little land area and produce a very high-quality effluent, but require properly constructed concrete channels and tanks, a lot of electrical power, the use of complex and costly equipment and a lot of skilled maintenance.

Most anaerobic 'digestion' units are also costly, complex and require skilled maintenance. They are usually employed to reduce the 'biodegradable' content of very strong wastes to a level at which normal aerobic process can be employed. These anaerobic processes are also often used for 'stabilizing' the sludges produced during conventional wastewater treatment processes. However, 'anaerobic' stabilization ponds are a simple and effective means of employing an anaerobic process without using costly and sophisticated equipment.

'Polishing' processes

Following normal wastewater treatment processes it is sometimes necessary to improve even further the quality of the effluent. This is achieved by one or more of a number of processes which include

Sand filtration - rapid gravity
upward flow
slow sand

Microstrainers
Upflow pebble filters
Maturation ponds
Grass plots

These processes normally reduce further the amount of biodegradable organic material in the effluent and reduce considerably the number of remaining bacteria.

PAPER 2

National Course on Waste Stabilization Ponds :
Design and Operation

Alexandria, Egypt, 12-20 September 1981

INTRODUCTION TO PRELIMINARY TREATMENT .
SCREENING AND GRIT REMOVAL

by

J. Lumbers
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London

PRELIMINARY TREATMENT

1 Definition

Removal of grit, rags and large objects in order to protect pipes from blockage and pumps from undue wear. Preliminary treatment comprises two stages.

1. grit removal
2. screening.

2 Applications & Methods

2.1 Grit Removal

Grit removal is required for all waste waters likely to contain significant quantities of grit i.e. where road or yard run-off enters the sewers.

The settlement of grit in pipes and chambers can ultimately lead to blockage and in tanks a reduction in capacity e.g. in digestion tanks.

Wind-blown sand is a particular problem in some areas.

Grit removal is preferable, but not essential, prior to waste stabilisation ponds. A deepening of the pond at the point of inlet may be provided to allow for grit accumulation, but periodic emptying will still be required.

Grit removal is usually located upstream of all other treatment stages other than coarse-medium screens which are used as a first stage prior to grit removal.

The principal methods of grit removal are.

- A. constant velocity grit channels
- B. detritors
- C. aerated grit channels
- D. vortex grit tanks
- E. piston grit traps

2.1.1 Constant Velocity Grit Channels

Figure 1 shows a plan of a grit channel. The design objective is to ensure a constant velocity of 0.3 m/s in order to settle out grit but not the lighter organic matter. A constant velocity with depth is achieved by using an approximation to the ideal parabolic cross-section.

Designs should always provide at least two channels to allow for cleaning. The liquid depth in the channels is regulated by a downstream control in the form of a standing-wave flume or flow proportional weirs.

The length of the channel is chosen to be about 20 times the maximum depth of flow, to allow for inlet and outlet turbulences and different grit particle sizes. For a particle with a settling velocity of 0.03 m/s the length required to settle in a flow rate of 0.3 m/s is 10 times the maximum depth and so a safety factor of 2 is applied.

Grit is removed manually on small works or by vacuum suction equipment on a travelling gantry on large works. Mechanically removed grit is often washed in a cyclone tank prior to disposal.

2.1.2 Detritors

Detritors, a patented design, are shallow square tanks across which flow is uniformly distributed to maintain an average velocity of 0.3 m/s. Rotating scraper blades are used to move the settled grit to an inclined cleansing channel, Figure 2. The cleansing channel consists of a reciprocating rake which assists in washing the organic matter from the grit. A small pump also provides a counter current of sewage along the cleansing channel to carry away the organic matter.

2.1.3 Aerated Grit Channels

Aerated grit channels are similar to constant velocity channels except that the organic matter is kept in suspension by offset aeration lines causing a spiral flow pattern over the cross-section. The grit falls to hoppers in the floor of the tank and is removed by air-lift or other pumps. Where pre-aeration of a sewage is desirable, for example in warm climates, aerated grit channels serve a useful dual function.

2 1 4 Vortex Grit Tanks

A vortex grit separator is shown in Figure 3 and comprises a circular conical bottomed tank and an adjacent grit washer. Flow enters the tank tangentially in which a vortex is set-up assisted by paddles or air diffusers. Grit falls to the bottom and is air-lifted to the adjacent grit washer. The grit washer comprises a vortex unit below which is a still-water chamber from which clean grit is removed. Organic matter is syphoned back to the main treatment stream.

2 1 5 Pista Grip Traps

The pista grit trap, Figure 4, is another patented device and also works on the assisted vortex settling principle. Grit is air-lifted to a drainage basin and is free from organic matter due to counter-current air scouring in the sump prior to removal.

2 2 Screening

Screening removes rags and other debris likely to cause a nuisance during subsequent treatment. Floating masses of debris would otherwise collect in tanks, chambers or ponds and in addition to being unpleasant to remove can also provide breeding sites for flies. Screening, therefore, is regarded as being preferable at most works.

Where regular cleaning of hand-raked screens or maintenance of mechanical screens is not ensured then, depending on the following processes, it may sometimes be sensible to allow the flow to pass directly to a pond say where intermittent removal or sinking of floating molecules of debris may be the only practical solution, despite the disadvantages.

Where screens are installed (manual or mechanical) it is always necessary to provide an overflow and by-pass facility to prevent localised flooding around the works and "backing-up" in the sewer in the event of the screens becoming blocked.

Screenings removed from the screens must be disposed of in a sanitary manner, either by burial or incineration. Material left lying in the open will attract flies and rodents and represents a health hazard.

Screens may also be used for storm-water overflows and also sea outfalls.

Screens are typically divided into two categories: coarse and fine screens. The dividing size of aperture or bar spacing is about 10 mm. Microscreens or ultrafine screens are usually those with mesh openings of less than 0.065 mm. The principal types of screens in common usage are:

- A. bar screens - vertical or semi-vertical
- B. semi-rotary or rotary screens
- C. hydrasieve or "run-down" screens
- D. comminutors and "screezers"

Typical volumes of screenings in the UK are about 0.01-0.03 m³/1000 people but the amount depends on the:

- o type and characteristic of waste water i.e. industrial content, housing type
- o size of aperture between the bars.

The size of opening between the bars depends on the minimum size allowable forward to whatever process follows. For example the following openings are appropriate:

Location	Minimum Opening (mm)
protection of sewage pumps	100-150
treatment works inlet	15-25
storm overflows	20
sea outfalls	5

Screens are often referred to as coarse medium, fine and micro, and the classification is roughly as follows:

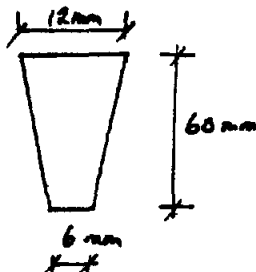
micro-screens (strainers)	≤ 0.065 mm opening
fine screens	≤ 10 mm
medium	10 - 20 mm
coarse screens	> 20 mm

Screens may be either hand-raked or mechanically raked

Hand-raked screens are installed where regular cleaning is guaranteed and where mechanically-raked screens are inappropriate due to lack of skilled maintenance staff, power supply or whatever

All screens are design to ensure no deposition of solids at low flow periods and no "forcing-through" of material at high flows. Typical velocities are not less than 0.3 m/s and not exceeding 1.0 m/s wherever possible.

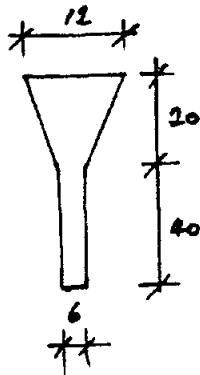
Screen bars are used having various cross-sections.-



tapered - most common and easy to rake



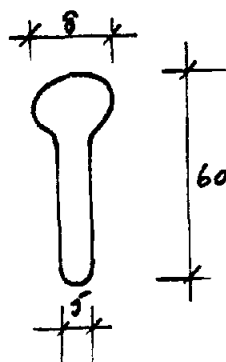
parallel - tend to block more easily



tapered and parallel -



round



profiled

2.2.1 Semi-rotary Screens

Semi-rotary screens (Figure 5) are used in relatively shallow channels of 1.0-2.5 m deep and of similar widths. Minimum openings of 12 mm between bars are usual and larger openings possible according to manufacturer.

2.2.2 Rotary Screens

Rotary screens (Figure 6) describe a full circular motion and are more prone to jamming of the arm than the semi-rotary. Widths of 0.6-1.5 m and depths of up to 1.5 m are typical.

2.2.3 Vertical or Semi-vertical Screens

The vertical or semi-vertical screens (Figure 7) are suitable for much greater depths and have the advantage of lifting the screenings to any height required. The vertical screen is cleaned by a single grab action while the semi-vertical is cleaned by multiple rakes mounted on a continuous belt.

2.2.4 Hydrosieve or "Run-down" Screens

"Run-down" screens (Figure 8) have bar openings of 1.5 mm and a face width of 1-2 m and a height of about 1.4 m. A high level feed box is required and the total head loss required is of the order of 2.4 m which is a positive disadvantage of an otherwise efficient screen with no moving parts. These screens are commonly applied to fibrous industrial wastes with considerable success.

2.2.5 Comminutors and "Screezers"

Comminutors are hollow drum screens vertically mounted with an internal blade which shreds the material caught on the outside (Figure 9). The cutting teeth require regular changing about once a year and problems have been encountered with the tearing of synthetic rag material into strips which become entangled on pump impellers or collect in "stringy" masses.

"Screezers" are hybrid comminutors which also remove screens and dewater and bale them in addition to performing as a comminutor.

2 2.5 Disposal of Screenings

Screenings from most screens either must be moved by hand from the collection trough or washed automatically by a separate system timed to the screening raking mechanism.

Screens are raked automatically either on a regular timed basis or/and by measurement of differential head-loss over the screen. Head differences may be measured by float chambers or ultra-sonic techniques

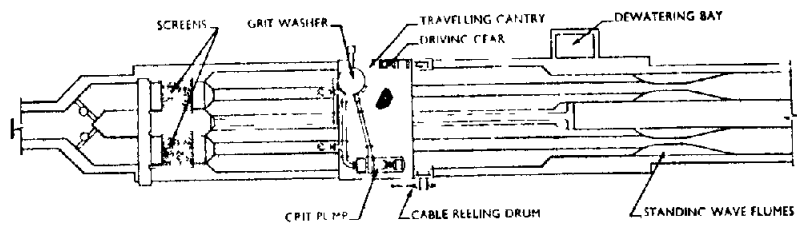
Screenings materials require proper handling and some form of dewatering is desirable in larger works, such as presses or rollers

Figures

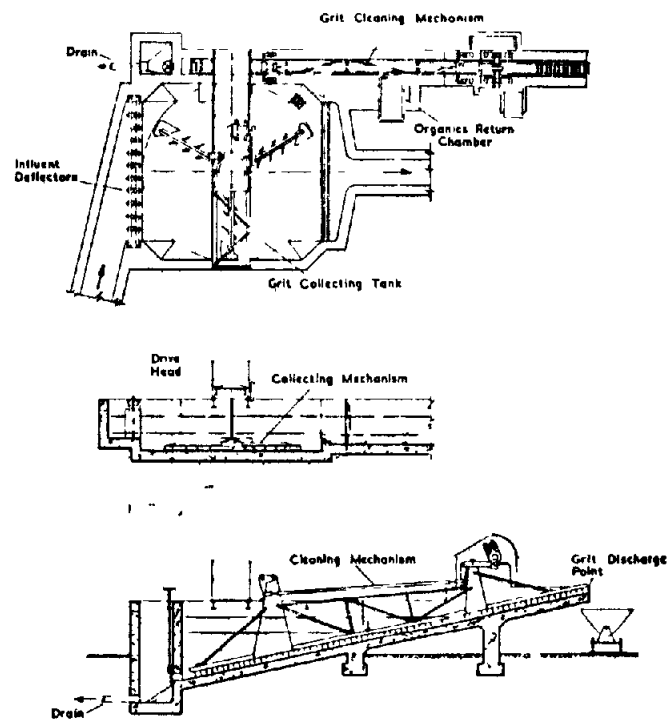
Figure	1	-	Plan of grit channel
	2	-	Detritor
	3	-	Vortex grit separator
	4	-	Pista grit trap
	5	-	Semi-rotary screens
	6	-	Rotary screens
	7	-	Vertical and semi-vertical screens
	8	-	hydrosieve
	9	-	comminutors

References

- o Preliminary Processes (Unit Processes), Manual of British Practice in Water Pollution Control, The Institute of Water Pollution Control, Maidstone, Kent, UK, 1972.
- o Separation, Treatment and Disposal of Screenings, Construction Industry Research and Information Association, UK, Project Report 234, April 1977.

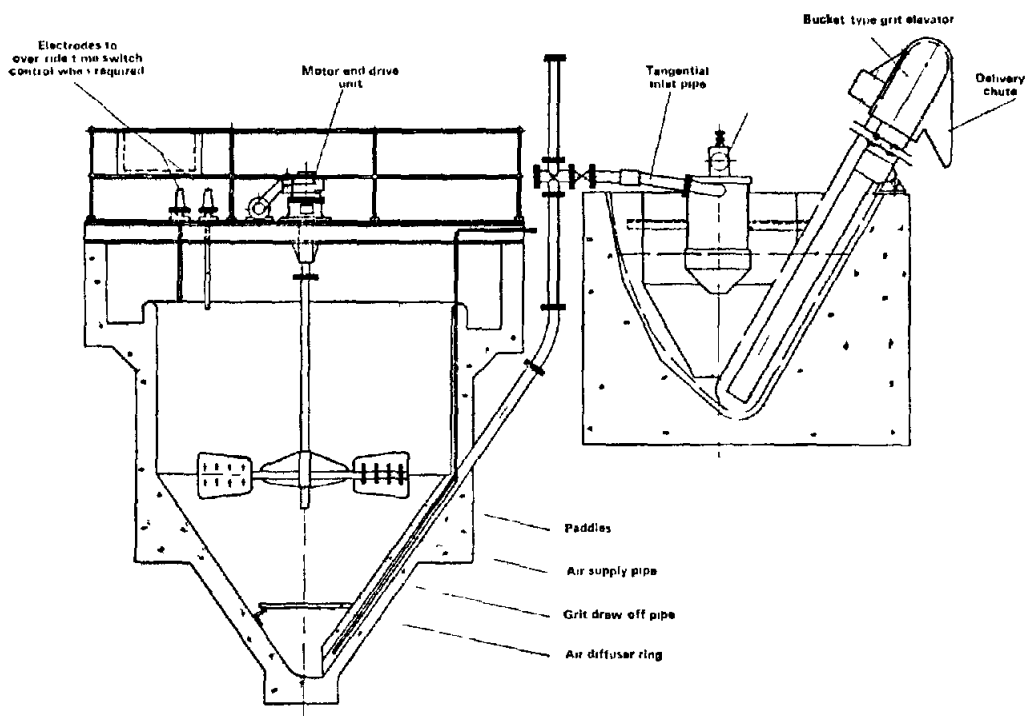


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Control Preliminary Processes (Unit Processes) 1972

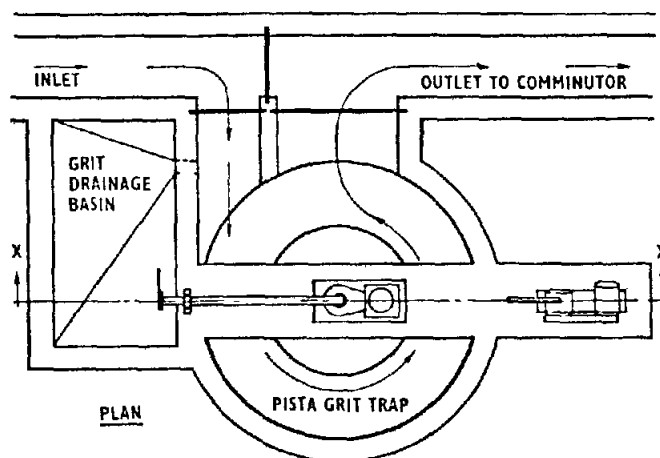
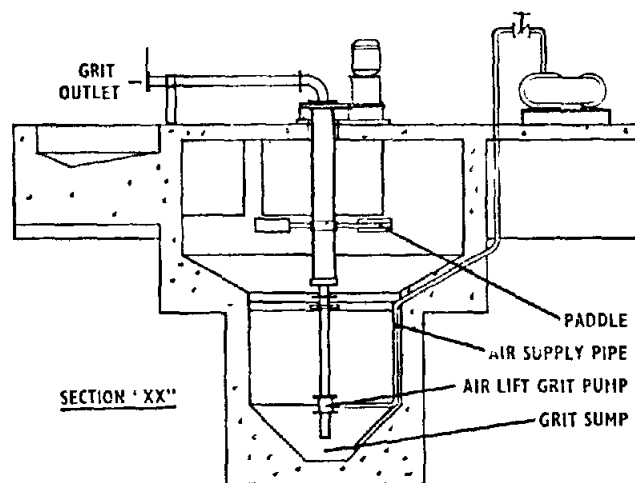


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FIGURE 2



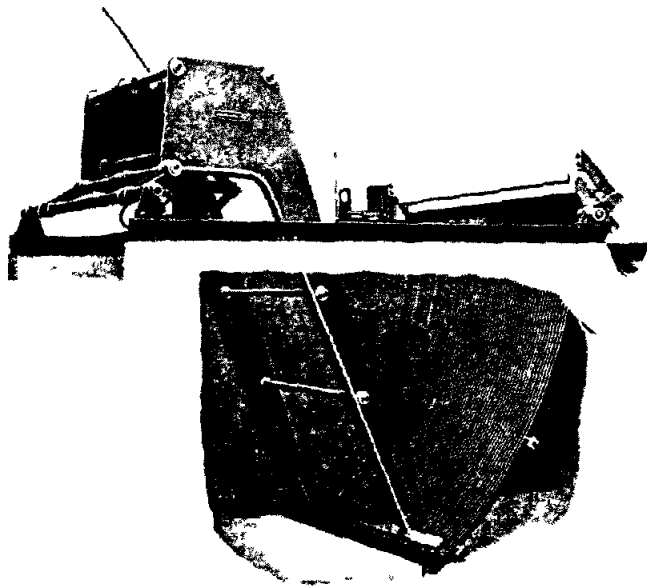
bucket type grit elevator



Pista grit trap

FIGURE 1

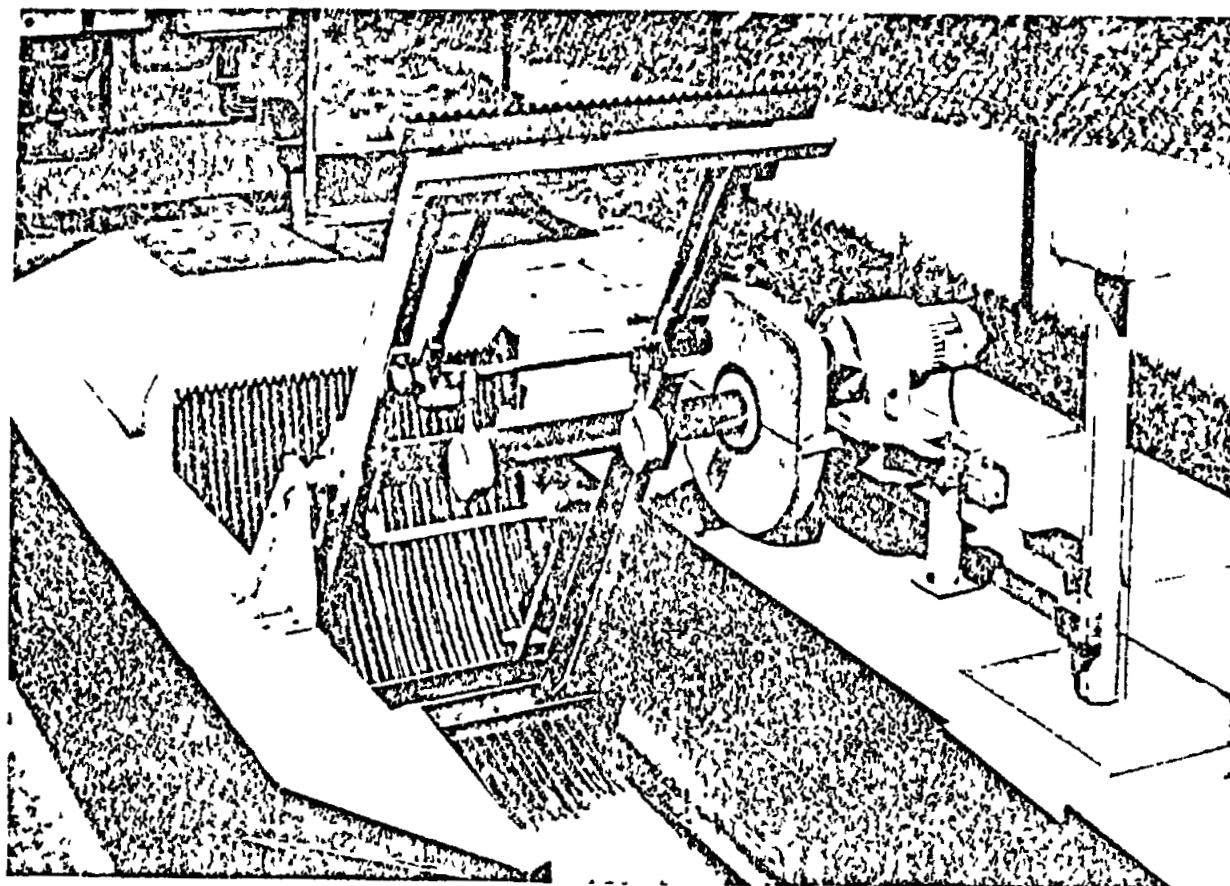
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Curved bar screen with rake drive above water level.

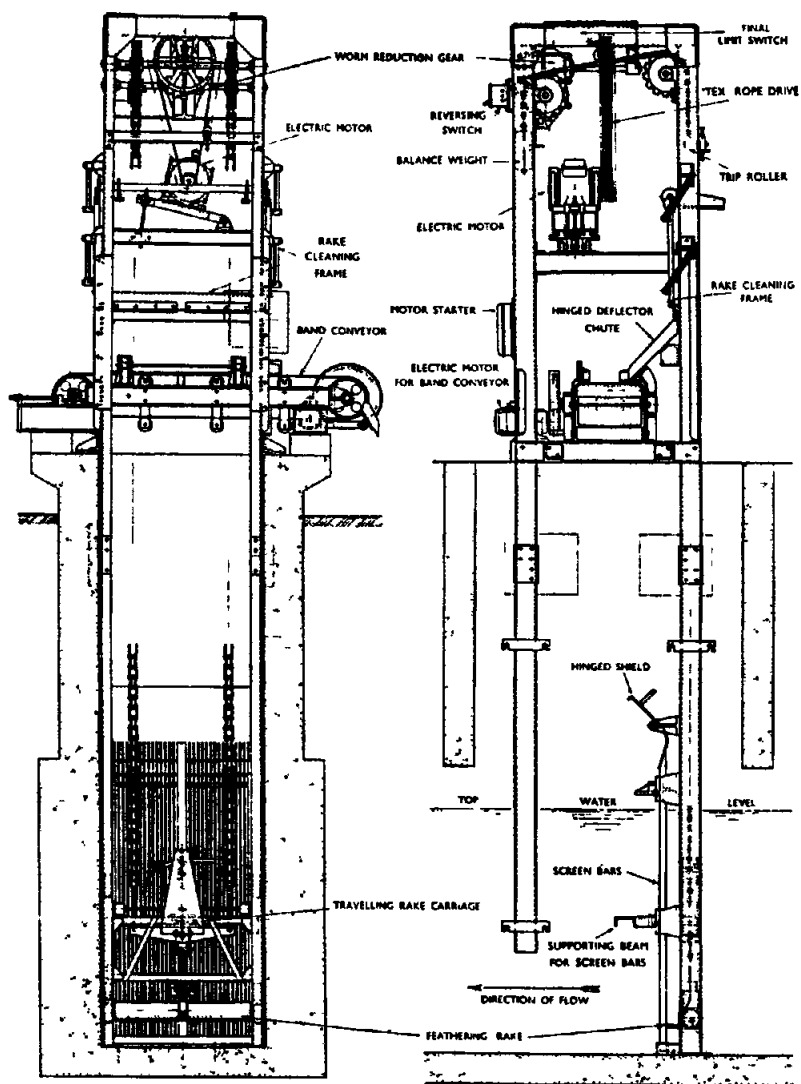
FIGURE 5

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Rotary screens

FIGURE 6

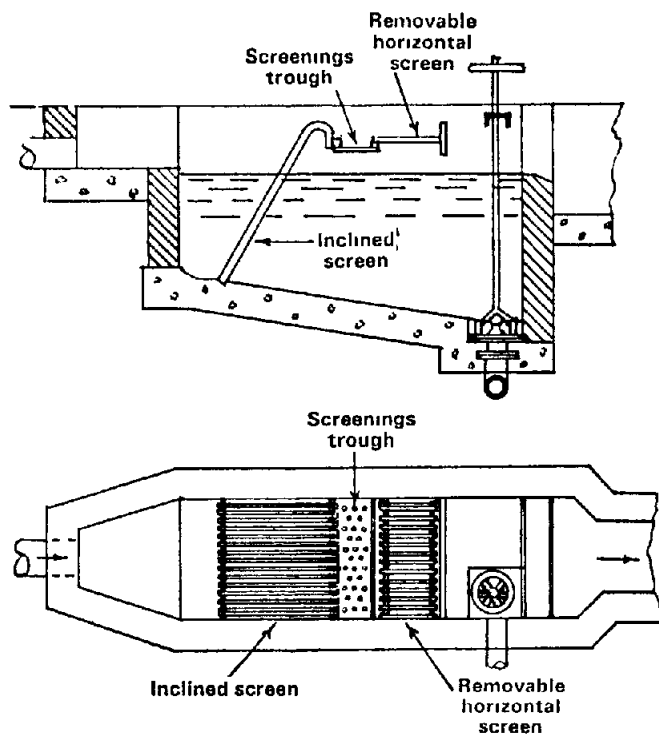


Mechanically-raked bar screen suitable for use in a deep chamber

Vertical screen

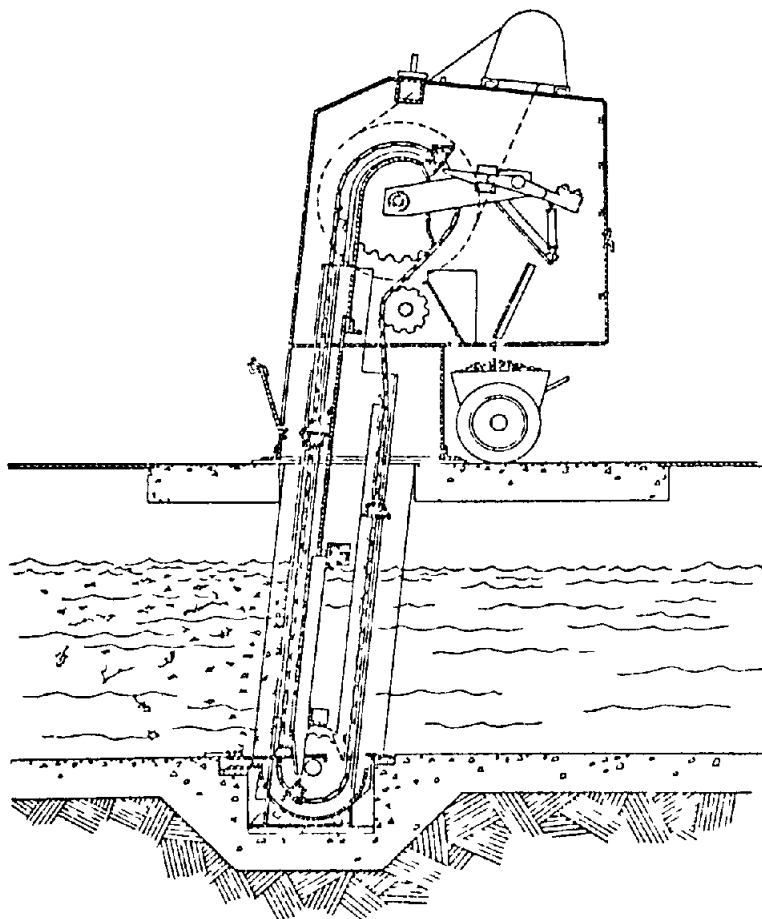
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FIGURE 7a



hand raked inclined screen

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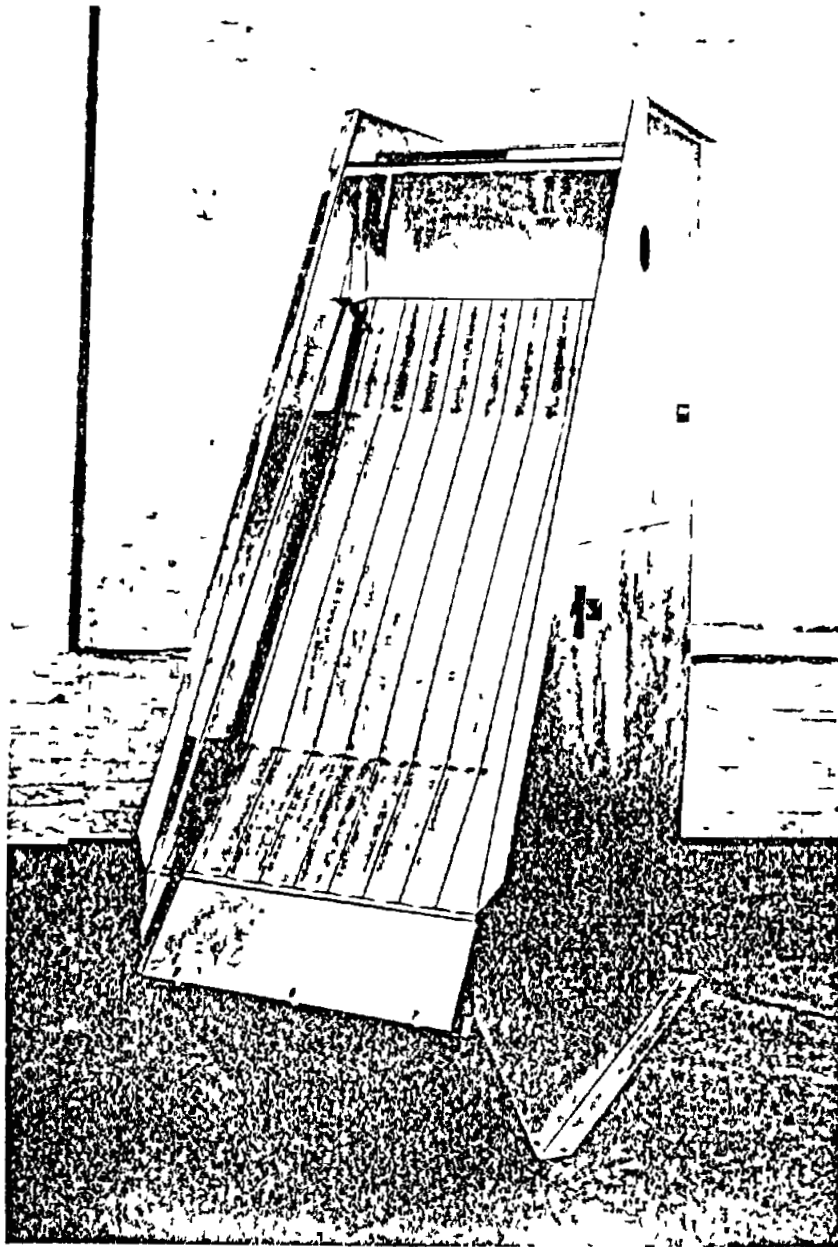


Inclined back-raked screen

(after CIRIA)

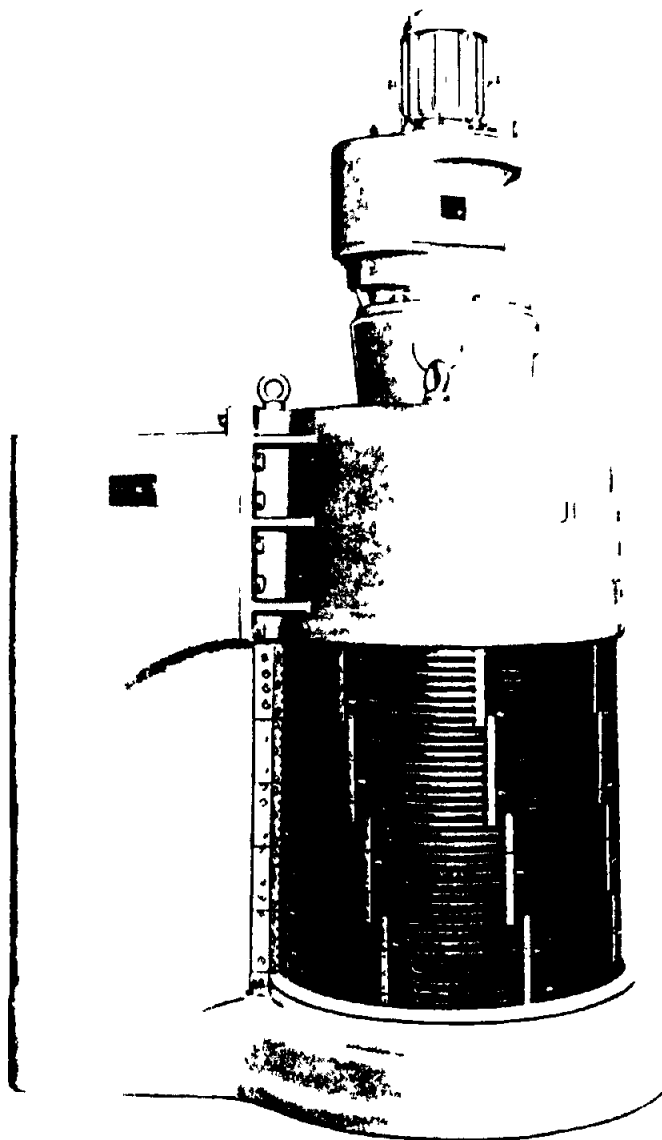
FIGURE 7b

Separation, Treatment and Disposal of Screenings Construction
Industry Research and Information Association UK Project
Report 234 April 1977



Hydrasieve (after CIRIA)

FIGURE 8



Comminutor

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From the Manual of British Practice in Water Pollution
Control : Preliminary Processes (Unit Processes) 1972.

FIGURE 9

PAPER 3

PRIMARY SETTLEMENT

by

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PRIMARY TREATMENT

1. Definition

Primary treatment in the process of removing settleable solids from raw waste water in primary settlement tanks.

2 Application & Methods

Primary treatment in settlement or sedimentation tanks can remove up to 50 percent of the BOD of the raw waste and up to 70 percent of the suspended solids, the subsequent pollution load to the secondary treatment stages is thereby substantially reduced, although the problem of treating and disposing of primary sludge is introduced

Primary settlement is provided on most treatment plants, the exceptions being.

- o waste stabilisation ponds sometimes anaerobic primary ponds or zones employed
- o oxidation ditches "complete" oxidation in aeration tanks
- o rotating biological contactors small plants with integral septic tank.

Primary settling tanks are designed as continuous flow units and are either rectangular or circular Sludge is moved by scraper blades to a hopper or trough and removed by hydrostatic head. In addition to settled sludge removal it is necessary to provide surface scum-draw-off facilities to remove floating debris

Equal flow distribution to each tanks is important and this must be ensured by using properly designed flow-splitting chambers

There are three principal types of primary settlement tank

- A horizontal flow, rectangular
- B. radial flow, circular
- C. upward flow, square or circular

2.1 Horizontal Flow, Rectangular Tanks

A typical horizontal flow settling tank is shown in Figure 1

Where settlement column tests cannot be performed on the waste water to be treated empirical design must be used. There are two basic parameters to be satisfied in design: surface loading and retention period.

For horizontal flow tanks, the maximum surface loading is normally about $30 \text{ m}^3/\text{m}^2\text{d}$ for flow rates up to 3 DWF. Where maximum flow rates exceed this level higher surface loading rates have been used. The ASCE Manual of Practice (1977) gives some guidance on the adjustments to be made for peak flow ratios averaged over different time periods, i.e. $\frac{1}{2}$, 3, 6 hour.

The retention period in primary sedimentation tanks appears to be the most influential factor (given normal design) and a typical value used is 2 hours minimum retention at peak flow rate, giving over 6 hours retention at average flow rates.

In warm climates, where sewage tends to become septic rapidly, retention periods of over 6 hours for much of the time will lead to poor tank performance and serious smell nuisance. Minimum retention periods of $1\frac{1}{2}$ hours or less have been used.

Frequent desludging is necessary to avoid anaerobic activity causing sludge to be lifted to the surface.

Typical dimensions are about 1.5 m deep at the outlet end for small tanks and up to 3.0 m deep for large tanks.

The surface dimensions for small tanks are a breadth to length ratio of 1.2 to 1.4, while larger tanks may be up to 100 m long with the width being dictated by the total area required and the specification of available scraper mechanisms.

Weir overflow rates of not more than $450 \text{ m}^3/\text{md}$ have been recommended although the importance of this factor is not proven as this depends on the other hydraulic characteristics of each tank.

The inlet design is important to avoid resuspension of settled sludge or short-circuiting. The IWPC Manual of Practice (1973) discusses the various designs in some detail with examples of improvements which have been made.

Typical floor slopes of 1 in 40 have been used in small, manually-desludged tanks, while tanks with mechanical scrapers should have a fall of at least 1 in 200. There are two main types of scraper: travelling bridge and flight scrapers, Figure 1. The travelling bridge type has one vertical or trailing scraper blade which is raised on return after sweeping part or all of the tank. The flight scrapers have a number of blades which are driven by a continuous chain and return along the surface

Sludge is collected into hoppers or a trough and withdrawn under hydrostatic pressure. Scum often accumulates in primary tanks and it is necessary to provide scum boards, submerged to a depth of about 500 mm and about a metre from the outlet weir, in order to prevent it passing on to the subsequent treatment stages. An automatic or mechanical scum removal device is desirable. The sludge removed from mechanically scraped tanks has a typical solids content of 3-7 percent.

Rectangular tanks make efficient use of land and are less expensive to construct with water levels 2-3 m above ground than circular tanks, which is sometimes an advantage.

2.2 Radial Flow Tanks

Radial tanks have grown in popularity, perhaps because of their improved appearance, ease of flow distribution and collection of sludge. A typical tank is shown in Figure 2 showing the scraper blades and central sludge hopper

The design surface loading of radial flow tanks is quoted by the IWPC Manual of Practice (1973) as $45 \text{ m}^3/\text{m}^2\text{d}$ at peak flow rate, although the definition of this maximum flow rate is not given. Retention periods of not less than 2 hours at maximum flow rate are typical, but again $1\frac{1}{2}$ hours or less has been used with success.

The typical diameters of radial tanks are in the range of 12 to 30 m and any design should be done in the light of the available equipment. The liquid depth at the side wall should not be less than 1.5 m and is normally about $1/6 - 1/10$ of the diameter. Floor slopes

of $7\frac{1}{2}$ - 10° are typical and the central hopper should be sized to hold about one day's sludge production. Various inlet designs have been tried and no consistent practice has emerged. Generally the inlet baffle or diffuser drum is supplied by the scraper manufacturer.

The outlet weir must again be protected by a scum board and castellated weir plates are commonly employed as they require less accurate setting than flat weir plates or copings.

There are two types of scrapers: fixed-bridge and rotating bridge. Rotating bridge scrapers tanks tend to be used on smaller tanks. Scum removal devices are readily incorporated and occasionally wall scrapers are also included.

2.3 Upward Flow Tanks

Upward flow tanks were once commonly used on small-medium sized works and are still preferable where mechanical sludge removal is not preferred. A typical tank is shown in Figure 3. Because of their great depth they tend to be expensive to construct and are not practicable in bad ground conditions.

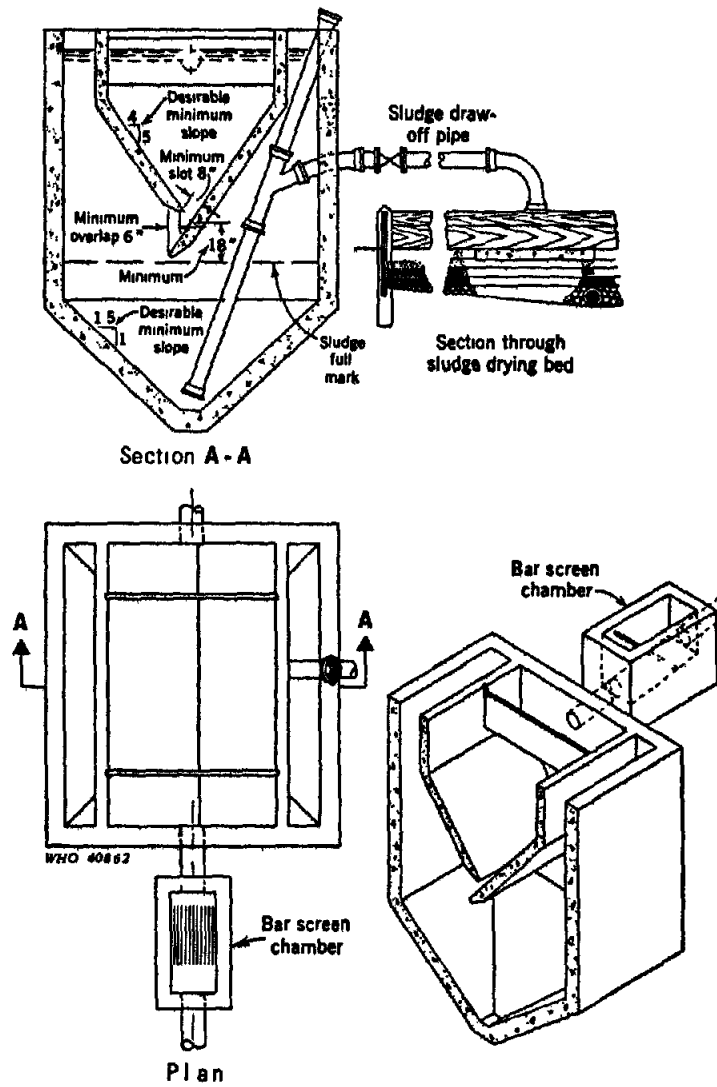
The surface loading rate should not exceed about $29 \text{ m}^3/\text{m}^2\text{d}$ ideally, but rates up to $40 \text{ m}^3/\text{m}^2\text{d}$ have been successfully employed. Minimum retention periods of $1\frac{1}{2}$ -2 hours are common. The slope of the tank walls should not be less than 60° for square tanks and 45° for circular tanks.

Figures

- Figure 1 - Horizontal flow tank
 2 - Radial flow tank
 3 - Upward flow tank

References

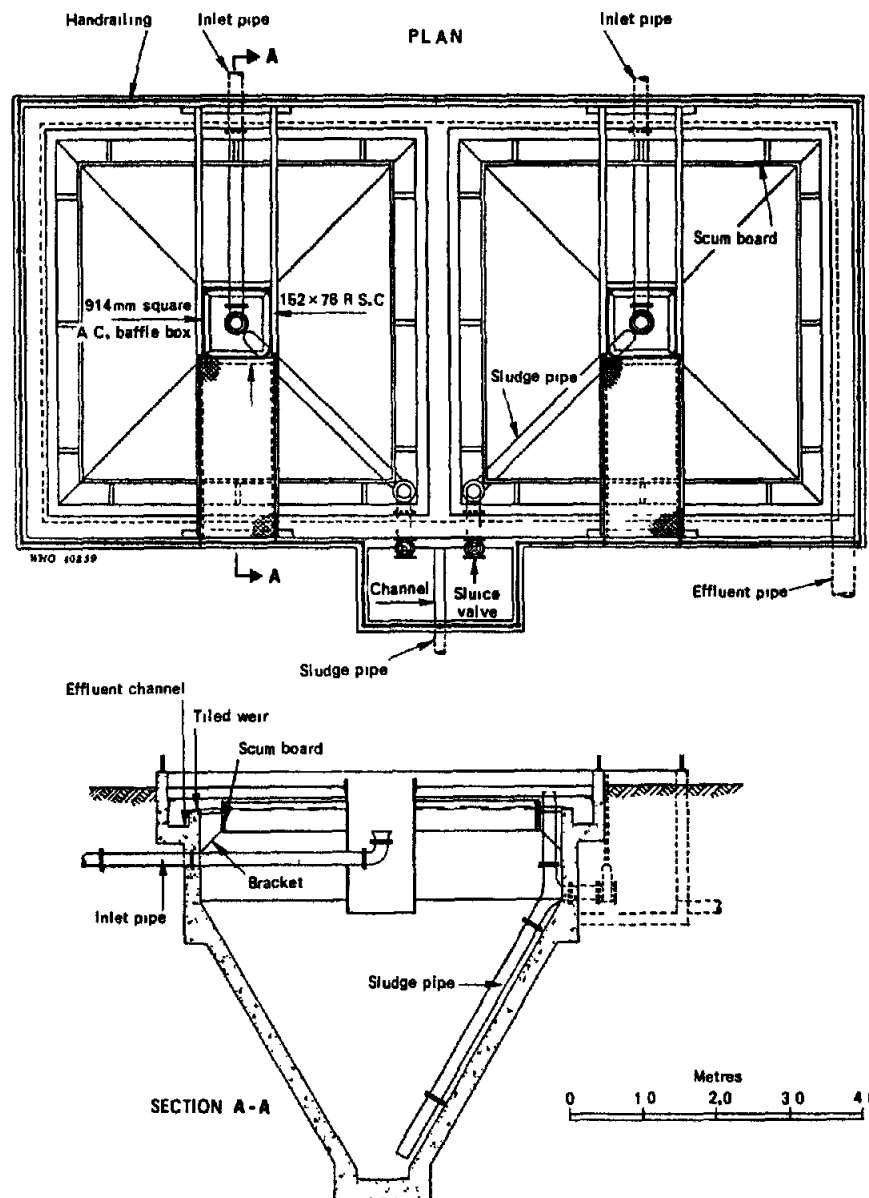
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* Source Phelps (1948), reproduced by kind permission of John Wiley & Sons, Inc. New York

Imhoff tank (after Okun and Ponghis WHO 1975)

FIGURE 4

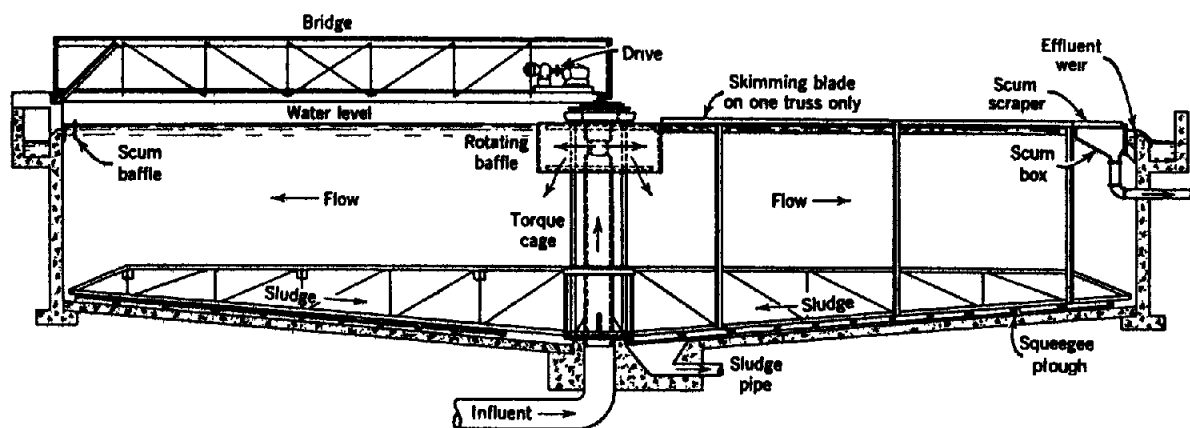


* The square pyramidal shape is suitable for multiple units. Source: Bartlett (1971) reproduced by kind permission of Applied Science Publishers Ltd, Barking, England.

Upflow tank hopper bottomed (after Okun and Ponghis)

WHO 1975

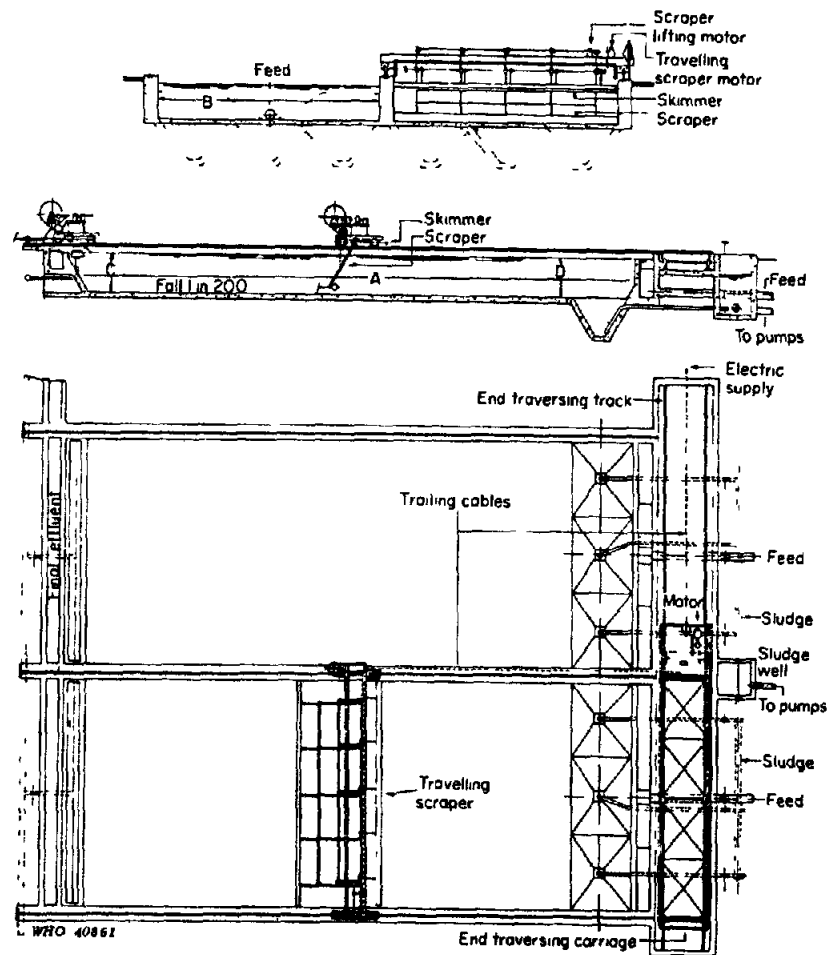
FIG 3



radial flow circular tank Reproduced by kind permission of L G Rich (Rich 1981) source the Link Belt Company

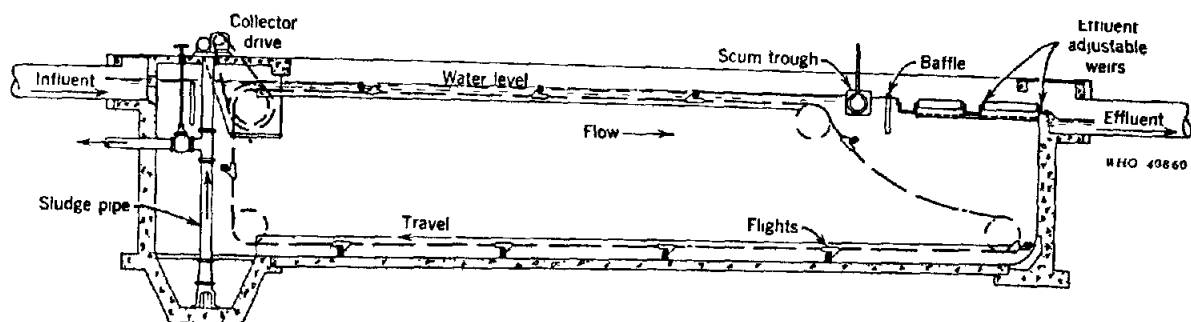
Radial flow tank with sludge hopper (after WHO 1975)

FIGURE 2



* Source: Escrib (1972) reproduced by kind permission of MacDonald & Evans (Publications) Ltd London.

Horizontal flow tank with traveling scraper (after Okun and Ponghis)
WHO 975



Horizontal flow tank with flight scraper (after Okun and Ponghis)
WHO 1975

FIGURE 1

PAPER 4

CHEMISTRY AND PARAMETERS OF WASTEWATER

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Measurement of Basic Water Pollution Parameters and their Interpretation

The basic parameters may be listed as

1. pH
2. Temperature
3. Suspended Solids
4. Dissolved Oxygen
5. B.O.D. (Biochemical Oxygen Demand)
6. 4 hour Permanganate Value ($N/80$)
7. C.O.D. (Chemical Oxygen Demand)
8. T.O.C. (Total Organic Carbon)
9. Total Organic Nitrogen
10. Total Oxidised Nitrogen
11. Free and Saline Ammonia

Dissolved Oxygen

Air is only slightly soluble in water and only - 20.9% of air is oxygen.

The dissolved oxygen content of water varies:

- (i) inversely with salinity
- (ii) directly with pressure
- (iii) inversely with temperature

Normally the pressure effect can be ignored. At 0°C and 100 mg/l Cl the saturation concentration of dissolved oxygen in water is about 14.0 mg/l

at 10°C	11.0 mg/l
at 20°C	9.0 mg/l
at 30°C	7.5 mg/l

On this small amount of dissolved oxygen depends all the aquatic life of a normal stream. The addition of easily bio-degradable organic material to the water (pollution) leads to an increased activity among the bacteria and other micro-organisms which are able to use the added complex organic chemicals as food sources. These micro-organisms are aerobic i.e. they require oxygen dissolved in the water to support their metabolic reactions.

Hence a stimulated activity in these organisms can lead quickly to a depletion in the dissolved oxygen content of the water, to the death of all normal aquatic life, and to complete deoxygenation which is characterised by the emission of hydrogen sulphide.

The rate of reoxygenation of a partially deoxygenated water is proportional to the oxygen deficiency and described by the first order equation. (1)

$$\frac{dC_B}{dt} = k'_2 (C_s - C_B) = k'_L \frac{A}{V} (C_s - C_B)$$

Where C_s - saturation value of the dissolved oxygen

C_B - bulk dissolved oxygen concentration

k'_2 reaeration coefficient (T^{-1})

k'_L reaeration coefficient (Mass transfer units LT^{-1})

k_2 increases with temperature according to the expression

$$k_2 (T^\circ C) = k_2 (20^\circ C) 1.024^{(T-20)}$$

Hence, although the solubility of air in water decreases with increase of temperature the rate of reaeration increases.

The Biochemical Oxygen Demand Test (B.O.D.₅)

When readily degradable organic matter (sewage) is discharged to a stream a natural process of self-purification is initiated. All surface waters contain bacteria and other micro-organisms which are capable of attacking the complex chemicals of pollution and utilizing them as sources of energy and of new cellular material. These micro-organisms are essentially 'aerobic' in that they require oxygen, dissolved in water, in order to carry on their metabolic processes. As this dissolved oxygen is used by the aerobic bacteria, the limited amount available at any one time in water is decreased. The decrease in dissolved oxygen as a result of bacterial activity is directly related to the amount of pollution added.

Basically the Biochemical Oxygen Demand (B.O.D.) is an attempt to utilize this natural phenomenon to quantify water pollution (2,3). The B.O.D. is a measure of the dissolved oxygen required by purifying organisms in a definite volume of liquid in order to break down the available organic material.

If a quantity of polluted water is placed in a stoppered, air-tight bottle, the concentration of dissolved oxygen will decrease according to a first-order equation until, after about 20 days, the action will have effectively ceased. This total drop in the dissolved oxygen is commonly referred to as the Ultimate Oxygen Demand (U.O.D.) but, naturally, the period of the test is far too long for normal laboratory practice. For this reason a standard time for the B.O.D. test of 5 days has been selected and this period is often indicated by the subscript to the letters B.O.D.₅.

Biological activity increases with temperature and therefore the higher the temperature of the test the more dissolved oxygen will be used up in five days. To enable results to be reproducible and comparable a standard temperature of 20°C is always used.

For the same reasons, i.e. those of reproducibility and comparability if it is necessary to dilute the original sample because of its excessive organic strength then a standard dilution water containing prescribed concentrations of ferric chloride, calcium chloride, magnesium sulphate and a buffered phosphate solution is used.

The B.O.D. is therefore an attempt to simulate the natural decomposition of organic wastes through aerobic microbial action under standard conditions and for a standard time.

It is first necessary to decide on a suitable dilution of the original sample that will result in a decrease of the d.o. content of the water by between 3 and 5 mg/l. The sample is then diluted with the necessary volume of the standard, well-aerated dilution water, placed in a suitable air-tight bottle and the initial dissolved oxygen content determined. After five days in a dark incubator at 20°C the final d.o. content is determined and the B.O.D. is reported as mg/l.

The dark incubator is essential as the water sample may contain photosensitive algae which produce oxygen under the influence of light which would invalidate the whole test.

The B.O.D. test is essentially one of carbonaceous oxidation i.e. the breakdown of organic (carbon containing) molecules to carbon dioxide, water and ammonia. It is not a measure of the dissolved oxygen used by autotrophic bacteria in the oxidation of ammonia to nitrite and nitrate. Until fairly recently it was considered that oxidation of the ammonia to nitrates in the B.O.D. test did not commence until after five days and hence was of no significance to the result.

This is now appreciated to be false and under certain conditions ⁽⁵⁾ i.e. the presence of substantial quantities of the nitrite and nitrate-forming bacteria in the original sample together with an initial concentration of ammonia, the oxygen demand of the ammonia oxidisers can be intense and can cause a complete misrepresentation of the carbonaceous oxidation result. For this reason the ammonia oxidising bacteria are normally inhibited in the B.O.D. test by the addition of a small quantity of Allyl Thiourea (A.T.U.)

The micro-organisms in the B.O.D. test are also liable to be inhibited by the presence of toxic elements and compounds (lead, hexavalent chromium, cyanide etc). This inhibition will lead to artificially low results which will only be recognised by comparison with the result of chemical oxidation tests.

The B.O.D. test is generally accepted, for all practical purposes, to follow 'first-order' kinetics in that the rate of the reaction (bacterial decay of organic matter or uptake of d.o.) is directly proportional to the amount of organic matter remaining.

$$\text{i.e.} \quad -\frac{dc}{dt} \propto C \quad \text{or} \quad -\frac{dc}{dt} = k'C$$

where C is the concentration of the organic pollutants at the beginning.

If L, the ultimate B.O.D. replaces C then on integration one obtains

$$\frac{L_t}{L} = e^{-k't} = 10^{-kt}$$

$$k = \frac{k'}{2.303}$$

where L_t is the B.O.D. remaining at any time t.

If y is the B.O.D. already exerted at time t then

$$y = L - L_t$$

$$y = L (1 - 10^{-kt})$$

This is the classic form of the B.O.D. equation. k is the rate constant for the reaction and is normally considered to be about 0.1 days^{-1} although higher figures are obtained with many organic compounds, particularly those in industrial wastes.

It might be considered that the ultimate oxygen demand (U.O.D.) should be equal to the amount of oxygen used in the chemical oxidation of a substance. This is not normally so. A glucose solution with a theoretical oxygen demand of 300 mg/l, i.e. the oxygen necessary to convert the glucose entirely to water and carbon dioxide, will normally only give a U.O.D. of between 250 and 280 mg/l. This is a result of the conversion of a proportion of the glucose into fresh bacterial cells instead of directly oxidising it all completely.

The B.O.D. has many disadvantages which include the inaccuracy of the test procedure, the length of the test, the variability of the rate constant, susceptibility to the presence of inhibiting agents and uncertainty as to what exactly is being measured. The test, however, remains the best single parameter of water pollution and of the bio-availability of organic waste. If possible the B.O.D. results should always be compared with those from a chemical oxidation test of the same sample.

Chemical methods of evaluating water pollution

These methods depend upon the chemical oxidation of the complex chemicals of pollution. Chemical oxidation tests are valuable either as a direct measurement of the polluting organic material present or as a check on the validity (absence of inhibiting effects) of the B.O.D. results. Their general weakness is that they do not oxidise sufficient organic material (4 hours P.V.) or that they oxidise too much. They do not represent a measure of the biodegradable organic material in the water.

Four Hour Permanganate Value Test (N/80)

This test relies upon an acid solution of potassium permanganate to oxidise organic material. It takes place at a standard temperature (27°C), for a standard time (4 hours) and with a standard initial concentration of permanganate $\frac{(N)(M)}{(80)(400)}$. Its principal value is in its ease of performance and the relatively short period of time necessary compared with the B.O.D. test.

There are two 4 hour PV tests; one for wastewater and another, utilizing a larger volume of sample and a smaller volume of permanganate, for potable waters. The wastewater test employs 10 ml of 25% sulphuric acid, 50 ml of $\frac{N}{80}$ potassium permanganate together with 100 ml of the sample. The oxidation is usually incomplete so a precise control of the test is essential, but even so, results are of limited value (6). The ratio of B.O.D. to 4 hour PV varies from about 4:1 for crude municipal wastewater to 1.6:1 for a well purified effluent.

Other Permanganate Tests

A 3-minute test employing $\frac{N}{80}$ permanganate is sometimes used to determine the immediate oxygen demand of trade wastes.

The McGowan strength of a wastewater is still sometimes determined using a stronger, $\frac{(N)(M)}{(8)(40)}$ permanganate solution. The formula

$$\text{Strength of wastewater} = 4.5N + 6.5P$$

is employed where P is the $\frac{N}{8}$ permanganate value and N is the separately determined ammoniacal and organic nitrogen content.

Chemical Oxygen Demand (C.O.D.)

This represents a later development than the 4 hour P.V. test and has now largely superseded it. The C.O.D. relies on the stronger oxidising power of $\frac{(M)}{(48)}$ potassium dichromate solution operating in boiling sulphuric acid. The reaction time is two hours.

In order to assist in the break-down of certain organic molecules such as straight chain aliphatic acids the catalyst silver sulphate is added to the reaction. Despite the presence of the catalyst some compounds possessing a benzene ring structure may exhibit only partial and non-reproducible break-down (7). Pyridine and certain other heterocyclic compounds are not oxidised by this reaction. Ammonia is not oxidised to nitrate, although nitrogen in higher oxidation states in some organic compounds will be converted to nitrate.

Chloride ions interfere and may lead to the formation of volatile compounds that will be lost by volatilization. This interference is largely prevented by the addition of mercuric sulphate which supposedly represses all chloride interference up to 1,000 mg/l. and permits only slight interference up to 1,000 mg/l. In practice the addition of the mercuric salt does not overcome all chlorine interference.

The normal range of application of the C.O.D. test is from about 50 mg/l to 400 mg/l. There is always an appreciable blank and the tests should always be carried out at least in duplicate.

The C.O.D. test is quick, relatively simple and appreciably reproducible. Its disadvantages result from the indiscriminate oxidation of organic matter. Both glucose and lignin are readily decomposed and consequently there is no differentiation between biodegradable and non-biodegradable compounds. The ratio of C.O.D. to B.O.D. increases with the increasing concentration of biologically resistant organic compounds.

Total Organic Carbon (T.O.C.)

With this test all the organic carbon is oxidised to carbon dioxide which is then quantitatively determined. In the wet method (2) the oxidising agent is acid potassium dichromate solution and the CO_2 is collected in a solution of barium hydroxide. The carrier gas is air from which CO_2 has been removed by soda-lime. A dry-method is also employed which uses an electric furnace and nitrogen gas to which about 200 mg/l of oxygen is added

The T.O.C. is rapidly becoming a most popular parameter and there are many different equipments on the market. These frequently incorporate a catalytic furnace operating at about 900°C . The CO_2 produced is determined either by a non-dispersive infra-red analyser or by reductive pyrolysis to methane followed by a flame ionization detector.

Again, with the T.O.C. there is no distinction between biodegradable and non-biodegradable organic material.

Combined Nitrogen Tests

These are the Total Organic Nitrogen, the Albuminoid Nitrogen and the Total Oxidised Nitrogen tests. (2) The organic nitrogen test may be carried out on a sample from which the ammonia and oxidised nitrogen have already been removed and determined; or the oxidised nitrogen may be destroyed and the ammonia included in with the result obtained. Basically, the organic material present is digested at the high temperature produced by a boiling solution of sodium sulphate in concentrated sulphuric acid. The breakdown of the nitrogenous material is assisted by the presence of selenium and copper sulphate catalysts. Later the ammonia is distilled off from an alkaline solution and determined. Most of the organic material will be recovered as ammonia.

The Albuminoid Nitrogen test produces less meaningful results but is still useful, in some situations, because of its simplicity and because of the short time required. The alkaline permanganate solution used possesses only a limited ability to convert nitrogenous, organic material to ammonia and the results obtained should only be regarded as a guide to the total organic content of the sample.

Total oxidised nitrogen includes both nitrites and nitrates. It is best determined using Devarda's alloy on the residue from the free and saline ammonia determination although the copper/zinc couple method is simple and reliable, if overlong. It is rarely necessary or useful to determine nitrites and nitrates separately.

Ammoniacal Nitrogen

Ammoniacal nitrogen is usually determined by distillation from a slightly alkaline solution. The distillate is cooled, collected and made up to a standard volume (200 ml) and then the ammonia content determined by the addition of a small quantity of Nessler's reagent. The Nessler's reagent (alkaline solution of mercuric chlorinate and potassium iodide) produces a yellow colouration in the ammoniacal solution, the density of which is proportional to the concentration of ammonia. The colour density is determined either by direct comparison with prepared standards, or by an absorption meter or by a spectrophotometer (410nm.).

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PAPER 5

PERCOLATING FILTERS

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Biological Filtration of Sewage

Principles of the Biological Process

The biological filter (also known as bacteria bed, percolating filter or trickling filter) provides a large area of solid surfaces over which sewage is made to flow in a thin film, with air in contact to give a continuous supply of oxygen. After a maturing period of two to six weeks (according to temperature and the character of the sewage) a biological film becomes established on the surface. It contains those bacteria and fungi which can, under aerobic conditions, metabolize the organic matter in the sewage to obtain energy and to synthesize their cell material, with the release of CO_2 , H_2O and ammonium compounds. The ammonium compounds are further oxidized to nitrates and nitrates by autotrophic bacteria which usually become established in the lower levels of the filter if the rate of application of the sewage is not too high.

The biological process involves the following steps.

- (a) Flocculation and adsorption of the dissolved and colloidal organic solids on the biological film - sometimes known as "biosorption". This appears to be partly a physical and partly a biological process. Larger suspended solids are also likely to be held by the sticky biological film, but since they may overload the film and lead to blockage of the voids, they should, as far as possible, be removed by sedimentation before the biological treatment.
- (b) Biological breakdown and oxidation of the adsorbed organic material by the respiration of bacteria and fungi. Part of the material broken down by enzymic reactions is then synthesized into new cells.

- (c) Re-solution of products of oxidation, CO_2 , H_2O and ammonium compounds in the flowing liquid film.
- (d) Oxidation of ammonium compounds to nitrite and nitrate by nitrifying bacteria in the lower levels of the filter (under suitable loading conditions). This involves a similar sequence of adsorption, oxidation and re-solution.
- (e) Scouring of the biological film by protozoa, worms and insect larvae prevents excessive build-up of film which would block the filter (though such blockage does sometimes occur in cold weather when the scouring organisms are less active or when the character of the sewage promotes excessive growth of the filamentous organisms). In low- or "normal"-rate filters it is more or less continuous, probably helped by the higher rates of liquid flow. Because the filter effluent contains particles of scoured film, it must be settled in final sedimentation tanks (known often as "humus tanks").

Biology

(a) Bacteria

Primarily Zooglea ramigera, which are gram-negative, capsulated, aerobic, mobile non-chain-forming rods, of several strains. They flocculate sewage solids and form a gelatinous film on the filter media. In pure cultures, Zooglea ramigera can rapidly oxidize carbohydrates and other organic substances, but cannot oxidize ammonia to nitrites and nitrates. Filamentous bacteria such as Sphaerotilus ("sewage fungus") and Beggiatoa (sulphur bacteria) are also usually present. Other

bacteria may develop which can break down specific substances in the waste, e.g. cyanides, phenols, thiocyanates, etc. Nitrifying bacteria Nitrosomonas and Nitrobacter occur usually in the lower levels of a filter.

(b) Fungi

Compete with bacteria for organic food, but do not usually predominate when domestic sewage is being treated. They may find trade wastes more favourable. Bind film together, but may cause blocking by their rapid spread.

Common genera. Fusarium, Geotrichum, (these two become firmly attached and can withstand concentrated jets of sewage), Sepedonium, Ascoidea, Penicillium.

(c) Algae

Restricted to surface of bed where light is available. Play a minor part in purification, but excessive growth may cause blocking.

(d) Protozoa

Mainly ciliates, either attached (e.g. Vorticella, Carchesium, Epistylia), crawling (e.g. Euplotes) or free-swimming (e.g. Paramecium, Colpidium).

Rhizopods (e.g. Amoeba, Arcella, Actinophrys) and Flagellates (e.g. Euglena, Monas) are also common.

The protozoa feed directly on solid organic matter and on bacteria and some take in soluble food. By preying on the bacteria they help to maintain a healthy growth of film.

Different genera tend to become established at different depths in the bed, according to stage of purification and nature of bacteria and other organisms.

(e) Worms

Feed on the film and help to prevent blockage. Their faeces form a large proportion of humus washed through filter.

(i) Nematodes (round-worms). Fine white threads up to 1 or 2 mm long.

(ii) Oligochaeta (bristle worms) Pachydriulus or Lumbricillus, up to 15 mm long x 1.25 mm diameter. Segmented with bundles of bristles or setae. Single worms are yellow, but clumps of worms appear red.

Tubifex, bright red, segmented bristle worm, up to 40 mm long. Lumbricus and other earthworms, up to 75 mm long.

(f) Insects

Many insects breed in biological filters, the larvae feeding on the living film and thus helping to scour it and prevent blockages. Adult flies, however, may cause nuisance in the vicinity, though (except for the midge Culicoides nubeculosus) they do not bite humans.

The following genera are so commonly found on sewage filters that they are often called "filter flies".

(i) Psychoda (3 species, alternata, severini and cinerea)

(ii) Spaniotoma (2 species, minima and perennis are common).

(iii) Metriocnemus (2 species, longitarsus and hirticollis are common).

(iv) Anisopus fenestralis

Other insects which often breed in filters:

(v) Eristalis tenax or drone-fly. Its "rat-tailed" larvae up to 20 mm long may be found in choked filters or in sewage sludge.

(vi) Chironomus or chironomid flies. Non-biting midges which breed in underdrains and effluents channels. Larvae are red ("blood-worms") 20 mm long and are aquatic.

(vii) Achorutes subvaticus or "springtails". Wingless insects about 2 mm long which can leap several inches. Scouring organisms, which may be found in large numbers on surface of filter.

(g) Other animals

Beetles, spiders, mites, snails and slugs may also be found in biological filters.

Engineering Features of Biological Filters

The essential features are.

- (a) An adequate volume of suitable solid medium, which may be supported by walls, or may be self-supporting.
- (b) An underdrainage system to collect effluent and permit air flow through the filter.
- (c) A dosing arrangement to give a periodic sprinkling of sewage on the surface of the filter.

(a) Filtering Medium

(1) Properties

B.S. 1438: 1971 covers single-sized granular media of nominal sizes 63, 50, 40, 28, 20 and 14 mm. The main properties considered are

1. Durability: If the material is known to be satisfactory in practice, no special test is needed. Otherwise, it should pass the sodium sulphate soundness test (20 cycles, each of 24 hours, consisting of 19 hours' immersion in a saturated solution of sodium sulphate at $20 \pm 1^\circ\text{C}$, 4 hours drying at $105-5^\circ\text{C}$ and 1 hour's cooling and examination). The specification gives criteria of unsoundness in terms of number of pieces which disintegrate and percentage loss of weight.
2. Size: Grading limits are given for each nominal size.
3. Shape, Nature of Surface and Classification:

Descriptions, and illustrations are given to explain the terms rounded, irregular, angular, flaky, elongated

as applied to particle shapes. Surface characteristics are classified by the terms glossy, smooth, granular, rough, crystalline, pitted, honeycombed.

4. Flakiness: An "index of flakiness" is specified, determined as in B.S. 812 dealing with mineral aggregates, sands and fillers.
5. Cleanliness. Media should be free of clay, soil or organic or other impurities, and of any substance toxic to living organisms.

(ii) Material used

Almost any durable and chemically inert material may be used, depending upon cost and availability. The most commonly used are granular mineral materials such as broken stone, gravel, clinker, blast furnace, slag and coke. Several designs of thin plastics sheeting (e.g. corrugated sheets such as Surfpac, Flocor or in partitioned tubular form such as Cloisonyle) are available. They have a high specific surface and a large voids ratio, making them particularly suitable for high-rate filtration of sewage or industrial effluents. Their light weight permits construction of deep filters occupying a small area - a useful feature where industrial effluents have to be treated on a limited site.

Certain other proprietary packings of special shapes, such as are used in chemical engineering practice, are also available.

(iii) Specific Surface

Since the work done by a filter depends upon the total area of the biological film it is important that the

medium should have a high specific surface, i.e. surface area per unit volume of filter. (The specific surface of a solid body, e.g. an individual stone in a filter, is the surface area per unit volume of the solid, but in considering the filter as a whole it is more logical to use a unit volume of filter, including the voids).

Examples of specific surface.

Granite, nominal size 25 mm	185 to 237 m ² /m ³
" " " 63 mm	75 to 85 "
Blast-furnace slag, nominal size 25 mm	200 to 246 "
" " " " 63 mm	79 to 90 "
Crushed gravel, nominal size 25 mm	169 to 208 "
" " " " 63 mm	67 to 77 "
Flocor	85 "
Surfpac (standard)	82 "
" (crinkle-close)	187 "
Cloisonyle	220 "
Proprietary ring packings 100 diameter	70 to 110 "
" " " 50 diameter	130 to 200 "

In practice some of the specific surface is ineffective, especially with granular media, because of the area of contact between particles and parts of the surfaces which do not become wetted by the sewage.

(iv) Size of Granular Media

The need for a high specific surface points to the use of media of small particle size, but a limit is met by the risk of blockage of the small voids by the biological

film. The choice of size of media is therefore a compromise based on experience taking into account the nature of the sewage or other waste, the loading rate (both organic and volumetric) and whether the effluent is to be recirculated on to the filter.

For the treatment of domestic sewage, granular media are usually of a nominal size between about 25 and 65 mm. They should be graded within close limits of the nominal size. It is usual to employ a uniform size throughout the depth of the filter, though many older filters were built with a layer of finer material at the top (intended to aid uniform distribution of the sewage) and coarser material at the bottom.

Smaller sizes may be used for weak sewages or some industrial effluents, or where recirculation is practised, or for secondary filters where two-stage filtration is employed).

(v) Depth of Filter

The following factors are relevant to the choice of filter depth.

Rate of application of sewage (organic and hydraulic loading)

Size of media

Time of contact of sewage with film

Head loss between inlet and outlet to filter

Resistance to air flow

Area of land required

Effluent standard required

Practical experience had led to a fairly general adoption of about 2 m as the usual depth for normal-rate filters. Greater depths are commonly used for enclosed filters (4 to 6 m), and filters built of plastics media, especially where space is limited.

Shallow filters (1 m) are used in some recirculation systems and where two stages of filtration are employed.

(vi) Walls

Media may be supported by large pieces of rock built to a steep batter or by brick or concrete walls. Honeycomb brickwork, formerly used to assist ventilation, is not now favoured, since it encourages flies. Solid walls protect the interior of filter from cold winds.

Plastics media may be contained within light steel or timber frameworks with cladding of, e.g. corrugated plastics or asbestos-cement sheet.

(b) Underdrain

Usually concrete floor with half-round or specially shaped tiles, open-jointed brick, etc. Circular filters usually drain to peripheral channel, but sometimes to central wall or to diametral channel. Gradients 1 in 50 to 1 in 100.

Underdrains run only partly full, and hydraulic capacity is not critical, except possibly for very high rate filtration. But adequate cross-sectional area must be allowed above liquid for air flow.

Reasonable air velocity is 1 m/s.

Allowing for a maximum air velocity at surface of filter (i.e. over whole area of surface) of 0.015 m/s, the area of air passages in underdrains required is 1.5% of surface area of filter.

(c) Ventilation

Air movement through filter depends mainly on difference of temperature between sewage and air, and, to a smaller extent, on wind velocity. Air flow is upward unless air temperature is appreciably higher than sewage temperature.

Halvorsen (Sewage Works Journal, 8, 891, 1936) derived formula

$$v = 0.135 (T_a - T_s) - 0.46$$

where v = vertical velocity of air downwards in ft/min
(measured over whole surface area of filter).

T_a = temperature of air, °F

T_s = temperature of sewage, °F

According to this formula, air flow is downward when air temperatures is 3.4°F or more above sewage temperature.

Truesdale and Eden (J.I.P.H.E., 62, Pt.4, 283-302, Oct. 1963) found similar trend, but with lower velocities. They concluded that downward air flow rarely occurred.

With good air access and unblocked filter, the quantity of air flowing is nearly always ample in relation to oxygen required for biochemical action. Truesdale and Eden found that oxygen content of air which had passed through filter rarely fell below 20% (from normal value of 20.9%) and CO₂ content rarely exceeded 1% (normal 0.03%).

Fair suggests that in summer, when oxygen requirements are higher, air flow should be encouraged, and in winter air flow should be kept to a minimum to avoid cooling filter.

(d) Application of Sewage(i) Types of DistributorA. Fixed Nozzles

Used for very large or irregularly-shaped beds.

Nozzles in upper side of distribution pipes laid directly on filter medium. Equilateral triangle layout for most uniform distribution.

Operated by automatic siphons at 2 to 15 minute intervals

For hydraulics of dosing siphons and nozzles see Fair, Geyuer & Okun, Vol. II, pp. 24-3, 24-11 to 24-14 and 35-12.

The head on the nozzles decreases from its maximum throughout the period of operation of the siphon.

Sprays from three adjacent nozzles should overlap at moment of maximum head in order to achieve reasonably uniform distribution.

Head required to operate nozzles through dosing siphon may be up to 2 m, with minimum effective head on nozzle of 0.3 to 0.8 m.

B. Rotating Distributors

For circular filters

Sewage is fed into central standpipe and thence to distributor arms. 2 or 4 arms according to size of filter.

Arms are driven either by

- I. Jet reaction. head of 0.5 to 0.8 m above medium level is required. Jets spaced closer together at outer end of arms to give uniform dosing per unit

area. Or use larger jets. Spreader plates permit larger nozzles at greater spacing, with less risk of blockage.

II. Water-wheels driven by part of the sewage flow.

Head required 0.8 to 1.5 m above medium level.

or III. Electric motors for larger and heavier distribution, or where it is desired to control frequency of dosing.

I and II may be fed through dosing siphons to ensure adequate head to move distributor. Siphons will require extra 1 to 1.3 m head.

C. Rectangular Distributors

Rectangular beds usually laid out in parallel pairs with interconnected travelling distributors to neutralize wind resistance. Sewage is siphoned into distributor arms from longitudinal open channel between the two beds.

Drive may be by waterwheel or electric motor.

(11) Frequency of Dosing

American practice tends towards high frequency, i.e. almost continuous application of fine spray, especially for high rate filters.

In Britain, on normal-rate filters, there is much evidence of improved purification if the interval between applications is lengthened to several minutes - even up to 15 or 20 minutes.

Possible explanations

A. increased scouring by heavy instantaneous flow keeping biological film thinner and more active

B. increased contact time in filter (of part of flow)

C. more uniform film growth through depth of filter.

The lengthened dosing interval appears to reduce growth of fungi, and to encourage fewer flies, but more worms.

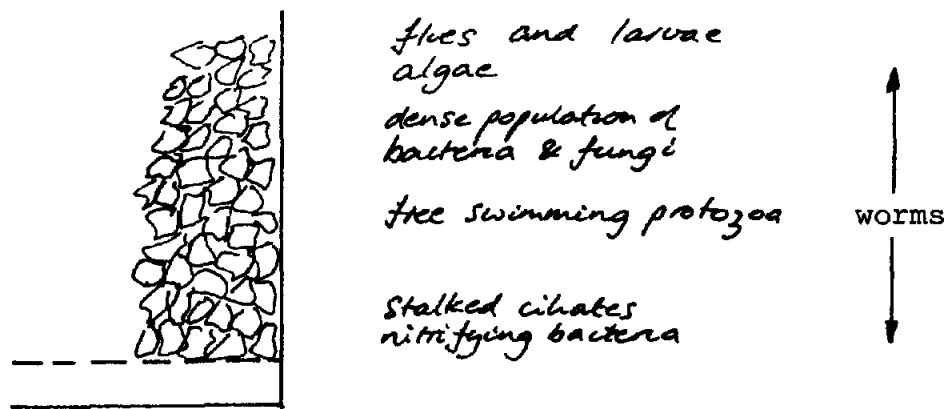
Dosing interval can be lengthened by:

I. reversing direction of jets on one arm of distributor

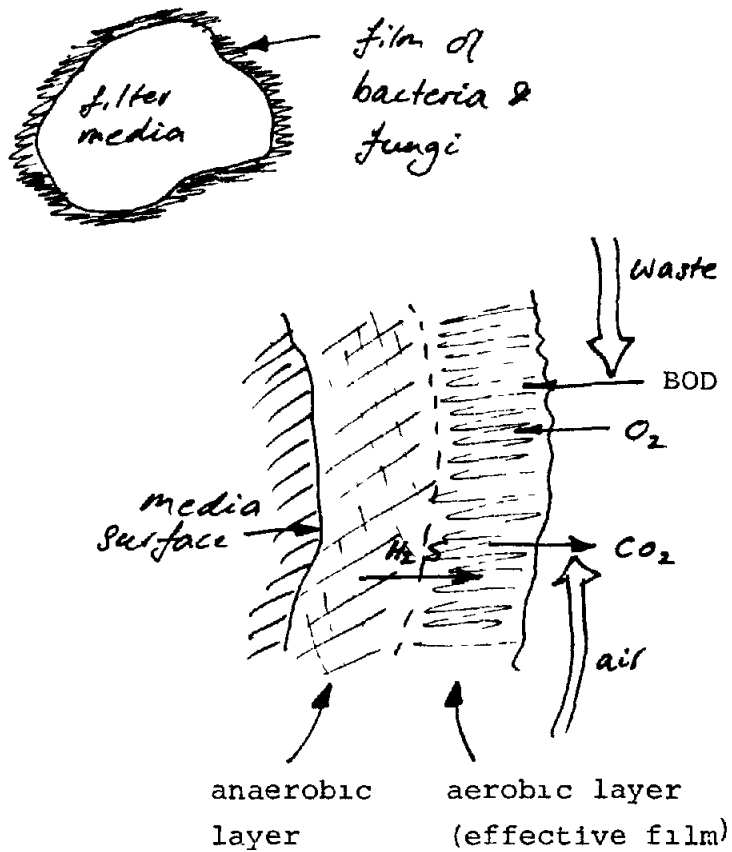
II. using motor driven distributors.

References

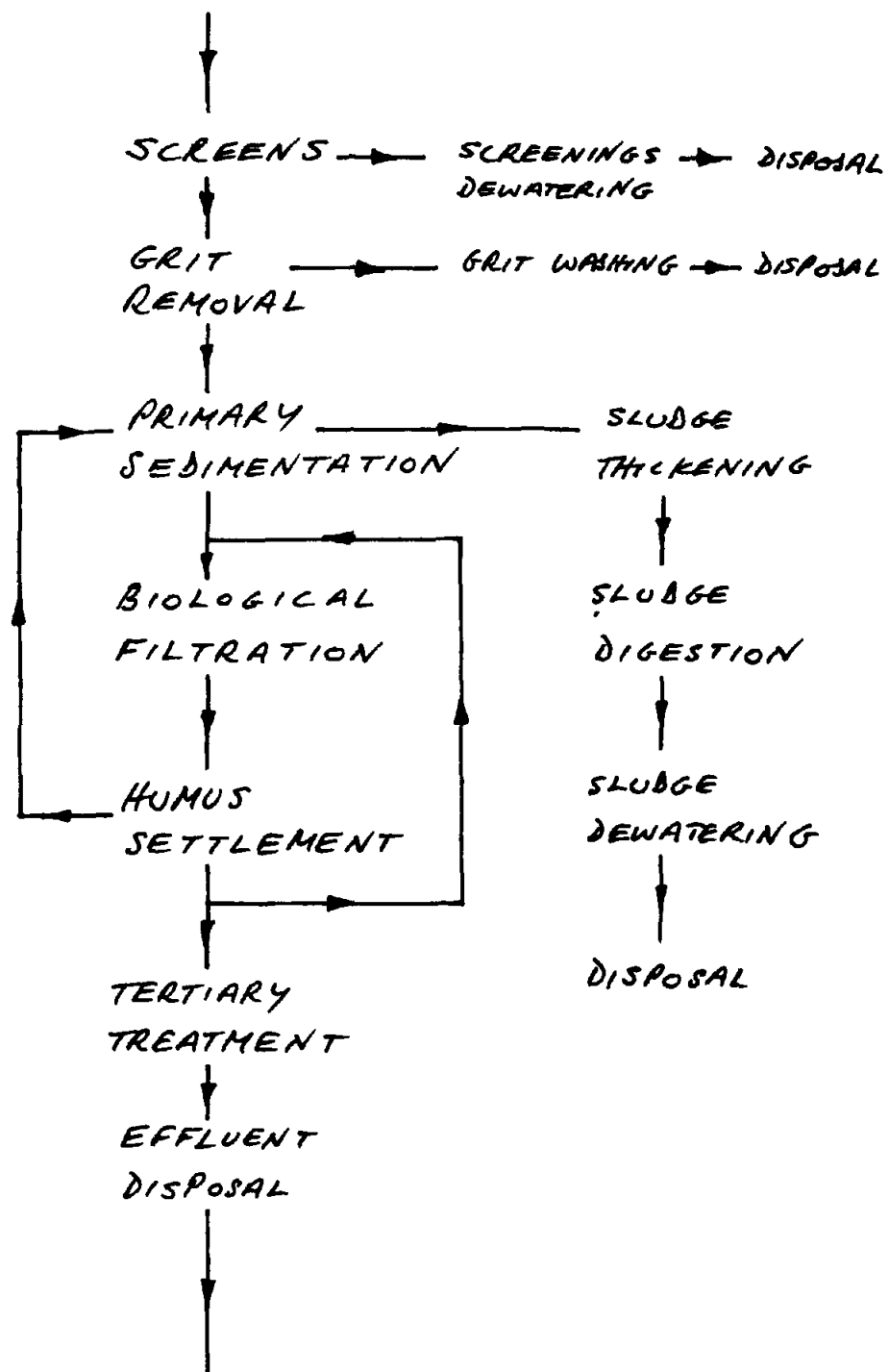
- ° The Design of Percolating Filters and Rotary Biological Contactors, including Details of International Practice. Water Research Centre, TR93, November 1977.
- ° HAWKES, H.A. The Ecology of Waste Water Treatment, Oxford and London, Pergamon Press, 1963, 214 pp.
- ° ECKENFELDER, W.W. Trickling Filter Design and Performance. American Society of Civil Engineers, 128, (III), 371-398. 1983.
- ° British Standards Institution, Small Sewage Treatment Works, CP 302, U.K., 1972.



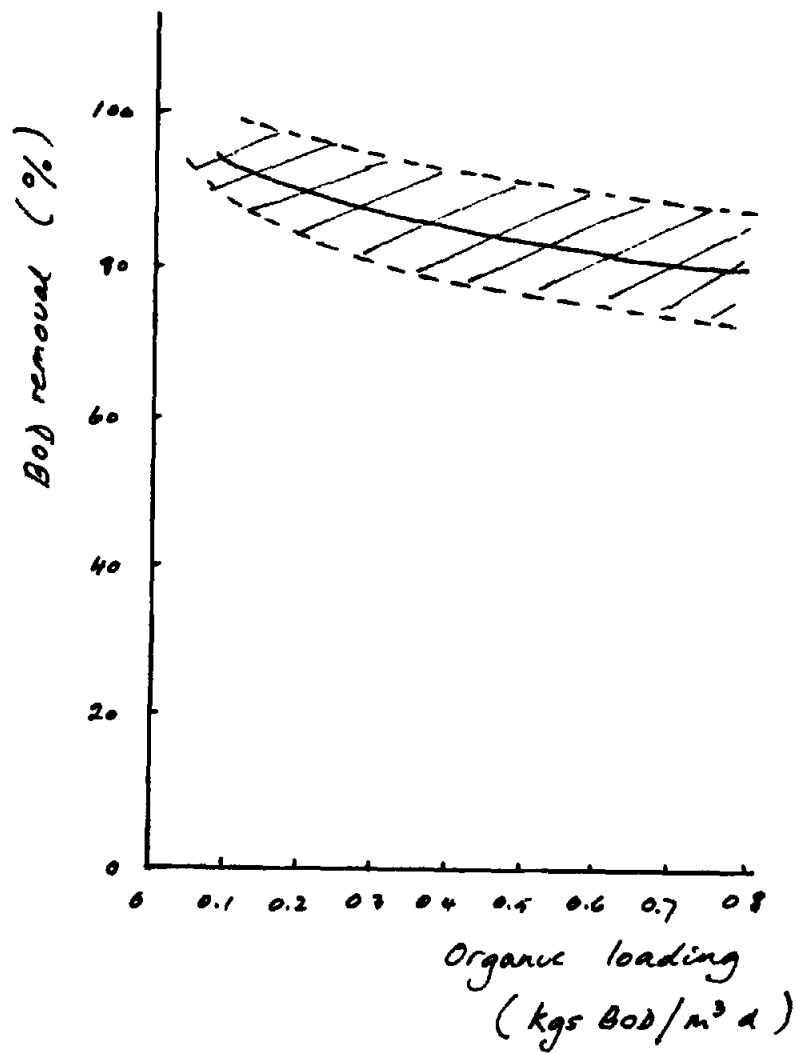
Cross-section through filter bed



The biological film



Typical Process Flow Diagram - Biological Filter



Loading rate (kgs BOD/m ³ d)	BOD removal (%)
> 0.8	70 - 80
0.4 - 0.8	80 - 90
0.2 - 0.4	85 - 95
< 0.2	90 - 95

Typical UK loading rates: 0.12 kg/m³d single pass

0.24 - 0.36 recirculation

0.47 - 1.48 high rate with reare

Loading Rate and Removal Efficiency.

FACTOR	HIGH RATE	LOW RATE
hydraulic loading	9.4 - 37.4 m ³ /m ² d	0.94 - 3.74
organic loading	0.12 - 0.61 kg BOD/m ² d and up to 1.48	0.04 - 0.12
recirculation	1 to 1 up to 4 to 1	none
nitrification	little or none	complete
flies	few	many
sloughing	continuous	intermittant
dosing interval	≠ 15 secs	≠ 5 mins
power required	0.006 kW/m ³	negligible
sludge production	high	average
sludge dewatering	difficult	easy

High rate and Low rate filters.

PAPER 6

ROTATING BIOLOGICAL CONTACTORS:
CURRENT PROBLEMS AND POTENTIAL DEVELOPMENTS
IN
DESIGN AND CONTROL

by

J.P. Lumbers

11 3 JULY 1983

Rotating Biological Contactors: Current Problems and Potential Developments in Design and Control

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Paper presented at a meeting of the Metropolitan District Centre in London on 9th November 1982

SUMMARY

This paper briefly reviews some of the current approaches to rotating biological contactor (RBC) design in the light of two field studies undertaken in 1980. The kinetics of the process are not considered here.

It was concluded that the following areas of design and performance assessment are in need of improvement:

- (i) there is a need to be able to design for and control dissolved oxygen concentrations in the trough
- (ii) RBC specifications should include both oxygen transfer capacities and oxygenation efficiencies
- (iii) the performance of the RBC is most reliably assessed by relating soluble or filterable BOD in the final effluent to the total BOD load
- (iv) an adequate performance of the final liquid-solid separation device is assumed in most designs but is not always realized in practice
- (v) equations relating BOD removed to BOD load (either mass or concentration) by some form of regression equation are invalid due to the two variables being functionally inter-related
- (vi) equal numbers of discs in each stage of multi-staged systems common in many plants may not represent the optimal use of disc surface area
- (vii) the mixing characteristics should be specified
- (viii) the role of biofloculation in enhancing the performance of RBCs requires evaluation

The implications of the above points are discussed subsequently following a brief review of the process and a qualitative examination of some of the current design approaches.

THE RBC PROCESS

The RBC process comprises a semi-immersed cylinder of media or discs on which a fixed biological film is developed by rotating the cylinder about its axis and so passing the biomass through air and wastewater alternately. Rotational speeds of 1 to 5 rpm are common with a typical peripheral speed of 0.3 m/s.

The biological film is similar to that of a percolating filter and the process is capable of achieving normal carbonaceous and nitrogenous oxidation. The process is not exclusively aerobic; anaerobic applications have been reported (Davies & Pretorius (1975), Cheung (1979 & 1980), Murphy *et al* (1977), Sovapak *et al* (1979) and Watanabe & Ishiguro (1978)). This paper is restricted to aerobic applications for carbonaceous BOD removal.

A diagrammatic representation of the RBC process is shown in Figure 1 and three RBC systems are shown in Figure 2.

RBC APPLICATIONS

In Europe there are about 1000 operational Rotating Biological Contactors treating domestic and industrial wastewaters (Pike 1978). Plant sizes in population equivalents exist from 300 to 100 000 with most common applications in the 5 000-20 000 range. In Alexandria, USA, secondary treatment for a population of 400 000 people is provided by RBC equipment.

In the UK the process is winning consideration for medium-scale applications because of its low running costs, compactness, low head loss and minimal smell and noise nuisance.

Compared with conventional activated sludge processes or normal biological percolating filters, the RBC process claims the following advantages:

- (i) RBCs are more compact than percolating filters and hence occupy less site area; equivalent surface loading rates of 5 to 6 times that of percolating filters are possible.

- (ii) RBCs require negligible hydraulic head compared with percolating filters.
- (iii) complete contact between the biological film and waste water is ensured in the RBC compared with percolating filters which often require recirculation to ensure the minimum wetting rate required.
- (iv) RBCs do not require recirculation at times of low flow to keep the biological film wet as do percolating filters.
- (v) the power requirement of RBCs is negligible compared with activated sludge processes.
- (vi) problems of seasonal shedding of biomass from percolating filters are reduced in the semi-controlled environment of the RBC.
- (vii) the fly nuisance associated with percolating filters is not present in RBCs.

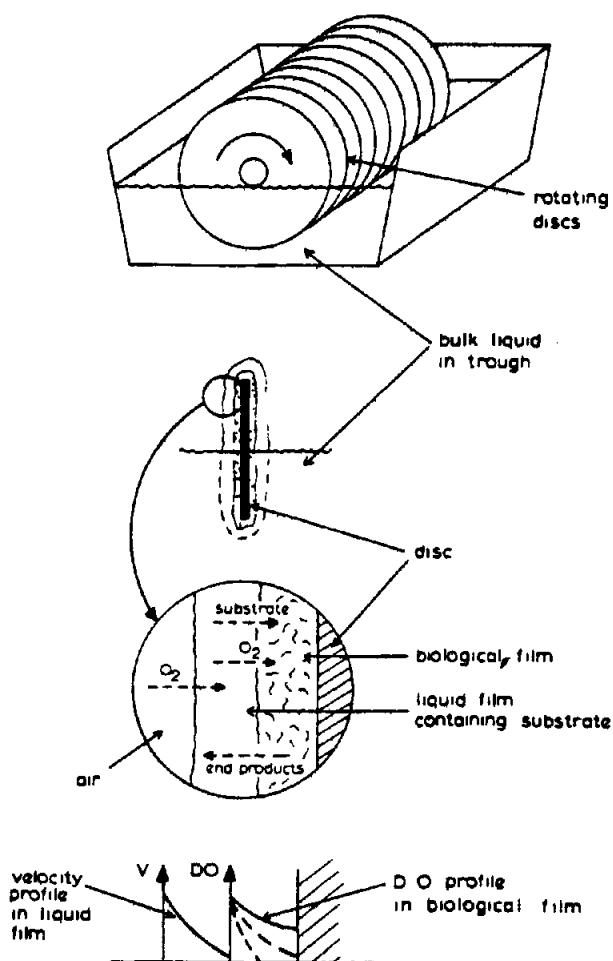


Figure 1 The RBC process

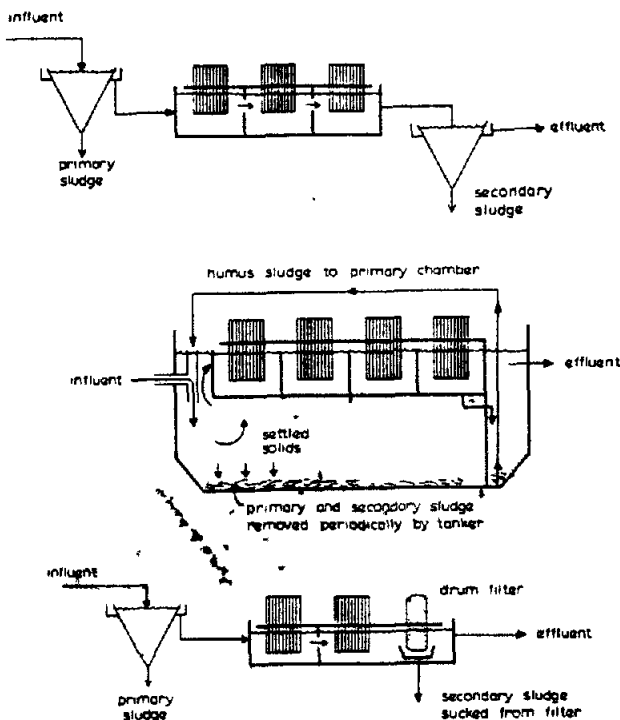


Figure 2 Three RBC system configurations

- (viii) some RBC systems offer a greater potential for operational control than do percolating filters,
- (ix) the disc trough provides additional dilution of shock loads not available in biological filters,
- (x) the short retention period in RBCs results in slowly-degradable suspended particles flocculating in the trough and being removed by the final clarification device before exerting their full oxygen demand

Disadvantages include

- (i) relatively high capital costs of equipment
- (ii) failures in both shaft and drive
- (iii) oxygen deficiencies in the trough liquid,
- (iv) greater and more rapid responses to load variation than in similarly loaded activated sludge plants (Filiop *et al.*, 1979),
- (v) uncertainty in design and performance

SOME CURRENT DESIGN APPROACHES

Most design methods are based on empirically-derived overall organic loading rates per unit area of disc related to the removal of organic load (e.g. BOD_5). Removal efficiencies assume a performance of the liquid-solid separation device (e.g. final settlement). Other methods have related BOD removals to hydraulic loadings per unit disc area only. Many manufacturers quote BOD loading rates per unit disc area that are claimed to produce effluents of BOD and suspended solids not exceeding 20 and 30 mg/l respectively. All designs assume a satisfactory performance of the final clarification stage, although Antonie (1978) recognizes the problem and uses soluble BOD for performance assessment.

The following review of four typical design approaches uses unscaled illustrations from the references to ensure that the source material is always consulted.

Hartman (Steels, 1974)

An empirical design approach relating the percentage BOD removal to the hydraulic loading per unit area of disc was developed by Hartman and reported in English by Steels (1974). The design curves (Figure 3) are for normal fresh domestic sewage and for two packs of discs in series. Correction factors are given for more than two shafts and for peak flows to plants serving more than 10 000 population equivalents. An upper limit to the organic loading of 100 g BOD/m^2d is suggested.

The approach is limited in application in that

- (i) it is only applicable to normal fresh domestic sewage,
- (ii) it is assumed that equally sized stages are optimal

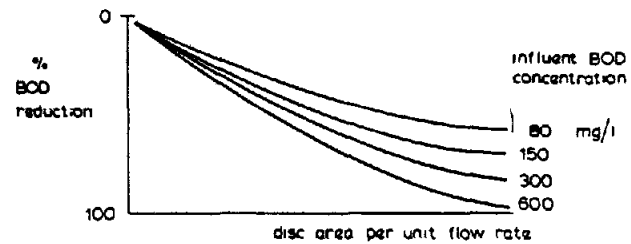


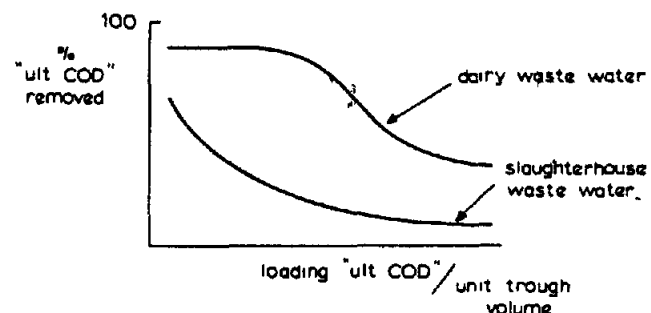
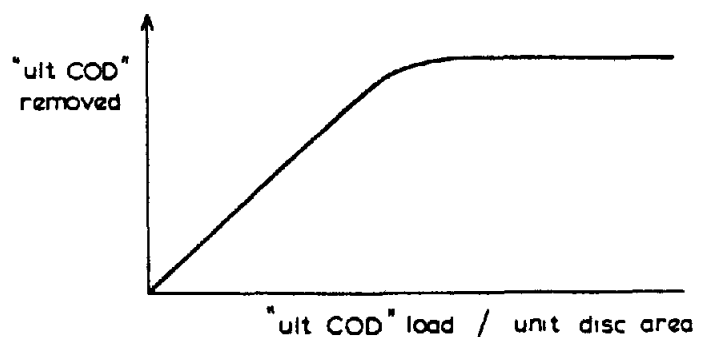
Figure 3 Design curve developed by Hartman (after Steels 1974)

Stover & Kincannon (1976)

An approach based on ultimate COD load is proposed with design curves for various substrates relating percentage ultimate COD removal to ultimate COD load per unit area of disc (Figure 4). These curves are based on graphs relating mass ultimate COD removal per unit area of disc versus ultimate COD mass load (Figure 5). Removal rates were found to be independent of wastewater concentration over the range considered.

Some of the limitations of the approach include

- (i) the ultimate COD as defined will depend on the design of the RBC system and therefore requires pilot studies,
- (ii) the BOD removal is related to the loading per unit trough volume (Figure 4), which will vary for different designs in relation to disc area

Figure 4 Percentage 'ultimate COD ' removal v volumetric loading rate (after Stover and Kincannon, 1976)Figure 5 Ultimate COD ' removal v ultimate COD areal loading (after Stover and Kincannon, 1976)

Pike (1976)

A review of the design approaches for biological contactors concludes that the first-order kinetic equation may be used for RBC design and performance assessment

$$S_1 = S_0 \exp \left(-K \frac{A}{Q} \theta^{T-15} \right)$$

where A = total surface area of disc

S_1 = effluent concentration (e.g. BOD_5)

S_0 = influent concentration

K = reaction rate coefficient

Q = flow rate

θ = temperature coefficient

T = temperature of reaction

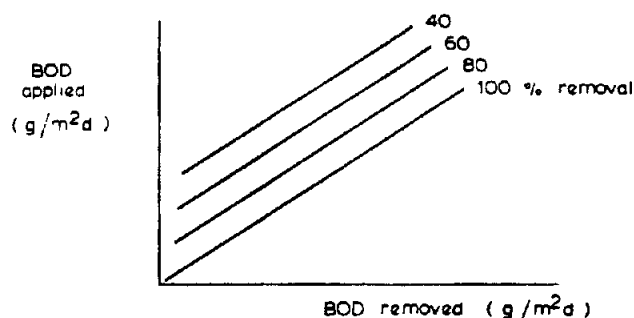


Figure 6 BOD removed v BOD applied for a range of percentage removals (after Pike 1978)

This form of the first-order equation is for plug flow reactors and whilst it is appropriate for trickling filter design it is not so for RBC units which are nearer to being completely mixed reactors. A graph of BOD applied versus BOD removed is given with lines for various percentage removals (Figure 6).

The design approach has two important drawbacks:

- (i) the graph of BOD removal versus applied BOD load provides little information as the variables are functionally inter-related
- (ii) the suggested exponential performance function assumes a plug flow regime

Antonie (1978)

Antonie (1978) presents a design approach developed from the work of McAliley (1974) which relates soluble BOD removal to influent soluble BOD concentrations. The design parameter used is soluble (filterable) BOD influent concentration on the basis that settleable BOD is removed in the primary settlement tank and that suspended solids passing forward are rapidly bio-flocculated and removed in the final clarifier without exerting a significant oxygen demand on the system. Plots of soluble BOD removed versus soluble BOD influent concentration are given with regression coefficients in the order of 0.9 (Figure 7). This high regression coefficient is spurious as both sides of the regression are interrelated, BOD removed being a function of BOD influent.

Relationships such as in Figure 7 were used to derive the proposed design chart (Figure 8) for multi-staged units.

Figure 9 was derived from Figure 8 to predict effluent soluble BOD from influent soluble BOD for given hydraulic loading rates to a four equal sized stage system.

The design basis has some difficulties, including:

- (i) the regression equation in Figure 7 is invalid due to the functional inter-relationship of the variables BOD removed and BOD loading. Furthermore, the line would reach a plateau and then perhaps decline when the rate of oxygen transfer becomes limiting.
- (ii) Figure 9 predicts only the soluble BOD in the effluent and a factor of 2 is suggested to convert to total BOD, such a ratio cannot be applied universally and in particular depends on the efficiency of the final clarifier which is assumed.
- (iii) the term 'soluble' needs defining, i.e. specification of filter paper.
- (iv) the BOD used in developing the method was not nitrification suppressed.

FIELD STUDY EXPERIENCES

Two field investigations were undertaken in 1980 evaluating the performance of two RBC systems, one treating rural domestic sewage

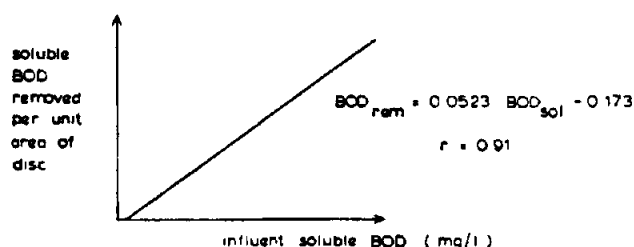


Figure 7 Regression line of soluble BOD removed v influent soluble BOD (after Antonie, 1978)

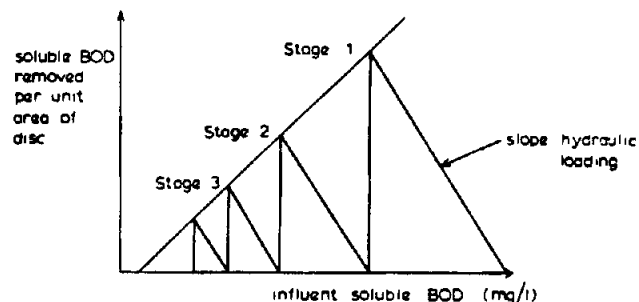


Figure 8 Multi stage RBC design chart (after Antonie 1978, McAliley 1974)

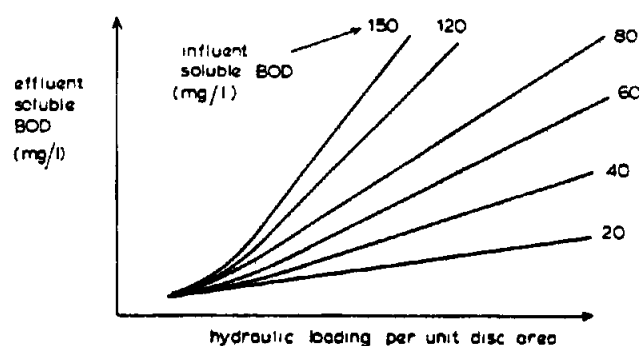


Figure 9 Design for specific effluent soluble BOD concentration (after Antonie, 1978)

and the other a full-scale pilot plant treating dairy wastewater (Santos 1980 and Thompson 1980). The domestic sewage plant was multi-staged and over-sized in terms of disc area, whilst the dairy waste plant was a single stage and purposely over-loaded during pilot study trials. Another contrasting feature was the final liquid-solid separation device which for the domestic plant was a rectangular hopper-bottomed tank, and for the dairy waste plant was a rotating drum filter. Both plants were unable to achieve their designed performance but for different reasons.

For the domestic sewage plant only about 30 per cent of the bulked daily samples complied with both criteria of less than 20 mg/l BOD and 30 mg/l suspended solids. During the period studied the average effluent BOD₅ (ATU) was 21 mg/l with only 55 per cent less than 20 mg/l. However, during a week when measurement was made of both total and soluble BOD (filtered through a standard glass fibre filter paper (Whatman GF/C grade or equivalent)), an average BOD₅ (ATU) of 22 mg/l corresponded to a soluble BOD₅ (ATU) of 6 mg/l and an average suspended solids of 51 mg/l. Clearly, poor performance in the final settling tanks was indicated, this was observed to be due to both rising sludge and severe hydraulic short-circuiting.

An attempt was made to apply some of the derived performance equations reported in the literature such as those by Schulze (1960), Eckenfelder (1978), Antonie (1976), Kornegay (1975) and Popel (1964) but with no success. All these models assume an adequate performance of the final liquid-solid separation device which in this case was performing inadequately.

With regard to dissolved oxygen concentrations in the trough, some useful observations were made from 24-hour data. Dissolved oxygen concentrations were observed to respond rapidly to changes in the incoming BOD mass load (Figure 10).

A plot of the dissolved oxygen concentration in the first stage trough against the incoming mass BOD loading is shown in Figure 11; an exponential relationship might be suspected. Whilst such a relationship would give a general indication of the probable dissolved oxygen concentration associated with an organic loading rate, it provides no information regarding the dynamic response of the system. Considerable high frequency data are required to identify such dynamics as emphasized by Filion *et al.* (1979) where sampling frequencies at every 30 minutes were used and a response in TOC measurements observed within this period.

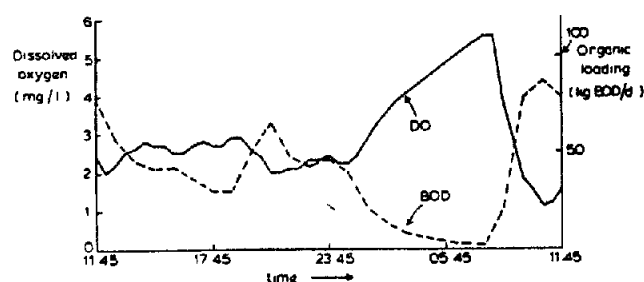


Figure 10 Time series of dissolved oxygen in trough and mass organic loading (Source Santos, 1980)

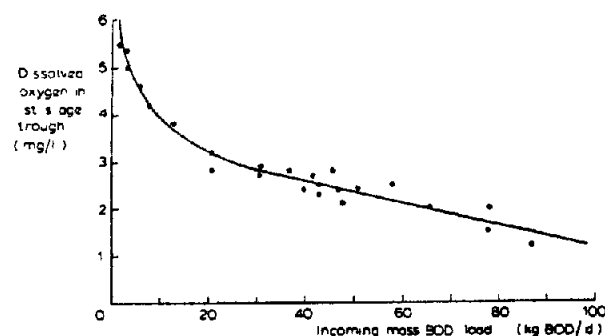


Figure 11 30-minute lagged dissolved oxygen v. mass BOD₅ (ATU) loading (Source Santos, 1980)

The dairy waste plant was intentionally over loaded during full-scale pilot studies. It was observed that poor organic removals were associated with a zero dissolved oxygen in the trough. High suspended biomass concentrations in the trough arising from severe toxic shock loads causing the biomass to shed from the discs also gave rise to poor performance. Satisfactory effluent quality was only achieved when a dissolved oxygen concentration of about 2 mg/l was present in the trough, this being attained by lowering the organic loading rate and/or reducing the high concentrations of solids present in the trough.

Again only limited data were collected relating soluble BOD in the effluent to the total BOD loading (Figure 12). It is interesting to note that a regression of soluble BOD removed versus soluble BOD load (as by Antonie (1978)) gave a regression coefficient of 1.00, but the regression analysis is not valid as previously noted. In Figure 12 both variables are merely divided by the constant disc area. Obviously a regression analysis of so few data points is suspect.

DIRECTIONS FOR IMPROVED DESIGN

From the foregoing, it is concluded that the following areas have potential for improving design.

Dissolved Oxygen

Many workers have referred to the importance of a residual dissolved oxygen in the trough, for example, Pike *et al* (1980), Hynek and Chou (1979), Bintanja *et al* (1975) and Chesner and Molof (1977).

Figure 11 shows that the dissolved oxygen in the trough should be predictable with confidence from the incoming mass BOD loading rate, for the particular oxygenation capacity of the plant considered, the respiration rate of the wastewater and the expected diurnal load variations.

A knowledge of the response of the dissolved oxygen concentration in the trough to input load variations would prevent over sizing to meet short duration demand peaks. Further, an understanding of the dynamic response of dissolved oxygen would allow the evaluation of operational control.

The RBC should be regarded primarily as an oxygen transfer device and the oxygenation capacity and efficiency should be specified, Steiner (1979) argues strongly for this approach. The importance of mixing on the mass transfer coefficient was identified by Zeevalkink *et al* (1979), who performed laboratory scale aeration tests.

Final Liquid Solid Separation

The incorporation into design and performance evaluation of an assumed performance of the final liquid-solid separation device has led to a neglect of the essential rôle of final effluent clarification. However, a

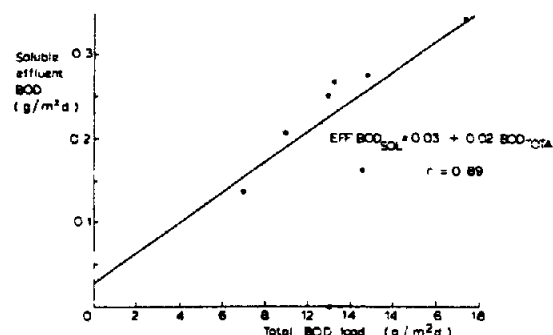


Figure 12 Effluent soluble BOD v. total BOD load for dairy waste (Source Thompson 1980)

performance specification for final settlement or filtering requires a prediction of the concentration and nature of the suspended solids passed forward from the disc chamber. This problem has resulted in the use of soluble or filterable BOD values in the effluent as a better measure of RBC performance. Performance investigations should use this approach in order to identify the separate efficiencies of the RBC and the final liquid-solid separation device.

For design, however, the use of soluble BOD predictions is of little worth until the relationship between the final total BOD and the soluble BOD can be reliably forecast. It has been claimed by Antonie (1978) that significant bioflocculation of suspended particles occurs in the trough. If this is the case then the nature of the floc particles produced will affect the choice of the final liquid solid separation device. The rôle of bioflocculation in the RBC process therefore requires evaluation and if beneficial optimization.

The Use of Soluble (Filterable) BOD

The field studies indicated that the performance of the RBC with respect to biological oxidation may be predicted with better confidence by relating the final effluent soluble or filterable BOD (or ammoniacal nitrogen) to the incoming mass load.

Work by Filion *et al* (1979) showed that the effluent filterable total organic carbon (TOC) and total Kjeldahl nitrogen (TKN) were predicted more precisely using the corresponding influent mass loading of TOC and TKN rather than influent concentrations, the correlation between flow rate and effluent concentration was very low.

A linear regression model of effluent BOD on influent loading was derived for a pilot plant treating domestic sewage in the USA (Newton *et al*, 1977) although it is not clear whether soluble or total effluent BOD was used and the regression analysis is not given.

Final discharge standards are for total BOD values and therefore, as previously noted, some means of predicting the total BOD from the soluble or filterable is required and will be a function of the efficacy of the final clarification stage.

Stage Sizing

The use of equal stage-sizing does not necessarily represent the optimal use of disc area. In some cases it can lead to overall process failure despite the total disc area being sufficient (Steiner 1979). If as has been indicated the effluent concentration is best predicted from influent mass load and is virtually independent of hydraulic loading then it is logical not to employ equal stage sizes.

It is in the treatment of strong industrial wastes that oxygen limitation most often occurs and Antonie (1978) comments that the first stage should be made sufficiently large to avoid the oxygen limiting condition. Where nitrification is required, this also will affect the stage sizing.

Mixing

The degree of mixing achieved by different RBC designs and configurations requires specification as it affects the aeration efficiency, the amount of short-circuiting, bioflocculation in the trough and the settlement of solids in the disc compartment. Inadequate mixing was cited by Zeevalkink *et al* (1979) as being the cause of deviations from the theoretical mass transfer coefficient in aeration tests.

Kinetic equations for process design and evaluation must take into account the mixing characteristics of the system.

POTENTIALS FOR OPERATIONAL CONTROL

If it is accepted that one of the principal design philosophies is the achievement of an optimal dissolved oxygen concentration in the trough,

then it is logical to look beyond the design to potentials for operational controls to achieve this objective.

Process variables which would appear to offer operational control potential with respect to dissolved oxygen concentrations are

- (i) speed of rotation
- (ii) depth of immersion
- (iii) supplementary aeration
- (iv) effluent recycling

Speed of Rotation is expensive to vary in mechanically-driven systems but it is a clear potential advantage of air driven systems. As would be expected, higher oxygen mass transfer coefficients have been measured for air-driven systems (Hynek and Chou 1979). It should be noted that increasing the speed of rotation increases the oxygenation capacity only at the expense of the oxygenation efficiency and that high speeds may lead to excessive scouring of the biological film. Biofloculation if significant, would also be affected by variations in the rotation speed.

Depth of Immersion could be adjusted simply by an automatic outlet weir. All RBC systems to date have had fixed immersion depths which have ranged from one third diameter to two thirds diameter. Most systems have adopted a 40-50 per cent areal immersion although one system recently examined by the author had a maximum oxygenation capacity at about 57 per cent immersion. Furthermore the optimum depth for oxygen transfer will vary with speed of rotation.

The relative proportion of the media in contact with the air and waste water should reflect the rates of oxygen and substrate diffusion, both of which will vary according to the operating conditions. Varying the depth of immersion might be found to be a worthwhile operational control.

The objectives of operational control would be to minimize the total media required and also to reduce energy consumption.

Supplementary Aeration offers significant advantages for operational control as it makes possible the maintenance of dissolved oxygen levels in the trough even when the oxygenation capacity of the RBC itself is inadequate. In addition there appears to be an improvement in treatment efficiency due to the air scouring resulting in a thinner and more active biological film (Hynek and Chou 1979).

Supplementary aeration may be achieved simply through air sparges at the bottom of the trough and of course is a permanent feature in air-driven systems.

Effluent Recycling offers a means of increasing the residual oxygen concentration in the trough by back-diluting with effluent having a relatively high dissolved oxygen content. In addition recirculation reduces the residence time in the trough and the substrate concentration. In plants employing integral drum filters, recirculation increases the frequency of backwashing and is useful in reducing high biomass concentrations in the trough which occur after shock loading. Drum filters backwash automatically in response to differential head loss which is dependent upon flow rate.

CONCLUDING REMARKS

- 1 Some of the design approaches currently employed have been discussed and the error in the use of linear regression equations relating organic removal to organic loading has been emphasized.
- 2 It has been argued that the use of soluble (filterable) organic concentration in the effluent for performance assessment has resulted from the need to specify separately the expected performance of the final clarification stage. Soluble effluent BOD is predictable from the influent mass loading rate for specific systems and wastes, but its use in design is limited.
- 3 The importance of design and control to achieve optimal dissolved oxygen concentrations in the trough has been stressed. It has been shown that the dissolved oxygen levels should be predictable from the incoming loading rate. Potentials for operational control of dissolved oxygen have been discussed.
- 4 Specifications for RBCs should include both oxygenation capacity and efficiency, plus mixing characteristics.
- 5 Final liquid/solid separation or clarification appears to have lacked emphasis in RBC design. For systems using drum filters the use of effluent recycling rapidly reduces any excess suspended biomass in the trough and in addition may contribute to the dissolved oxygen level in the trough.
- 6 The role of biofloculation in the RBC requires evaluation.

ACKNOWLEDGEMENTS

The author would like to thank Thames Water, The Milk Marketing Board and Clearwater Systems Ltd. for their help and cooperation and M. Santos and N. Thompson for their work on the field studies.

The views expressed in this paper are the author's alone.

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PAPER 7

INTRODUCTION TO MICROBIOLOGY

by

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The strength of the cell-wall enables the bacterium to maintain its particular shape and to resist the internal osmotic pressure which would otherwise cause the inner membrane to burst and the cytoplasm to be scattered. The wall is about 0.2 μ m thick. It must possess the ability to pass both the nutrient compounds inwards and the waste material out, although the strict differentiation between various substances as well as the highly selective control of material diffusion appears to be the role of the cytoplasmic membrane.

This cytoplasmic membrane is one of the three major sections of the protoplast - the other two being the cytoplasm and the nucleus - which exists as a thin, fragile sack just within the wall. This membrane controls the ingress and the egress of all nutrients and of all waste products, and must therefore be extremely selective in its operation. It is this membrane which contains the protoplasm and in which the bacterium lives. If the cell-wall is dissolved by enzymic action and the unprotected cell placed in an osmotically suitable environment the bacterium can still function. Inside the membrane is the complete cytoplasm, apparently consisting of a viscous liquid, which contains all the complexity of necessary organic materials essential to the metabolic processes of the organism. Reserve food granules are stored-up during periods of plenty, and are used up as external food becomes scarce and the cell is forced to rely upon endogenous respiration.

The nucleus is the source of all the material necessary for the life and reproduction of the cell and it appears not to possess any definite membrane. The nucleus contains one single molecule of double stranded DNA which, if straightened out, might be 1,000 times as long as the cell itself - this is a chromosome. All the enzymes in the cell originate in the nucleus although their area of activity appears to be just within the cytoplasmic membrane. The nucleus also contains the hereditary characteristics of the cell and the undamaged nucleus possesses the capability of creating new cellular components to repair cell damage and also of splitting itself in the initiating stage of cell reproduction by binary fission.

Some bacteria are capable of ensuring their continuing existence during prolonged periods of adverse conditions. This they accomplish by the production of a heavily protected, non-vegetative compaction of the cell's vital constituents. This is the spore which forms within the vegetative cell but occupies only about one-quarter of the available volume. It is often assumed that the spore formation is a response to adverse conditions but this is not necessarily accepted and it might be that this is just one phase in the cycle of bacterial life. Within the spore are all the necessary ingredients for the re-establishing of a fully vegetative cell. The spore contains all the necessary enzyme systems together with complete growth, respiratory and genetic equipment. All this is enclosed in a thick, strong wall. Inside the spore the contents are safe against intensities of heat, radiation, chemical disinfectant and dessication, which would rapidly kill its non-sporeforming relatives. And as the spore is non-vegetative it is unaffected by lack of nutrition. The resistance of the spore is due both to the toughness of the walls and to the state of partial dessication of the contents. Under favourable conditions and with environmental stimulation, which is not yet fully understood, the spore will burst out of its case to re-form a vegetative cell.

Spores are commonly formed by the aerobic *Bacillus* and by the anaerobic clostridia. *Bacillus anthrax* and *Clostridium tetani* are the causative organisms of well-feared diseases of man and cattle. *Cl. novyi* and *Cl. histolyticum* are the causative organisms of gasgangrene which was responsible for so many post-operative deaths before the days of aseptic surgery. *Cl. botulinum* (Lt. botulus - a sausage), which is an organism of food-poisoning, produces the most powerful exotoxin known and its spores are capable of enduring for more than six hours in boiling water. The only certain way of destroying spores is by complete sterilization in an atmosphere of steam in an autoclave at 122°C for a period of twenty minutes.

Pleuro Pneumonia-Like Organisms (PPLO)

The name of this group of microorganisms is derived from bovine pleuropneumonia. Initially these organisms were thought to be viruses but it was then appreciated that they could be cultured on artificial media to produce typical colonies and the PPLO were accepted as a distinct group of microorganisms. They are generally parasitic organisms although similar organisms have been discovered as saprophytes in sewage and in the soil.

The principal characteristic of the PPLO is that they do not have a rigid cell-wall not do they possess substances capable of forming a cell-wall. They are in many features similar to bacteria but also pass through an extremely small stage like the viruses. These small forms agglomerate in large masses which may be up to 10µm across and which readily disintegrate to form small, granular rods or spheres. The smallest elements may have a diameter as small as 100nm although the largest diameter will be of the order of several microns.

There is no doubt of the similarity between the L-forms and pleuropneumonia-like organisms but there are also essential differences. L-forms can be cultured on relatively simple media while the PPLO require a more complex growth material which usually includes blood serum, and which might also include growth factors from yeast extract and cholesterol. Unlike the PPLO L-forms are rarely found in diseased tissue. One the whole, if the L-form bacteria are not known to have developed from a normal, walled species they are classified as PPLO. PPLO are now sometimes classified as mycoplasmatales.

Rickettsiae

Rickettsiae are small, obligate parasitic organisms biologically resembling viruses but morphologically similar to bacteria. They are primarily parasitic to insects from which they are transferred to man and other vertebrates to create a number of febrile diseases. They occur as symbiotes (symbiotic - a mutually advantageous association of two organisms) of arthropods and are normally passed on to man through the bite of a tick, or a louse, or a flea, although infection might follow the rubbing of arthropod faecal matter into the skin.

Rickettsiae were discovered by Howard Taylor Ricketts in 1909 as the causative organisms of Rocky Mountain Spotted Fever. Later Ricketts also discovered the Rickettsiae prowazekii which causes epidemic, or classical typhus; from which disease he died as a result of his research. Rocky Mountain Spotted Fever is transmitted by ticks, endemic (or murine) typhus by fleas and scrub typhus by mites. Classical (or epidemic) typhus is louse-borne with an incubation period of between eight and fourteen days. Typhus invariably spreads as a result of natural or created catastrophe - particularly war - and develops most rapidly in winter as the result of overcrowding and the consequent ease of transfer of the carrier arthropod.

Rickettsiae are either rod-shaped or coccoid organisms. R. prowazekii is a short, bacillus-like rod organism with a diameter of about 0.3µm and a length of up to 2.0µm. Rickettsiae are usually just visible under an optical microscope and following fission often cling together in pairs or chains. They do not form spores. Most Rickettsiae would be retained by a bacterial filter but not the genus *Coxiella*. Rickettsiae, like viruses, cannot be grown on artificial media and are usually cultivated in the living chick embryos of hen's eggs. Being obligate parasites they cannot multiply outside a living cell. Unlike most viruses Rickettsiae are susceptible to anti-biotics.

Viruses

Viruses are the smallest known organisms and they differ both in their structure and in the manner of their reproduction from other organisms. The diameter size of a virus will be between 0.025µm and 0.25µm, which places them beyond the resolution powers of

Holozoic microorganisms are able to ingest solid food into the body. Holophytic microorganisms absorb soluble food material through the cell wall.

Yeasts

Yeasts and molds are only approximately definable and the two terms are often used merely for convenience. Yeasts, molds, bacteria, rickettsiae and PPLO all belong to the same major vegetable kingdom MYCOHYTA or FUNGI. Yeasts and molds possess

- | | |
|--|---------------------------|
| (i) thick rigid cell walls | (ii) holophytic nutrition |
| (iii) ability to grow on artificial cultures | (iv) enzyme activity |

There are subtle differences between molds and yeasts and it is normally difficult to differentiate sharply. Yeasts are unicellular plants possessing ovoid or elliptical cells and each cell exists as a separate complete individual. The cells are generally larger than bacterial cells (approx. $5\mu \times 8\mu$). Reproduction can be either sexual or asexual.

Yeasts form a special type of heat and drought resistant cell called Chlamydozooids. When these are formed the active growth of the cell ceases, food is stored, the cell acquires a thick protective wall, dehydration takes place and the spore can survive conditions that are unfavourable to the vegetative cell.

Molds

Molds form long, branched, hair-like filaments which are called hyphae and which often develop to form macroscopic masses called mycelium. Molds can grow in environments in which yeasts and bacteria cannot, e.g. high osmotic pressure areas such as jams and preserves. As well as the mycelium they possess sub-surface developments which are root-like and serve to fasten the plant in position as well as absorbing moisture and nutrients. All molds are strict aerobes. The cell walls are usually of chitin. The filament diameters can vary from 5μ to 50μ and the length may be enormous.

Molds may reproduce sexually or asexually. Septate molds - some mature molds exist as long chains of elongated cells arranged end to end and separated by septa. Non-Septate molds - with these there are no apparent cross-walls and the mycelium consists of a continuous, branching, protoplasm-filled tube.

Pathogenic forms of molds include ring worm, athlete's foot and dermatitis.

Bacteria

The entire group of bacteria known collectively as the class Schizomycetes (or fission-fungi) may be subdivided into the following major orders.

Rhodobacterales
Pseudomonadales
Beggiatales
Hyphomicrobiales
Actinomycetales
Spirochaetales
Eubacterales
Chlamydobacterales
Caryophanales
Myxobacterales

of which the true bacteria - or Eubacterales - are subdivided into families

Bacillaceae
Lactobacillaceae

Propionibacteriaceae
Corynebacteriaceae
Streptococcaceae
Micrococcaceae
Neisseriaceae
Enterobacteriaceae
Achromobacteriaceae
Pseudomonaceae
Spirillaceae
Brucellaceae

Further subdivisions is into genus, species, strains and clones,

An example of the classification is that of Escherichia coli (E. coli) which is common in animal intestines and is widely used as an indicator of faecal pollution. The group is the coliform group - members of which occur in the intestines of animals and man - and the genus is Escherichia. The species is coli which is characterised by its inability to utilize citrate as a sole source of carbon. There are then various strains.

Eubacteria

Eubacteria (true bacteria) which are non-filamentous, non-photosynthetic and possess rigid walls, include the smallest living organisms. The cell diameter can be as small as 0.5μ

The rate of metabolism of bacteria is proportional to the surface area to volume ratio which is far greater than with other organisms; hence bacteria are immensely physiologically active.

Shapes of bacteria

- (i) Sphere (coccus) with a diameter of 0.2μ to 4μ . These may be grouped in pairs (diplococcus), as a chain (streptococcus), divided in to planes, i.e. plates (staphylococcus), divided in three planes to give characteristic pockets of eight (sarcina) or exist as single spheres (micrococcus)
- (ii) Curved rods (vibrio) or as a screw or spiral (spirillum)
- (iii) Straight rods (bacterium) - some of these are able to form heat resistant, non-vegetative spores.

All food is absorbed through the cell-wall and all waste products diffused out through it. If the area of the wall in relation to the cell volume becomes too limited with growth, then division takes place to create a fresh cell.

The bacterial cell possesses four distinct sections (figure 1.5) which, working from the outside inwards, are the flagella

the capsule (or slime layer)
the cell wall
the protoplast

The flagella, which are each attached to a basal granule inside the cytoplasmic membrane, permit the bacteria to be motile and to move in a random search for equitable environmental conditions. Removal of the flagella does not effect the viability of the cell.

The capsule is the layer at which the bacterium contacts its environment. Its thickness varies with species and condition from about 10μ down to a fraction of a micron. Enlarged capsules are apparent with pathogenic bacteria and seem to be associated with the virulence of the microorganism. In some manner it appears that the mucoid gum allows the bacteria to evade the antagonistic actions of the phagocytes (white corpuscles) in the blood

introduction to microbiology

Microorganisms are both the enemy and the servant of the Water Engineer and the water pollution control specialist. The classic water-borne diseases of cholera (*Vibrio cholerae*), enteric fever (*Salmonella* Sp.) and bacterial dysentery (*Shigella sonnei*) are not the least in any list of the plagues of mankind, and two of the most widespread diseases which currently infect hundreds of millions of people throughout the world - Malaria and Schistosomiasis - rely directly or indirectly on water for their dispersion.

Parasites and pathogenic organisms are the enemy, but a wide range of heterotrophic saprophytes can be employed for the stabilization of waste putrescible organic matter, which may be either water-borne or as solid refuse. Microorganisms may be aerobic - require dissolved oxygen in order to metabolise or anaerobic - metabolise in the absence of dissolved oxygen or facultative - possess ability (faculty) to grow either aerobically or anaerobically, and they may be

heterotrophic - obtain energy and cellular material from a variety of organic compounds

or autotrophic - obtain energy through the oxidation of specific inorganic molecules

The microorganisms may ingest their food

holozoically - able to ingest solid food into the body

or holophytically - the food is absorbed in solution through the cell-walls.

They may exist as

saprophytes - live off lifeless organic matter

or pathogens - live on, and create disease in, living cells

and may obtain their energy either

chemosynthetically - from the oxidation of organic compounds

or photosynthetically - directly from sunlight.

But for simplicity and practicability all microorganisms may be divided into eight major groups

algae

protozoa

yeasts

molds

bacteria

pleuro pneumonia - like organisms (PPLO)

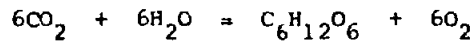
rickettsiae

viruses

Algae

These simple organisms belong to the large group Thallophytes - the thallus plants - in that they do not have definite roots, nor vascular stems for blood or sap, and do not produce flowers or seeds. They are all photosynthetic in that they contain chlorophyll and are capable of accepting energy from sunlight. In sunlight, as they respond photosynthetically, the algae utilize the $\text{CO}_2/\text{HCO}_3^-/\text{CO}^{+}_3$ complex in

water as their source of cellular carbon. They also require an appreciable level of both nitrates and phosphates as the source of cellular nitrogen and phosphorus. During photosynthesis relatively large quantities of oxygen are produced.



In the absence of sunlight, or in situations of reduced light intensity, many algae undergo endogenous respiration while others utilize organic substrates as their source of energy in the manner of bacteria. For both these forms of respiration dissolved oxygen is required and is absorbed from the carriage water. Many blue/green algae possess the capability of 'fixing' atmospheric nitrogen. Reproduction for the algae can either be sexual - fusion of gametes to form zygotes - or asexual. Successive generations frequently reproduce alternatively sexually and then asexually.

Algae exist either as individual cells (phytoplankton in water), or as clumps (Volvox, Gonium, Pandorina), or as extended filamentous strands. A common branched form of filamentous algae in fresh-water is the blanket-weed, Cladophora, which attaches itself to the substratum by modified basal cells. These filamentous algae are able to divide in one plane i.e. longitudinally and transversely. Sea-weeds are marine examples of filamentous algae; some of which are used for food, some as fertilizer and others (Gelidium, Gigartina) are harvested to produce that basic necessity of microbiologists - agar.

All algae possess chlorophyll which in many species is associated with one or more of a variety of other pigments such as phycocyanin, the carotenes, the xanthophylls, fucoxanthin and phycoerythrin. Algae also store intracellular food in the form of sugars, as the polyhydric alcohol mannitol, as starch, and as starch-like materials such as glycogen, and other characteristic polysaccharides. The cell-walls of algae consist of cellulose or of a cellulose-like material which might contain pectin or silica as well as the cellulose. Some unicellular algae are motile by reason of one or two flagella (e.g. Chlamydomonas, single flagellum, found in polluted waters), others are non-motile except for the spores, while others are completely non-motile (e.g. Chlorella, found in heavily polluted waters). Blue/green algae may be present in large numbers in surface waters. A 'bloom' in a reservoir may contain 1,000 to 2,000/ml, while in eutrophic conditions the numbers may increase up to 5,000 or 20,000 or even, on occasions, up to 400,000/ml.

Protozoa

These are single cell, microscopic animals which generally possess no rigid cell wall but instead have a soft and flexible, relatively fragile, outer cell membrane which is made of chitin (a glucose complex). Protozoa are the simplest forms of the animal kingdom but are many times the size of bacteria, i.e. a paramecium will be about 200 μ by 40 μ compared with an E. Coli which will be about 1 μ by 10 μ . Protozoa are sub-divided into 4 classes:

Sarcodina - move by means of pseudopods, e.g. Entamoeba histolytica - the causative organism of amoebic dysentery.

Mastigophora - move by means of long, whip-like lashes called flagella, e.g. Trypanosoma rhodesianse - the cause of African sleeping sickness.

Sporozoa - so called because they reproduce by spores. In the immature stages they move by means of pseudopods but the male gamete is flagellate, e.g. Plasmodium vivax which causes malaria from which 250 million people suffer each year and from which 2.5 million die.

Ciliata - move by means of hair-like processes called cilia, e.g. Paramecium caudatum which is non-pathogenic.

the optical microscope, and only the later invention of the electron microscope provided any idea of their shape. Viruses consist of a central coil of nucleic acid, which may be either RNA or DNA, surrounded by a protein coat, or capsid. In fact a virus is constructed of only a limited number of complex chemical units which give it a total molecular weight of a few million.

They are obligate parasites and reproduction can only take place in conjunction with the infected cell. The nucleic acid core of the virus enters the host cell and usurps the reproductive factors of the cell. All normal development of the infected cell ceases and its behaviour is specifically modified to create more viruses which are released by lysis of the infected cell. The cell organisms are in fact enslaved by the virus. The virus is, however, highly specific and is able to pass through many secondary hosts without reaction.

It appears that many viruses can exist for extended periods away from the host through being absorbed onto bacterial surfaces or colloidal debris in the environment. Moderate temperatures, however, of only 50°C to 55°C for a few minutes will effectively inactivate them, although they do possess the ability to withstand cold even better than do bacteria. Ultra-violet light is usually quite effective for virus inactivation, especially at the wavelength of 265 mμ. The peculiar characteristic of a virus is its ability to mutate readily. Although anti-biotics have only a strictly limited effect on a viral infection the body does not protect itself by the production of antibodies. These antibodies can provide extended immunity against a recurrence of a similar infection but the ability of the particular virus to mutate may mean that a fresh attack discovers no effective developed resistance.

The virus diseases in man include smallpox, infectious hepatitis, yellow fever, influenza, rabies and poliomyelitis.

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PAPER 8

CONCEPTS OF WASTE STABILIZATION
POND TREATMENT

by

J. Lumbers

CONCEPTS OF WASTE STABILISATION POND TREATMENT

Waste stabilisation ponds are relatively shallow man-made basins in which organic matter is oxidised or reduced, and pathogenic organisms destroyed by the action of naturally occurring biological degradation phenomena. Little operational control over these phenomena is possible, and the performance and character of a pond is dictated by such factors as:

- o depth
- o area
- o incoming pollution load
- o incident solar radiation
- o water temperature
- o toxic inhibition
- o wind induced mixing

Waste stabilisation ponds have found world-wide application and in some respects are more efficient than the higher rate, more intensive processes such as activated sludge and fixed-film biological reactors (filters and disc units). Certainly from the view-point of the removal of pathogenic organisms of most kinds, waste stabilisation ponds, of the correct design and sequence, are the most efficient process available.

Waste stabilisation ponds are particularly suited to sub-tropical and tropical applications as the elevated temperatures and high levels of incident radiation substantially reduce the land area requirements and the total cost of construction. It is coincidental that it is in these tropical and sub-tropical areas that surface waters are often used for drinking water supply with little or no treatment. A further consideration is that ponds are easily constructed with unskilled labour and do not require expensive mechanical equipment needing maintenance and spare parts. Simple construction is matched by ease of operation and maintenance, although like all treatment works regular and systematic attention is required. From the foregoing it is clear that waste stabilisation ponds should be considered as a first choice before alternative treatment processes in most countries of the world, where water-borne waste water or night-soil is to be treated.

The biological degradation processes which occur in waste stabilisation ponds generally include both aerobic and anaerobic zones either separately or within one pond. The process as a whole represents one step up the scale of intensification of waste water treatment which ranges from merely spreading or spraying untreated waste over suitable land areas to high rate activated sludge or fluidized bed processes requiring pure oxygen.

In the aerobic zones of waste stabilisation ponds the degradation of wastes is achieved by the symbiotic functions of algae and bacteria. The algal produce oxygen through photosynthesis and utilise the nutrients available in the raw waste water or produced as end-products of the bacterial breakdown organic matter occurring concurrently which in turn uses the oxygen produced by the algal. This relationship is illustrated in Figure 1

In the anaerobic zones or ponds the process of methane fermentation occurs as illustrated in Figure 2

The various reactions taking place in a facultative pond are shown in Figure 3

There are several types of stabilisation pond depending principally on the organic loading. These ponds are,

- | | |
|---------------------------|---|
| ANAEROBIC PONDS (AWSP). | anaerobic ponds are usually deep basins loaded sufficiently to ensure that the whole contents remain anaerobic and methane digestion occurs to some extent |
| FACULTATIVE PONDS (FWSP). | facultative ponds are those in which the liquid waste is aerobically and anaerobically and the majority of the oxygen required for aerobic oxidation being supplied by algal photosynthesis. At the bottom of the facultative pond is an aerobic sludge layer undergoing digestion or methane fermentation, |

Wind induced mixing is important to prevent thermal stratification limiting algal activity. Where insufficient wind mixing is expected artificial mixing by mechanical means is necessary.

MATURATION PONDS (MWSP).

maturation ponds have the principal function of achieving a high degree of pathogen removal. They are aerobic, shallow ponds supporting large numbers of rotifers and protozoa which assist in virus removal. In addition maturation ponds "polish" the effluent by providing further settlement and removal of nutrients and low concentration substrates likely to cause the growth of sewage fungus and watercourses.

HIGH-RATE AEROBIC PONDS: (H-RWSP)

high-rate aerobic ponds are very shallow ponds of not more than 300-400 mm and retention times of 1-3 days. The depth is such that full penetration of radiation is likely. The pond must be mixed several times a day and the algal material removed prior to discharge. Close control is required and they are not recommended for general application.

AERATED LAGOONS (ALWSP)

aerated lagoons are aerobic or facultative waste stabilisation ponds in which the oxygen required for oxidation is supplied wholly or in part by mechanical aeration facilities. Aerated lagoons may be "completely mixed" or "partially mixed". The completely mixed lagoons

(or nearing so) maintain all the solids in suspension and are considered to be fully aerobic with algal contribution to the oxygen budget relatively small. The incompletely mixed lagoons are facultative with an anaerobic sludge layer as in facultative ponds. The contribution of algal photosynthetic production to the oxygen budget may be significant.

Where pond systems incorporate some means of mechanical mixing requiring a power source consideration should be given to the provision of a recirculation facility which can assist in odour control of anaerobic ponds and relieving the load on the primary facultative pond by reducing the retention

The combination and sequence of different types of ponds is flexible and the possible "process trains" are as follows using the abbreviations indicated above

- FWSP →
- FWSP → MWSP → (MWSP)ⁿ
- AWSP → FWSP → MWSP → (MWSP)ⁿ
- ALWSP → MSWP → (MSWP)ⁿ
part mixed
- ALWSP → FWSP → MWSP → (MWSP)ⁿ
fully mixed
- AWSP → ALWSP → MWSP → (MWSP)ⁿ
part mixed
- AWSP → ALWSP → FWSP → MWSP → (MWSP)ⁿ
fully mixed

Note recirculation around some or all of ponds in series is possible

The advantages and disadvantages of waste stabilisation ponds may be summarised as follows:

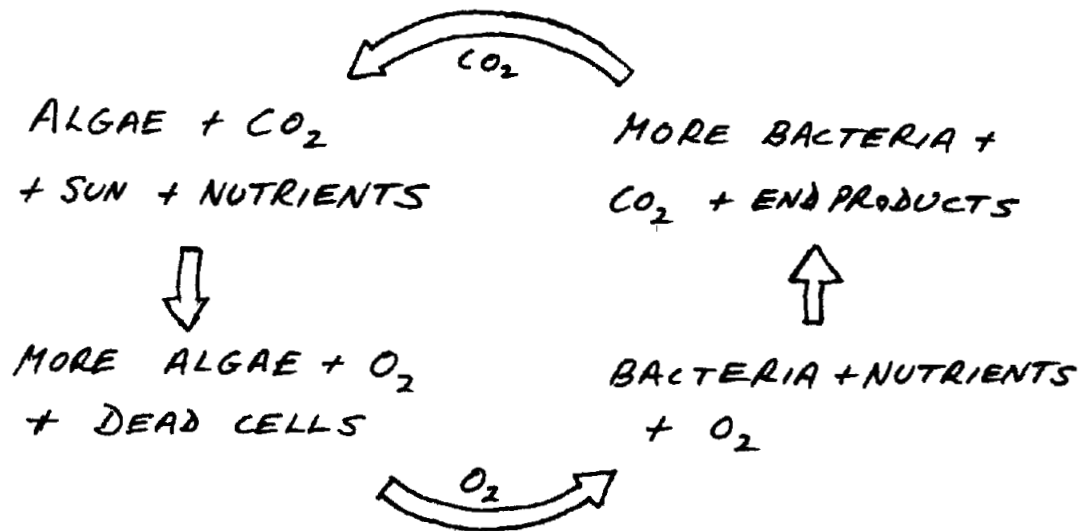
Advantages	Disadvantages
- o no primary sedimentation required - o no primary sludge to be treated - o easy maintenance - o good dilution of shock loads - o stabilised sludge produced - o no continuous sludge production - o flexibility in operation and extension - o no power requirements - o little mechanical equipment required - o excellent removal of pathogens - o land is easily reclaimed - o simple construction - o unskilled operators - o low hydraulic head loss	- o large land area required - o high algal content in effluent - o evaporative loss of water for reuse - o occasional odour nuisance - o algal toxic sensitivity - o

Figures

- Figure 1 - Symbiotic algae-bacteria relationship
- 2 - Anaerobic methane formation of complex waste
- 3 - Facultative waste stabilisation and mechanism

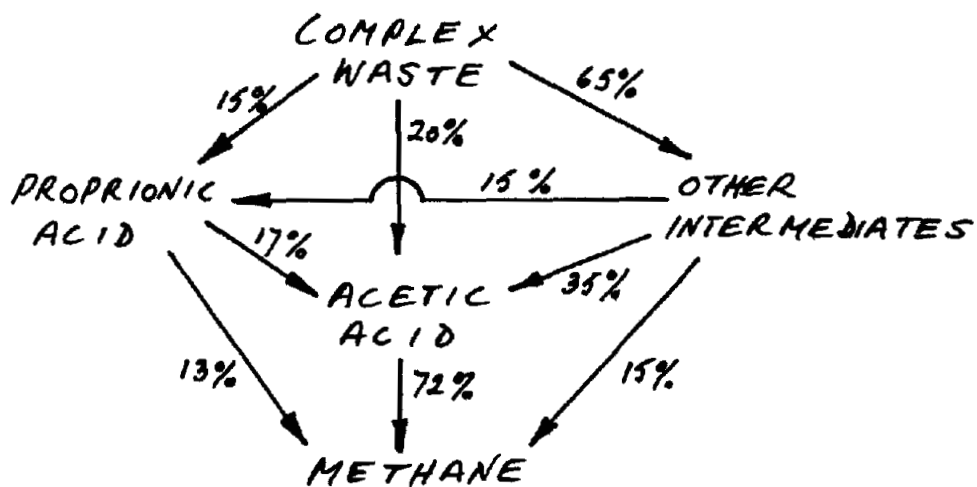
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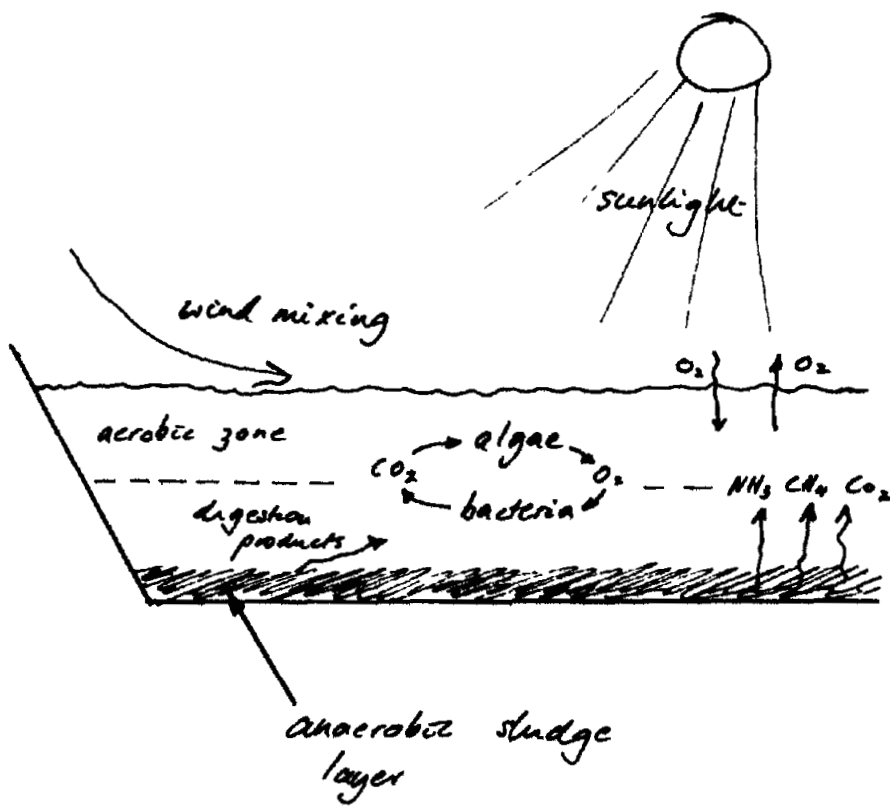
Symbiotic Relationship between Algae & Bacteria

FIGURE 1



Anaerobic Methane Fermentation of Complex Waste

FIGURE 2



Principal mechanisms in facultative ponds

FIGURE 3

PAPER 9

ANAEROBIC PONDS

by

J. Lumbers

ANAEROBIC WASTE STABILIZATION PONDS

1. Definition

Anaerobic ponds may be regarded as uncovered mesophilic digesters in which organic matter is converted to methane and carbon dioxide. The ponds are relatively deep and anaerobic conditions are ensured by applying a pollution load of a strength sufficient to exceed the potential oxygen production from algal photosynthesis. The anaerobic breakdown of a complex waste is shown in Figure 1.

2. Applications

Anaerobic ponds have been used to treat a wide range of waste waters including:

- slaughterhouse
- canning
- tannery
- textile dyeing
- meat packing
- fruit & vegetable processing
- dairy
- domestic
- refinery/chemical complex

They are particularly suited for the treatment of strong organic wastes which have a relatively low content of settleable solids. The removal of BOD in ponds of typical retentions is in the range of 25-80% and 50% is commonly achieved. Such removals greatly reduce the size of subsequent treatment facilities. An additional benefit of the anaerobic ponds is that they provide some role of primary settlement, settleable solids are removed and subsequently anaerobically digested within the ponds. The build-up of sludge in subsequent ponds is reduced as is the cost of desludging.

Typical BOD loadings are in the region of 0.2 kgs/m³d. Loading rates are often quoted per unit area although this has limited meaning. Typical

areal loading rates quoted are in the range of 1000-6000 kg BOD/ha d (McGARRY and PESCOD, 1970), although rates between 112 and 450 have been used in California, USA (OSWALD, 1968). Normal strength domestic wastes have been treated successfully in anaerobic ponds (PARKER et al, 1959 & 1968; BRISBAN et al, 1967) but this application does not make as optimal use of the process as does the treatment of strong organic industrial wastes.

The successful application of anaerobic ponds depends on a number of factors dictated by the relatively slow activity of the methane forming bacteria and their sensitivity to toxic shock loads. Such factors include:

- o temperature :

- over 15°C essential and in excess of 20°C preferable. The effect of temperature on retention time required is shown in Figure 2. Various forms of surface insulation materials have been used. The scum layer also provides insulation.

- o mixing:

- no mechanical mixing is usually used and gas bubbling to the surface combined with the inlet design provide some degree of mixing.

- o input pattern.

- anaerobic ponds operate more successfully if the input load is reasonably constant.

- o toxicity.

- anaerobic digestion processes (especially the methane formers) are particularly sensitive to toxic inhibitory substances in the raw waste water.

- o pH:

- below pH 5-4 methane bacteria become inactive and the optimal pH is 7-8

3 Design

3.1 Loading Rate

The principal design criteria is either a volumetric or areal loading rate i.e. kgs BOD/m³d or kgs BOD/ha d. The literature is divided on which to use although the tendency now is to use the volumetric loading, areal loadings not taking into account the depth of the pond. Typical loading rates have been quoted as in Table 1.

3.2 Design Formulae

Empirical modifications to the completely mixed first-order kinetics equations have been used, such as that proposed by VINCENT et al (1963).

$$L_e = \frac{L_i}{k_n \left(\frac{L_e}{L_i} \right)^N R + 1}$$

where L_e, L_i = effluent and influent BOD (mg/l)
 k_n = reaction rate coefficient (day⁻¹)
 N = constant
 R = retention period (days)

TABLE 1 : Loading Rates

	Volumetric Loading (kgs BOD/m ³ d)	BOD Removal (%)	Source
meating packing livestock wastes:	192-240	85	WHITE (1970)
o poultry manure	80-160	?	
o milk parlour	138	85	
o dairy manure	320	88	
	Areal Loading (kgs BOD/ha d)	BOD Removal (%)	Source
normal sewage	1008-1344 1120	60-70 ?	
fruit processing wastes	672	?	DORNBUSH (1970)
tomato processing wastes	448	75-80	
citrus fruit processing wastes	224	?	
domestic	560-1120 1000-6000	? ?	OSWALD (1968) McGarry & PESCOD (1970)

Values of k_n and N at 20°C were found to 6.0 and 4.8 respectively for a pond in Zambia. GLOYNA and TISCHLER (1979) use a similar equation for application to chemical refinery wastes:

$$L_e = \frac{32 L_1}{R} - 170$$

3.3 Retention

Retention is commonly regarded as the most important parameter of performance prediction but this is an over-simplification. Typical retention periods lie between 2 to 5 days (MALINA and RISO, 1976) and the importance of this factor is dependent upon the type of waste, the temperature, the variability etc.; retention period per se is not a fundamental design parameter. ARCEIVALA (1970) quoted a range of 2-7 days depending on temperature while McGARRY and PESCOD (1970) report that retention periods of longer than 5 days appear to have little effect on BOD removal for a given influent mass load. DORNBUSH (1970) reports various retentions used in Australia (2-5 days), USA (3-6 days), Canada (2-4) days

3.4 Depth

Anaerobic ponds are generally 3-5 m deep although shallower ponds have been used. Greater depths tend to conserve heat and ensure that the contents are fully anaerobic. Penetration to the ground water table is not desirable as this may lead to contamination of a water source and also cooling of the pond contents

3.5 Recirculation

There is some evidence that recirculation reduces odours from anaerobic ponds (OSWALD, 1968) and improves the performance (WHITE, 1970; DORNBUSH, 1970). However it is important to distinguish between sludge solids recirculation to increase the "active biomass" in the pond and effluent recirculation to reduce odours by the formation of an aerobic surface layer. Improved mixing may also be a beneficial result.

5.6 Configuration

Anaerobic ponds may be used in series or in parallel and some flexibility in operation is desirable. Breadth to length ratios of at least 3 to 1 have been suggested by COERVER (1964) in order to facilitate emptying by drag-line. OSWALD (1968) has suggested various inlet designs.

The effluent from anaerobic ponds is not suitable for discharge to the environment and requires further treatment

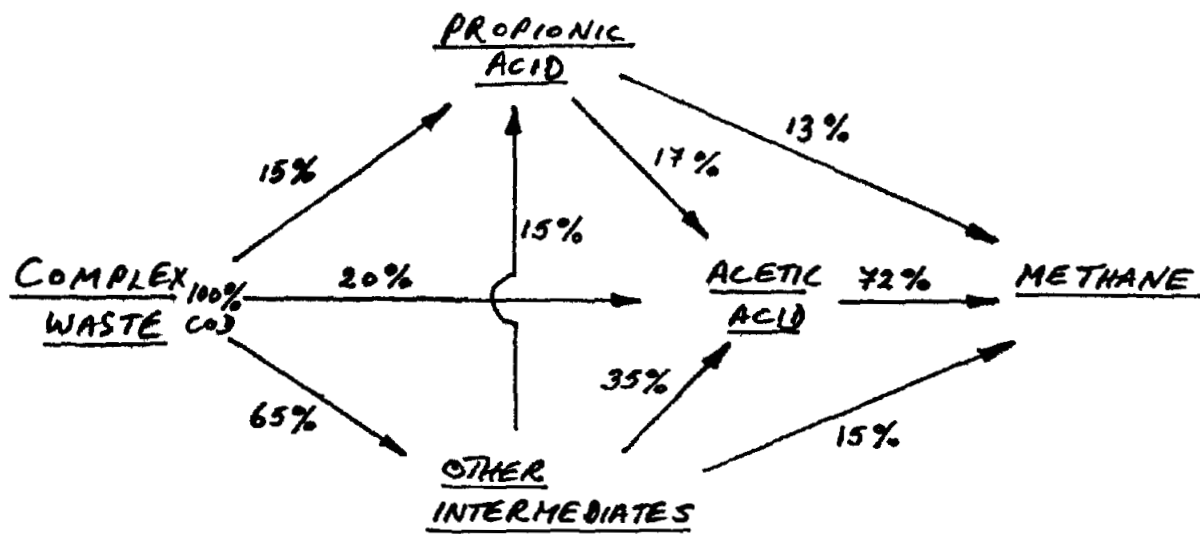
Figures & Tables

- Figure 1 - Breakdown of complex waste anaerobically
- Figure 2 - Effect of temperature on time for complete digestion
- Tables 1 - Anaerobic pond loading rates

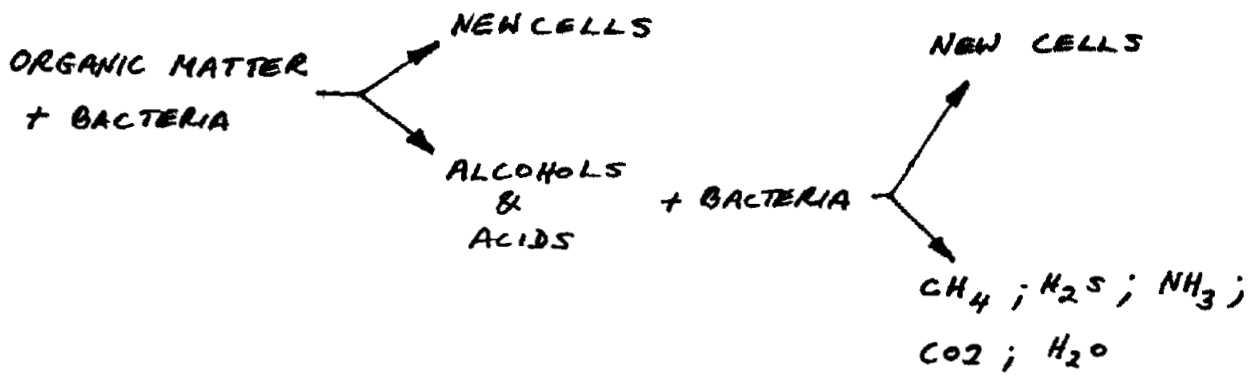
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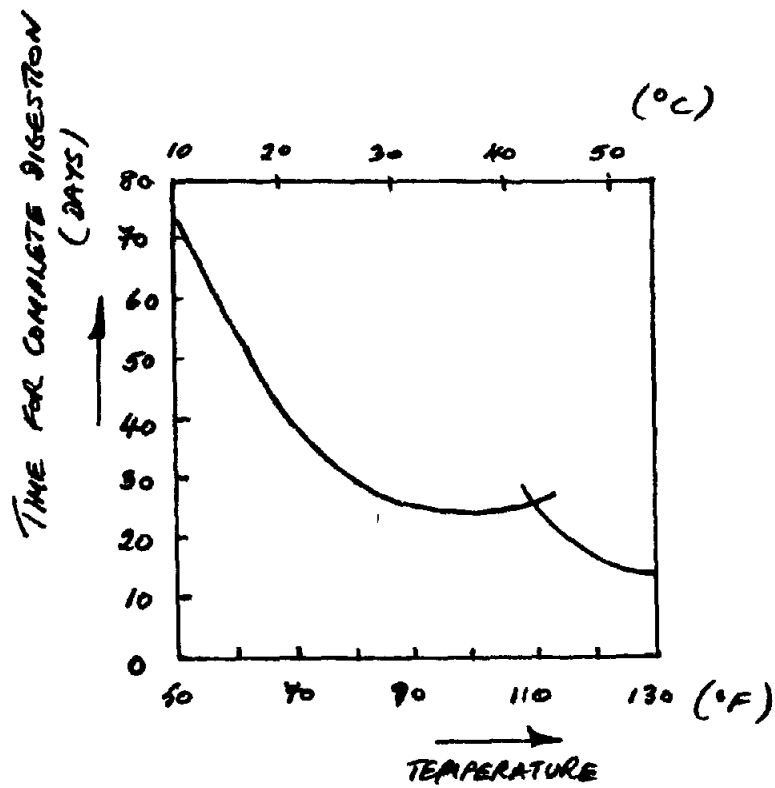


(after McCARTY 1964)



ANAEROBIC OXIDATION OF COMPLEX ORGANIC WASTE.

FIGURE 1



EFFECT OF TEMPERATURE ON TIME REQUIRED FOR DIGESTION.

(after : FAIR et al (1968))

FIGURE 2

PAPER 10

INTRODUCTION TO ACTIVATED SLUDGE

by

Mr Ken V. Ellis

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1. History

Activated-Sludge was developed by Messrs Arden and Locket at Manchester in 1914. They aerated a sample of settled sewage for a period of days before allowing it to settle in order to decant the top liquor. The container was then refilled with fresh settled sewage and the process continued until a highly active, flocculant sludge was produced. This sludge was found to be capable of purifying waste-water within a period of a few hours, when aerated in contact with it. This sludge was called activated-sludge.

2 What it is

Activated-sludge is a flocculant mass, mainly of saprophytic bacteria intimately mixed with the polysaccharide shells of dead microorganisms. Through or on this flocculant mass swim or crawl many varieties of protozoa. Fungi are common in activated-sludge, particularly in those sludges dealing with sewages containing a high proportion of industrial wastes. Algae, because of the limited continuing contact of the sludge with sunlight, are rare. Macro-organisms are far less common in activated-sludge than in percolating filters and are largely limited to rotifers and nematode worms.

An empirical formula for the sludge has been suggested as either $C_5H_7NO_2$ or $C_{11}H_{18}N_{17}O_{17}P$.

The processes of activated sludge treatment are:

- (i) Adsorption
- (ii) Carbonaceous oxidation
- (iii) Nitrification

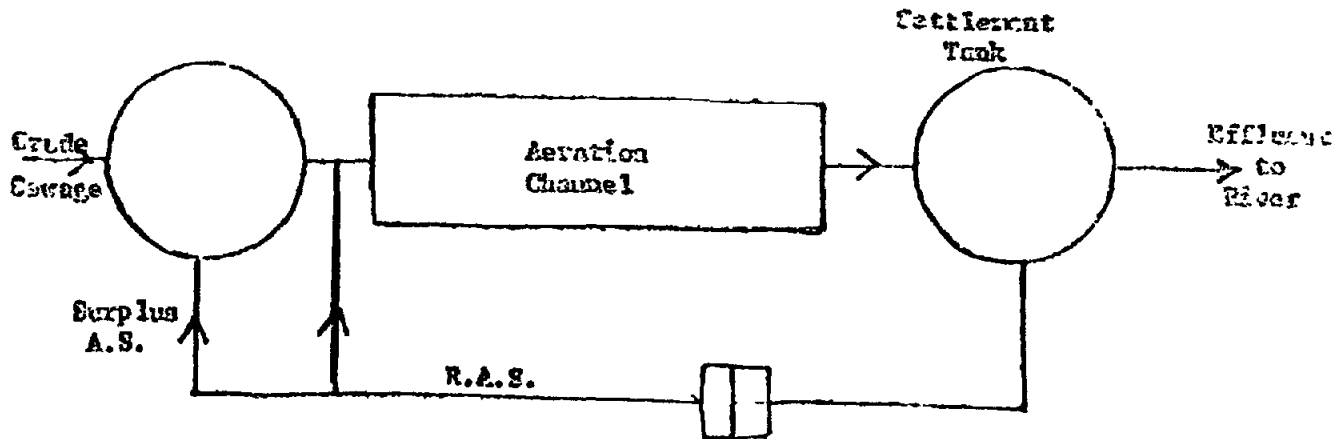
3. The Activated-Sludge Process

There are five fundamental components of the activated-sludge process.

- (1) A reactor in which waste-water to be treated is in contact with the activated sludge
- (2) A means of transferring atmospheric oxygen to the reactor contents
- (3) A means of agitation to ensure sufficient contact between the sludge, the polluted liquid and the dissolved oxygen
- (4) A system by which the activated-sludge may be separated from the purified liquid to better than 99%.
- (5) A system for re-cycling the activated-sludge from the secondary settlement tanks to the reactor.

Components 2 and 3 are invariably the same

FIGURE 1



In the aeration channels the build up of daughter-cells, i.e. the production of activated-sludge, continues until the process is inhibited by

- (i) Exhaustion of the substrate
- (ii) Build-up of toxic end-products
- (iii) Shortage of oxygen

4. Aeration Systems

Atmospheric oxygen is supplied to the mixture of waste-water and activated-sludge (mixed-liquor) either by means of a surface-aerator or through diffused air

a. Surface-aeration

Surface aeration is brought about by a mechanical agitator spinning partially in and partially out of the mixed liquor. In this manner the liquor is thrown violently across the surface of the tank, greatly extending the surface area of contact between it and the air, and thus facilitating the adsorption of oxygen. This aerator may be

- (i) Simplex-Type - in which a system of radial vanes (the cone) spins about a vertical axis at the surface of the mixed liquor and pulls the liquor up through a vertical updraught tube prior to dispersing it over the tank surface

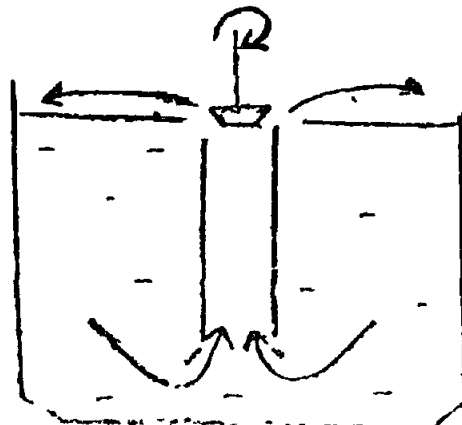


FIGURE 2

The mechanical aerators are usually matched to individual drive units and the cones revolve at between 30 and 60 r.p.m.

The oxygenation capacity is varied by altering the depth to which the cone is immersed in the mixed-liquor. This is achieved either by mounting the aerator on a retractable drive shaft or by employing an adjustable outlet weir from the tank. The oxygenation efficiency is maintained over a wide range of immersion which extends from the top of the cone to 200 mm below the top.

The equipment is robust and reliable and the overall gear unit transfer efficiency is at about the practical limit of about 96%. The gears have a design life of about 100,000 hours.

This aeration system may be employed for complete-mix units or for plug-flow situations which may use either tapered aeration or the incremental feed techniques.

- (ii) Kessener-Brush (TNO) aerator spins about on a horizontal axis. The aerator being a cylindrical steel brush operating at about 70 r.p.m.

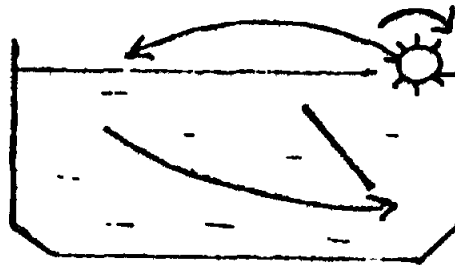


FIGURE 3

The oxygenation capacity may also be varied by altering the extent of the brush immersion in the mixed-liquor. This aeration system is widely employed in oxidation ditches.

(iii) The Sheffield Bio-Aeration System

The Sheffield Bio-aeration system of paddle-aeration in long shallow channels should also be added to this section for completeness although it is now of little more than historical interest. This system was an attempt to simulate the self-purification processes of rivers in which it was hoped that the majority of the required oxygen would be absorbed at the extended air/water interface of the long, narrow channels. This expectation was never achieved and the paddles had to be modified to create a definite surface aeration effect.

b Diffused-Air Processes

- (1) These rely on the diffusion of low-pressure compressed air in the form of fine bubbles into the bottom of the reaction tank through ceramic air-diffusers. The average pore-size of the diffusers is 150mm which will create air-bubbles with a diameter of 2.0 to 2.5 mm. The diffusers are normally constructed of silica and aluminium oxide held together by a ceramic binder. There is presently a move to introduce non-ceramic polythene diffusers.

In order to prevent over-early blocking of the fine pores of the diffusers it is essential that the air supplied should be largely free from dust - preferably as little as $3.6 \text{ mg}/10^3 \text{ m}^3$ air. The compressed air is supplied either by rotary positive displacement blowers for small supplies (up to about $17,000 \text{ m}^3/\text{hours}$) or by centrifugal blowers for larger supplies. The pressure to be overcome is the sum of the static head of the water and the back pressure created by the resistance of the diffusion. The back pressure from a new dome may be as little as $35 \text{ kg}/\text{m}^2$, rising to perhaps $140 \text{ kg}/\text{m}^2$ for an older dome.

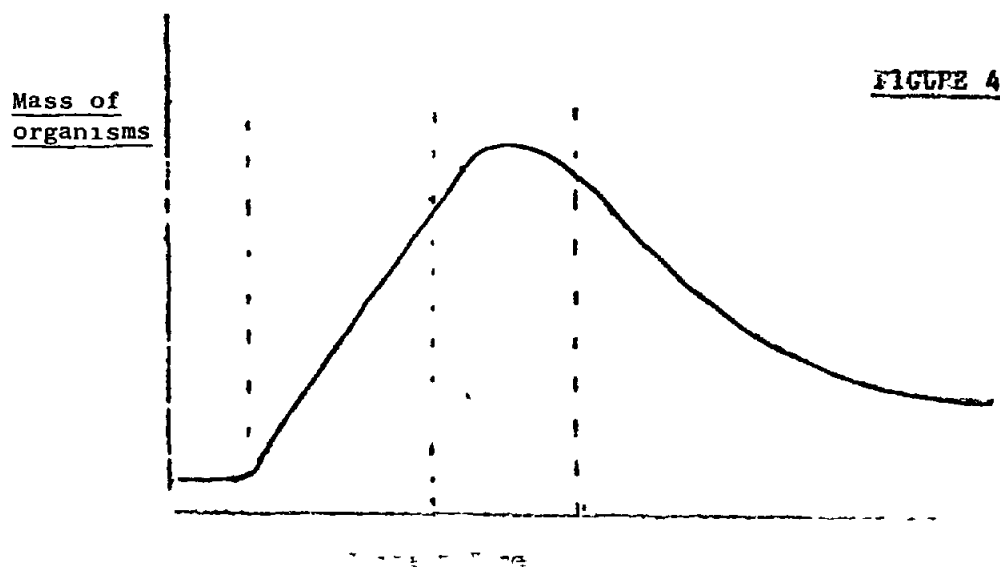
The throughput of air through each 176 mm diameter diffuser-dome will vary from about 14 litres/minute up to 58 litres/minute, but a normal flow is about 28 litres/minute. Between 37 m^3 and 74 m^3 of air will be required for every kg of BOD removed.

Porous polyethylene diffusers are now being presented on the market. The emphasis with these is on the ease of withdrawal for cleaning and other maintenance. Often the assemblies are fitted with swivel joints to allow the complete rig to be winched clear of the water. These diffusers are washed in detergent solution or in dilute hydrochloric acid

(11) Inka Type

The Inka process makes use of coarse bubble spargers mounted about 600 mm below the water surface. The coarse bubbles are in contact with the mixed-liquor for only a very limited period but the lack of efficiency of oxygen transfer is largely countered by the use of simpler, more robust air blowers delivering air at a lower pressure than could be supplied to a conventional diffused-air process. The orifices are 25 mm diameter and the gratings so arranged as to develop a spiral-flow effect in the channels.

5. Variation of Sludge Mass



It is suggested that when waste-water and activated-sludge are first mixed together there is initially a lag period during which the microorganisms acclimatise themselves to the new environment. Following this the development of the mass - particularly adsorption - progresses at a great rate (the log-growth phase) until the availability of substrate becomes limiting (declining and stationary phases). With a prolonged contact period the microorganisms are forced to use up reserve food sources within their own cells and also to metabolise the lysed cell contents of already dead cells. This is the endogenous phase and the longer this continues the smaller will become the mass of additional sludge created; although the mass will never return to that of the sludge before the addition of the waste-water.

The factors effecting production of activated-sludge have been stated are

- (1) Adsorption or entrainment of biodegradable insoluble matter and adsorption of biodegradable soluble matter.
- (2) Adsorption or entrainment of inert suspended matter and adsorption of inert soluble matter
- (3) Conversion of soluble and adsorbed insoluble degradable matter into bacterial cells
- (4) Ingestion of bacteria and possibly of other suspended matter by protozoa or other predators.
- (5) Degradation of microbial cells by endogenous respiration.

It has also been suggested (McKinney) that during the log-growth phase

1.0kg B.O.D. + 0.5kg O₂ + cells (activated-sludge)

→ 0.7kg new cells + CO₂ + H₂O

Then during and after endogenous respiration

0.7kg new cells + 0.75kg O₂ → 0.17kg dead cells + CO₂ + H₂O + NH₃

Following this would then be a further oxygen demand as the ammonia is oxidised to nitrate.

6 Plug-Flow Systems

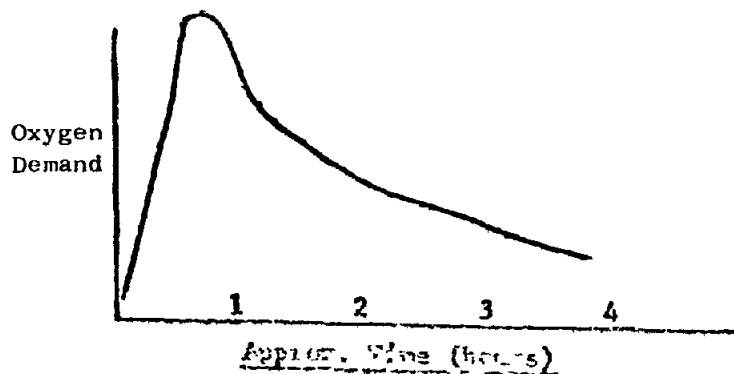


FIGURE 5

In a conventional activated-sludge unit the mixture of sludge and sewage is fed into one end of a long, narrow channel through which it flows with relatively little longitudinal mixing until the mixture of now purified effluent and activated-sludge is withdrawn at the far end. Thus a discrete volume of mixed-liquor flows as a 'plug' through the channel.

Plug-flow of this type leads to an uneven oxygen demand along the length of the aeration channel (figure 5). This may be initially about $300 \text{ mg l}^{-1} \text{ hr}^{-1}$, dropping to perhaps $80 \text{ mg l}^{-1} \text{ hr}^{-1}$ after 30 minutes to an hour and then decreasing slowly to 30 to $40 \text{ mg l}^{-1} \text{ hr}^{-1}$ after several hours

This uneven demand for oxygen is countered by either 'tapering' the air supply or by providing an 'incremental feed' of sewage to the unit

Complete-mix, or homogenous, units supply the mixture of activated-sludge and sewage along the whole width of a relatively broad and short tank, the effluent and sludge being withdrawn similarly along the whole tank breadth at the far end. In this manner the mixed-liquor tends to be completely homogenous with no concentration profiles through the unit.

7. Parameters of Activated-Sludge Operation

- a. Volumetric Loading - This is the mass of BOD applied per day to each unit volume of aeration tank capacity. The units are $\text{Kg/m}^3 \text{ d}$ and the loading will vary from about 0.4 up to 1.0 or even, on occasion, up to 2.0.
- b. Mixed Liquor Suspended Solids Concentration M L S S.

This is the mass of activated-sludge solids per unit volume of the aeration channel and is stated in terms of Kg/m^3 . Early plants were operated with low concentrations of perhaps 1.5 or 2.0 Kg/m^3 but the modern trend is towards greater concentrations which may reach 4.0, 5.0 or over 8.0 Kg/m^3 . Higher concentrations are limited by

- a. Air supply
- b. Settleability of sludge in final tanks
- c. Return sludge pumping capacity
- d. Size of final tanks

c. Sludge Return Ratio

The M.L.S.S. level in the aeration channel is maintained by returning activated-sludge from the final settling tanks. The rate of return depends upon

- a. the concentration of M.L.S.S. required
- b. the condition (settleability) of the sludge

S.R.R. is usually expressed as a percentage of the d.w.f. to the plant and is frequently about 100

d. Sludge Volume Index (Mohlmann)

The S.V.I. is a measure of the settleability of the sludge. It is the volume in ccs occupied by 1g of dry sludge solids after settling for 30 minutes.

$$\text{S V I} = \frac{\text{Settled Volume of Sludge (\% after 30 minutes)}}{\text{Suspended Solids (\%)}}$$

An S V I of 20 - 50 is good while over 150 indicates a 'bulking sludge'.

The reciprocal of the S.V.I, the Donaldson Sludge Density Index, is sometimes employed

$$S.D.I. = \frac{100}{S V I.}$$

The disadvantages of the S.V.I. are that the values vary inconsistently with the concentrations of the sludge and that they depend upon the dimensions of the container. These disadvantages have been overcome by the development of the Stirred Specific Volume (ml/g) which is largely independent of the initial solids concentration. For this latter test the sludge is slowly stirred (1 r.p.m.) as it settles and the result is always reported for a sludge concentration of 3.5 g/l.

e. Sludge Loading Factor

The S.L.F. is the Kg B.O.D. applied for each Kg of activated-sludge in the process each day (Kg. B.O.D /Kg activated-sludge day) It is frequently accepted that the following ratios are associated with the stated effluent condition

S L.R.

0.05 to 0.1	extended aeration
0.3	nitrification of effluents
0.4	90% to 95% B O D removal
0.5	90% B.O D removal

The S.L.R is closely related to the Removal Velocity (q) which is employed frequently in the theoretical considerations of activated-sludge operations Kg B.O.D. removed/Kg cells day

f Sludge Growth Index

This is the mass of sludge produced per unit mass of B O D applied each day

The S.G.I. 1 decreases with retention time
 2. decreases with increasing concentration of sludge under aeration
 3 increases with sewage strength
 4. varies with the dissolved-oxygen concentration in the mixed-liquor It passes through a maximum at 0.5 mg/l d.o and then declines to 1.0 to 1.5 mg/l, after which it remains constant. There are two proposed empirical formulae for calculating the S G I

$$S.G.I. = 0.2 + \frac{0.25}{tr^{\frac{1}{2}}}$$

$$S G.I. = 0.2 + \left\{ \frac{Kg B O D / 24 \text{ hours}}{Kg M L S S} \right\}^{\frac{1}{2}}$$

where tr - retention period in the aeration channels. Again, the S G I is closely related to the Yield Coefficient (Y) which is Kg cells produced/Kg of B O D removed day. The product of the Removal Velocity (q) and the Yield Coefficient (Y) gives the sludge Growth Rate (u)

$$q Y = u \quad (\text{kg cells produced/kg cells in process day})$$

g Sludge Age

The sludge Age is a measure of the residence time (in days) of the activated-sludge in the system. This age varies from about 2 days in high-rate plants to more than 30 days in extended-aeration plants

$$\text{Sludge Age} = \frac{\text{Mass of Activated-Sludge present}}{\text{Mass lost daily (Surplus sludge and in effluent)}}$$

8 Nitrification

Traditionally it has been accepted that nitrification in the activated-sludge process was impossible - or extremely difficult. This is not so. However, the factors that can lead to difficulty in nitrification are:

- (i) the presence of inhibiting substances in the sewage
- (ii) a temperature of less than 10°C
- (iii) a low M.L.S.S. concentration
- (iv) a young sludge age

The nitrifying bacteria (Nitrosomonas and Nitrobacter) reproduce at a far slower rate than the majority of the heterotrophic microorganisms in the sludge and for this reason an advanced sludge age is essential for the oxidation of the ammonia in the sewage. It can be shown that if the nitrifying organisms in the sludge returned to the aeration channels is equal to that lost in the effluent then the condition for a significant degree of nitrification is given by

$$\frac{C_{No} - C_{N1}}{C_{N1}} \geq \frac{\Delta S}{S}$$

where C_{No} - concentration of nitrifying bacteria at the outlet to the aeration channel

C_{N1} - concentration of nitrifying bacteria at the inlet to the process

S - M.L.S.S. concentration at inlet to aeration channel; ΔS - increase in concentration of M.L.S.S. achieved during purification process.

Hence, if nitrification is required in an activated-sludge plant it is essential to minimise the growth of the sludge mass by moving the process into the endogenous phase by

- (i) raising the concentration of the M.L.S.S.
- and (ii) extending the period of aeration

The concentration of nitrifying bacteria will never be high but will have a maximum of about 50 mg/l which represents about 2×10^8 cells per millilitre

Any increase in temperature from 7°C to 25°C greatly improves the rate of nitrification. The rate may double between 10°C and 20°C

9. Denitrification

There are two types of denitrification process

Assimilative reduction
Respiratory reduction

Assimilative reduction is the reduction of nitrate and nitrite to ammonia and its consequent involvement in anabolic cell metabolism. Respiratory reduction (true denitrification) is brought about by facultative bacteria, usually in the absence of dissolved-oxygen. With no d.o. available for the metabolic processes the bacteria turn to the chemically bound oxygen in the nitrate. This is reduced first to nitrite and then to gaseous nitrogen in which form it is usually evolved.

In the activated-sludge process denitrification is achieved using one of two techniques.

a. The separate form

In this technique the normal activated-sludge processes of carbonaceous oxidation and nitrification are followed by a secondary activated-sludge process, complete with its own sludge and sludge recirculation system. In the secondary mixed-liquor channels the normal processes of aeration are neglected so that the dissolved-oxygen content of the liquor is negligible. Suspension of the mixed-liquor solids is maintained either by minimal aeration or by mechanical paddles. With the d.o. of the mixed-liquor being negligible denitrification occurs and nitrogen is evolved as a gas. In order to stimulate the facultative denitrifying bacteria in the presence of an already purified effluent it is necessary to add an easily assimilable substrate such as methanol. By this technique as much as 95% to 98% of the nitrate present in the effluent from the primary activated-sludge unit can be removed. In some situations it is suggested that a three-stage activated sludge process be employed - carbonaceous oxidation, nitrification and denitrification.

b. Integrated System

With this system the early section of the aeration channel is maintained without aeration, merely employing mechanical paddles to suspend the mixed-liquor solids. Following this anoxic section the remainder of the channel capacity is operated to give full carbonaceous oxidation and full nitrification. The effluent is therefore well nitrified but as the ratio of activated-sludge to waste-water flow is maintained at about 1:1, 50% of the formed nitrate is carried back with the re-cycled sludge. As this sludge is mixed with fresh settled sewage in the anoxic section of the mixed-liquor channel the bacteria employ the combined oxygen of the nitrate in their desperate search for available oxygen. The result is a loss of about 50% of the nitrate produced and no additional substrate is required due to the presence of an excess of settled sewage.

PAPER 11

EXTENDED AERATION ACTIVATED SLUDGE SYSTEMS
(OXIDATION DITCHES)

by
J. Lumbers

EXTENDED AERATION AND OXIDATION DITCHES

1. Definition

Extended aeration is a low rate activated sludge system which operates in the endogenous respiration phase. It may be seen from Figure 1 that low rate systems (extended aeration) are characterised by a low F/M ratio of about 0.05, a sludge age of 20-30 days and surplus activated sludge production rates of 0.6 kgs DS/kgs BOD removed (N.B. where no primary settlement is provided total sludge production rates are in the order of 0.9-1.4 kgs DS/kgs BOD removed). The hydraulic retention period, based on the average incoming waste water flow, is usually between 1 and 2 days.

2 Applications

Oxidation ditches are the usual form of extended aeration systems and were developed to provide simple waste water treatment plus some aerobic sludge stabilisation. Since their development in about 1953 ditches have found wide application, particularly in Europe, especially Holland. In the UK they have been applied to treat a variety of municipal and industrial wastes.

The principal advantages of ditch systems include.

- o primary settlement is not essential (dependent upon economics of sludge treatment and disposal)
- o some aerobic stabilisation of sludge occurs
- o total sludge production is reduced (due to endogenous respiration)
- o relative simplicity of operation
- o some systems for small works do not require separate secondary settlement tanks
- o rapid dilution of incoming shock loads
- o nitrification and denitrification is possible in one tank.

The disadvantages include:

- o high energy consumption
- o reasonable level operator skill essential

Oxidation ditches are generally regarded as completely mixed systems, with over 100 turnovers in one residence time of about a day. However, short term analyses (< 15 mins) of the system will observe a plug-flow regime.

3 Configurations

A variety of configurations of extended aeration systems and oxidation ditches is possible. Figure 2 shows a concrete tank version with a short aeration channel. Other configurations are shown in Figure 3, and materials for ditch lining have included

- o gunite
- o precast concrete slabs or planks
- o fencing materials
- o stone
- o butyl sheeting
- o stabilised earth

The types of aerator used depends on the depth and aeration capacity required. Brush aeration devices have largely been superseded by the cage rotors or mammoth rotors shown in Figure 4. Cage rotors are used for 1-2 m depths while mammoth rotors can be used for up to 4 m depth. The oxygenation capacity of rotors is in the order 6 kg O₂/m³ hr in clean water under standard conditions. Typical immersions are around 200 mm and the speed of rotation about 70 rpm.

4 Design of Aeration Tank or Ditch

The principal design parameters for oxidation ditches and extended aeration activated sludge systems are as follows:

- | | |
|--|------------------------------|
| o F/M ratio, organic loading factor | |
| of sludge loading rate | 0.05-0.10 kg/kgd |
| o Mixed liquor suspended solids (MLSS) | 3.0-5.0 kg/m ³ |
| o Retention period based on untreated | |
| average inflow rate | 24-36 hrs |
| o Volumetric BOD loading | 0.2-0.4 kgs/m ³ d |
| o Sludge age or mean cell residence time | 20-30 days |
| o Surplus activated production rate | 0.6 kgs DS/kg BOD removed |

- o Oxygen requirements 2-3 kgs O_2 /kg BOD applied
- o Energy input for mixing 15-20 W/m³
- o Dissolved oxygen concentration in MLSS ≥ 2 mg/l

Where primary settlement is used prior to oxidation ditch treatment is reasonable to assume 40 percent of the BOD organic load is removed

The total oxygen requirement is a function of:

- o the mass of carbonaceous BOD removal per day
- o the mass of ammonia-nitrogen removed per day
- o the endogenous respiration of the MLSS (temperature dependent)
- o the degree of in-tank denitrification

JOHNSONE and CARMICHAEL (1980) quote the following equation for the oxygen requirements

$$kgO_2/d = a X_1 + bX_2 - cX_3 + 0.024 V M.R_{20}^{1.07^{(t-20)}}$$

where a = empirical constant $\cong 1$

b = 0.95 stoichiometric constant for nitrification = 4.34

c = 0.95 stoichiometric constant for denitrification = 2.71

X_1 = carbonaceous BOD removed per day (kg/d)

X_2 = ammoniacal nitrogen removed per day (kg/d)

X_3 = mass of nitrate denitrified per day (kg/d)

V = aeration tank volume (m³)

M = concentration of MLSS (kgs/m³)

R = endogenous respiration rate of the MLSS at 20°C = 3.9
(mg O_2 /g sludge hr)

T = temperature (°C)

In calculating the oxygenation capacity required under the actual site conditions it is necessary to adjust that calculated under standard conditions to those pertaining at the proposed location.

The oxygenation capacity (kg O_2 /m³h) must be adjusted for

- o the dissolved oxygen saturation concentration adjusted for temperature and pressure (altitude)
- o the required dissolved oxygen in the mixed liquor

- o the α factor - to account for the difference between the oxygen mass-transfer coefficient in clean water and waste water
 - o the β factor - to account for the difference between the dissolved oxygen saturation concentration in clean and waste water
- (NOTE see notes of aeration theory)

5 Secondary Settlement Tanks

The efficient performance of the secondary settlement tank (for those larger systems which require them) is absolutely essential to the process. Inadequate settlement will lead to an inability to achieve the design level mixed liquor suspended solids and a consequential reduction in treatment. Poor settlement tank performance also leads to suspended solids being carried-over with the effluent and this will almost certainly result in an exceedance of the required discharge standard.

The performance of the secondary settlement tank is a function of

- o the surface solids loading rate ($\text{kgs/m}^2 \text{ h}$)
- o the sludge settleability, SSVI (ml/g)
- o the underflow rate or rate of sludge withdrawal, typically 0.5-1.0 DWF ($\text{m}^3/\text{m}^2 \text{ h}$)

Traditional designs have relied upon empirical values of the hydraulic surface loading rate ($\text{m}^3/\text{m}^2\text{d}$) or upflow-rate typical figures being in the order 40-50 $\text{m}^3/\text{m}^2\text{d}$ at peak flow rate, including the returned activated sludge flow. Improved application of hindered settling theory have led to improved design approaches accounting for the principal influential factors listed above, DICK (1972), WHITE (1980 & 1975).

Figures

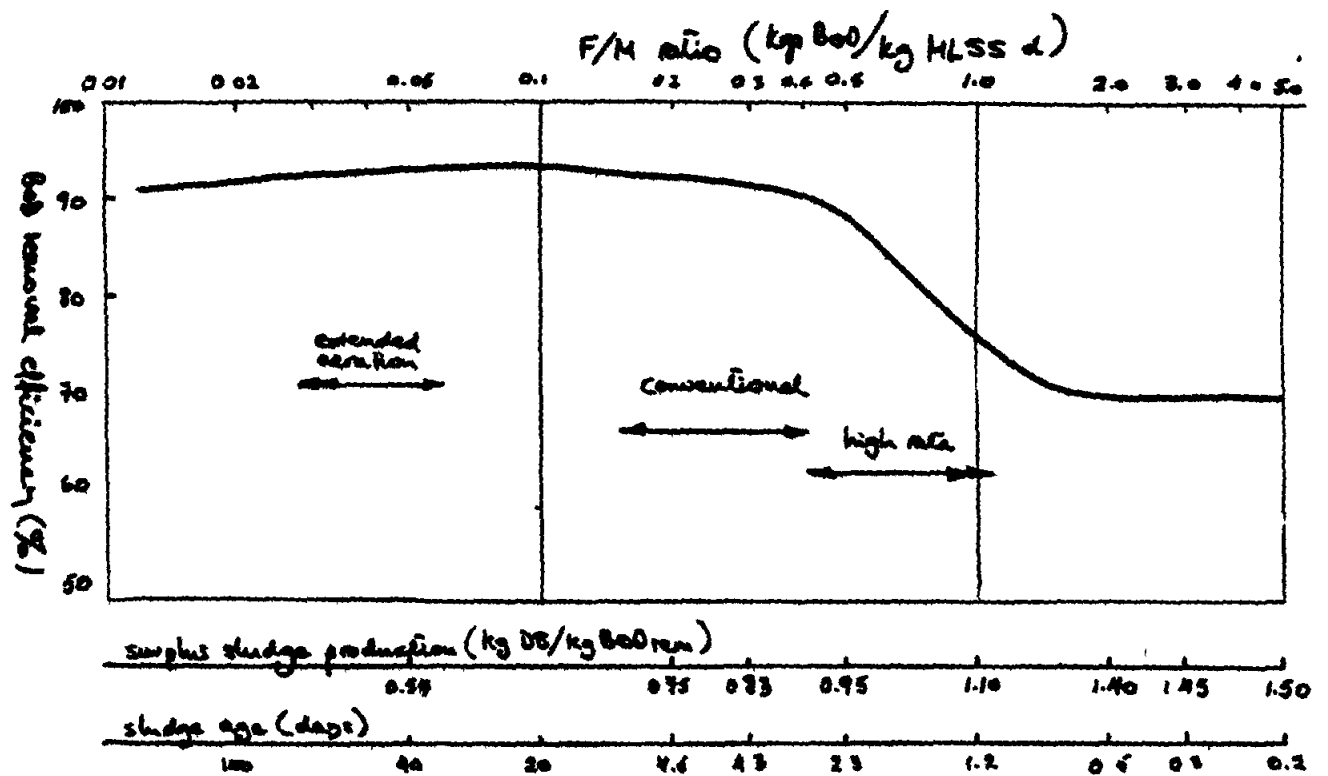
- Figure 1 - General characteristics of activated sludge systems
F/M, sludge age, efficiency and sludge production
- 2 - Concrete extended aeration tank
- 3 - Various ditch configurations
- 4 - Cage and mammoth rotors

References

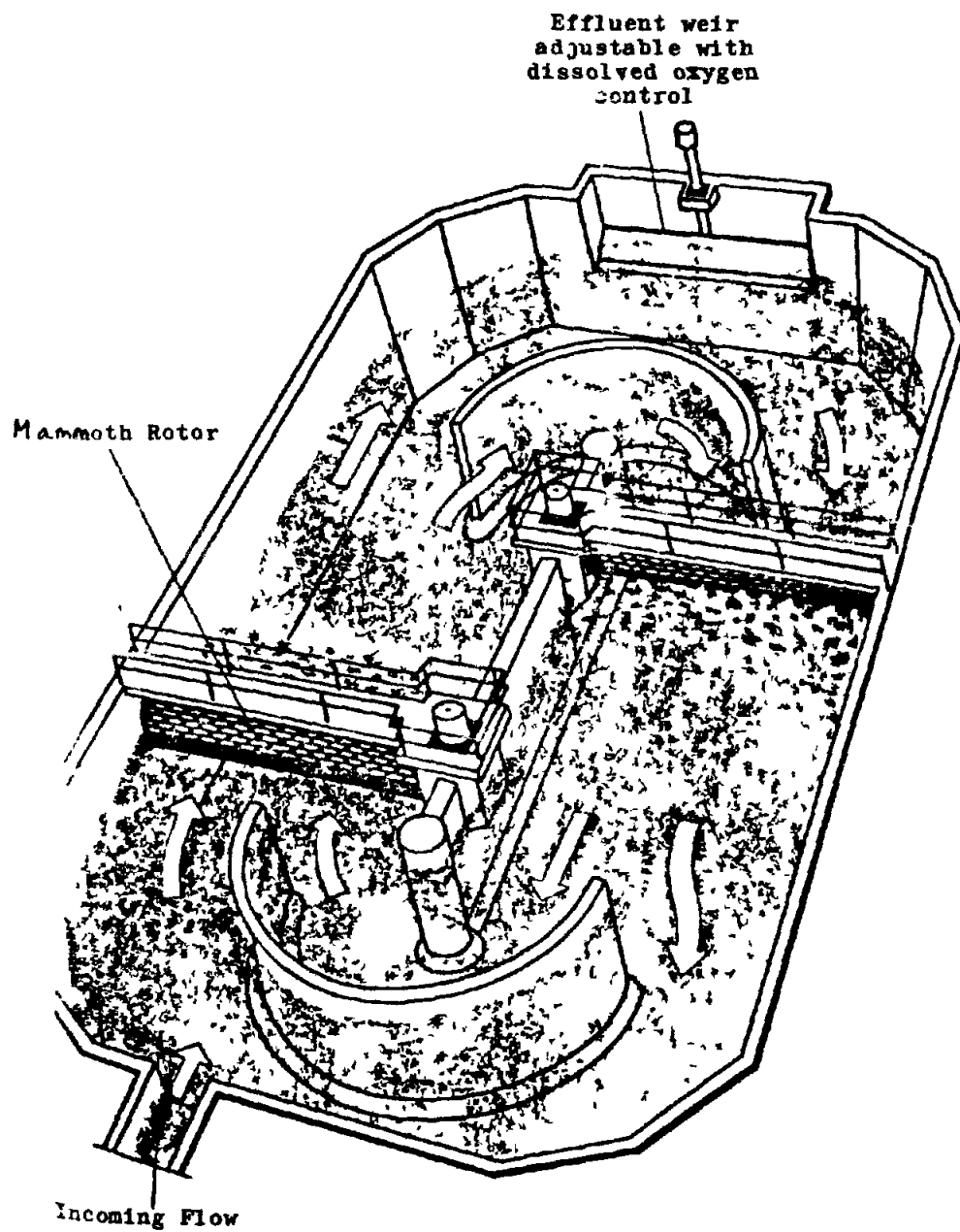
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Figure 1

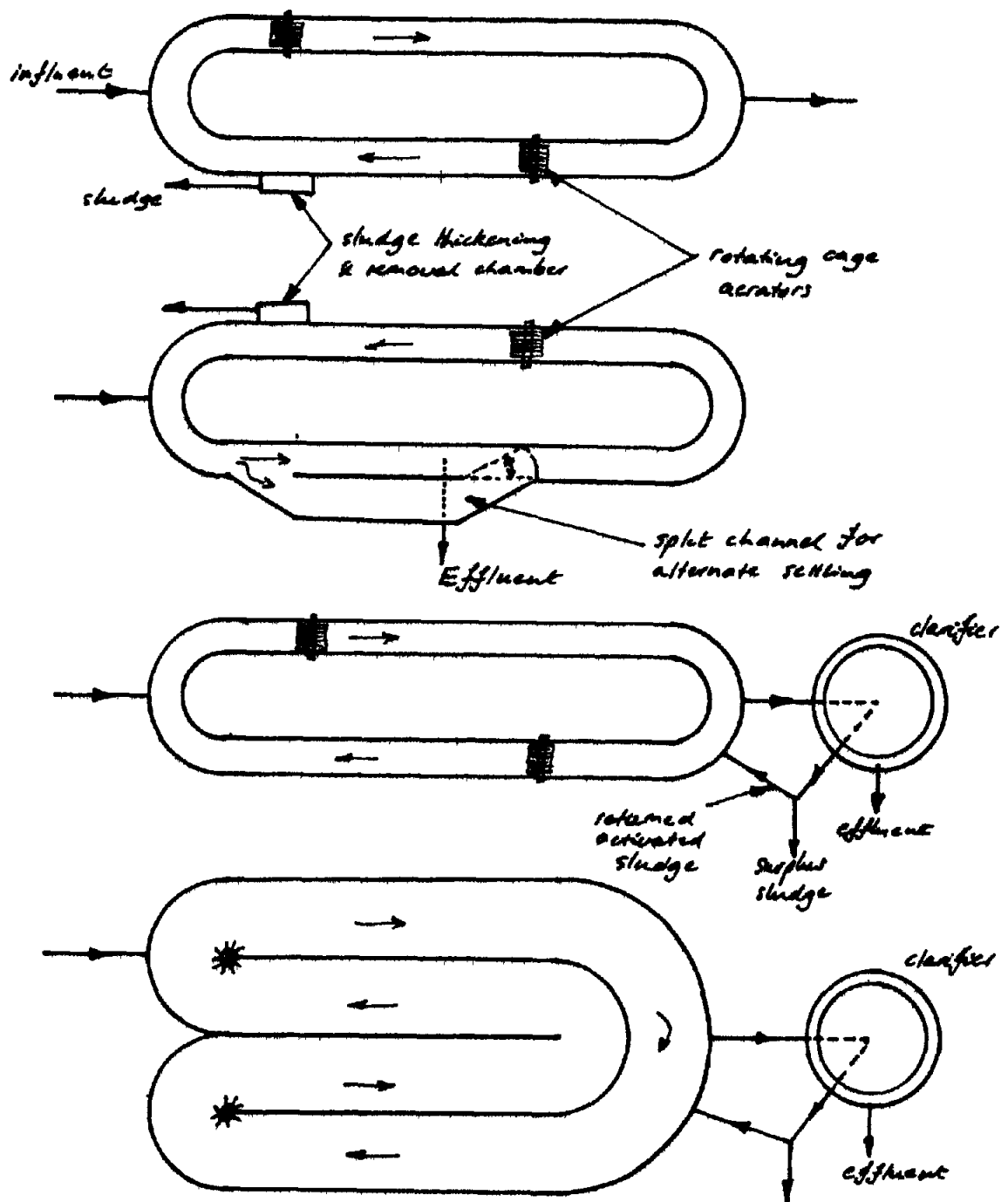


Influence of F/M on the Activated Sludge Process

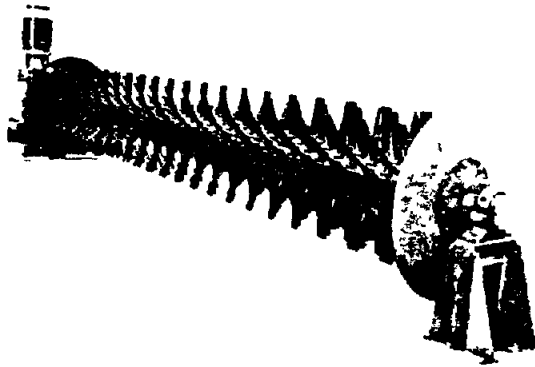


TYPICAL EXTENDED AERATION TANK

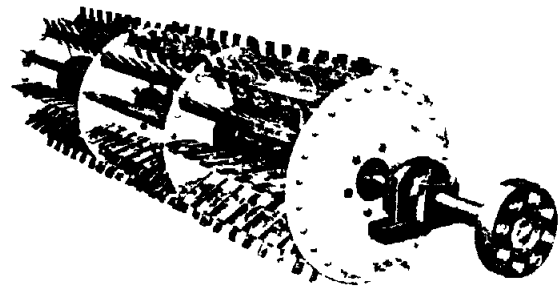
FIGURE 2



Ditch Configurations
Figure 3



G1) mammoth rotor



G11) TNO or cage rotor

Surface Aeration Equipment

Figure 4

PAPER 12

THE FACULTATIVE POND

by

K.V. Ellis

The Facultative Pond

Facultative ponds are designed either to receive the anoxic effluent from anaerobic ponds or to receive untreated wastewater.

Pre-treatment

Pre-treatment in the form of grit removal and screening can be employed but is not essential. For grit removal constant velocity grit channels (controlled by a parabolic flume) can be employed and screens will always be hand-raked. All removed detritus should be buried and covered with at least one metre depth of earth to avoid fly and rodent nuisance.

If the facultative pond is preceded by an anaerobic pond pre-treatment is not usually required as the anaerobic pond will fulfill the functions of both the grit remover and the screen. If the facultative pond is the first step of the treatment process and pre-treatment is not employed then a sump will be required under the inlet of about 1.0m extra depth and sufficiently extensive to contain all the grit received over a two-year period. It will also be necessary to rake-off and bury the floating scum and debris which would otherwise have been removed by a screen.

Depth

The optimum depth of a facultative pond is about 1.5m. This is deep enough to prevent the growth of rooted plants but not too deep to prevent solar radiation penetrating through most of the pond depth. Also the shallower the pond the more extensive is the water/air interface and the greater is the direct absorption of atmospheric oxygen.

Facultative system

As the name implies the facultative pond is a dual-layer system operating aerobically near the surface and anaerobically at the bottom. As the wastewater flows into the pond any settleable solids are deposited to form a layer of sludge over the pond bottom. One of the aims of pond design must be to spread this sludge layer over as much of the area of the pond bottom as is possible. Initially the sludge will merely settle out but if the temperature is greater than $(15^{\circ} - 19^{\circ}\text{C})$ anaerobic digestion at an appreciable rate will commence, although this might not be for several months following the pond commissioning. By nature of the situation and the limited thickness of the sludge layer (perhaps 0.25m thick) the anaerobic digestion cannot be expected to be properly balanced and, as a result, soluble partial-breakdown products - principally mainly non-degradable organic acids and alcohol - will leach into the upper aerobic layer and there create an additional oxygen demand. Periodically the accumulated sludge will have to be removed from the pond bottom although this should only be necessary after several years. To do this it must be possible to rake-off the supernatant water to allow the sludge to dry before being removed - usually manually.

Aerobic stabilization of the complex putrescent organic materials which cause pollution is both quicker and more efficient than the anaerobic process and so

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Relevant Wastewater Treatment in Developing Countries

this stratification into a narrow, hot, non-dense surface layer and the cooler, denser bulk of the pond water effectively prevents vertical mixing in the pond and results in the surface water being adequately laden with oxygen while the deeper water, away from the effect of surface re-aeration becomes progressively anoxic. Also as any inflowing fresh wastewater will tend to be warm there is a danger of short-circuiting across the surface layer and the discharge of a partially treated effluent without the benefit of the nominal retention period.

This situation is aggravated by the algae. The mobile algae tend to retreat from the intense radiation at the surface to form a thick, green layer, impenetrable to light, some distance below the surface. This distance can be between 0.3 and 0.5m but can be as little as 0.03m down. The top layer of water now gets warmer and warmer and as consequentially the viscosity of the water decreases the non-mobile algae also tend to slip down to cooler levels. The surface water above the algae will now be heavily charged with oxygen either as a result of direct absorption or as a result of intense photosynthetic action by the algae - or both. But below the layer the water will be increasingly anoxic and aerobic activity will rapidly decrease. There is then a situation in which only in a small fraction of the water depth is the required aerobic activity progressing - and if the stratification continues into the night the situation will further deteriorate as a result of algal oxygen consumption.

Marais has reported the following figures for a hot, calm day in a pond at Lusaka, Zambia at 1 O'clock in the afternoon

<u>Depth below surface</u>	<u>Temperature</u>	<u>Dissolved-Oxygen</u>
75 mm	30°C	24 mg/l
300 mm	26.5	5 "
450 mm	25.0	3 "
600 mm	24.0	2 "
1370 mm	24.0	0 "

While Stahl has reported from the USA an early afternoon temperature-drop from a depth of 10 mm to 500 mm of more than 10°C with only a further limited drop of between 1.5°C and 5°C between 500 mm and 1100 mm.

Wind action

Wind is the best available means of breaking up stratification as it both cools the water surface and creates turbulence through surface friction and wave formation. Wind always improves the efficiency of ponds and hence should be allowed free access. In addition if at all possible the major axis of a pond should always be in the direction of the prevailing wind and, again, if possible the minimum 'fetch' for the wind across the pond surface should be 100m and preferably 200m - small ponds are much more liable to stratification than large ponds. No trees should be allowed within 150 m of the pond edge so that they do not inhibit the wind effect and no solid fences should be allowed around the ponds. There is an example of the erection of a 2 m high fence around a pond causing a hitherto aerobic process to turn anaerobic, and the condition only being rectified when the fence was removed.

The important factor in overcoming stratification with wind is the pond stability which is the work required to mix the entire body of water without the addition or subtraction of heat. Stability increases with increasing pond depth, with increasing thermal stratification and, naturally, with temperature.

The effect of wind breaking up stratification is well illustrated by two examples from a pond in the USA. On a hot clear calm day the following situation

should be facilitated as much as is possible. The aerobic layer which preferably should include most of the water depth in the pond receives a biodegradable organic load

- a from the non-settleable portion of the pollution in the incoming wastewater
- b from the sludge layer in the form of soluble organic acids and alcohols as the result of partial anaerobic digestion

Oxygen sources

Oxygen is obviously essential to the operation of this layer, and it is received from two sources

- a. the direct absorption of atmospheric oxygen through the extensive air/water interface of the shallow pond and
- b. the photosynthetically produced oxygen resulting from the activity of the masses of algae that invariably appear in ponds in which conditions, in the form of available nutrients (phosphates, nitrates etc) and intensive solar radiation, favour their development

Heavy blooms of algae are capable of photosynthetically producing large quantities of oxygen in response to sunlight. During the hours of daylight, and particularly in the early afternoon, the pond water may be heavily supersaturated with dissolved-oxygen. This phenomenon has led various workers to consider that the algae are all-important in pond operation. Unfortunately algae are not only an asset, as their production of oxygen suggests, but are also a liability in the pond. As an adjunct to photosynthetic action the production of oxygen is paralleled by a rise in the pH of the water as a result of the algae employing the $\text{CO}_2/\text{HCO}_3^-/\text{CO}_3^{++}$ complex as their source of cellular carbon. This removes the natural buffering from the water and hence the rise in pH cannot be prevented. High pH's cannot be conducive to effective operation by the aerobic microorganisms present. In addition photosynthesis can only operate during daylight and must cease when the sun disappears. During the night not only are the algae not producing oxygen but they are consuming it as their respiration continues. Although the rate of removal of dissolved-oxygen by this continued respiration is only at a fraction of the rate at which oxygen is produced photosynthetically during the day the ability of water to store oxygen is very limited and the demand by a high concentration of algae can be high. As a result all available oxygen can be stripped from the pond water during the night leaving most of the pond depth anoxic and devoid of aerobic biological action.

The direct absorption of atmospheric oxygen through the extensive surface of the ponds is obviously also of great importance and this mechanism is little affected by the time of day or night. Anything that can be done to increase this effect must be done. This must include making the maximum possible use of any wind both to extend the surface area by means of developed waves and also to ensure that the near anoxic water deep in the pond is brought regularly to the surface for re-aeration.

Stratification

Stratification is the principal drawback to the efficient operation of facultative ponds. This will develop during any hot windless day and, although it will often break up during cooler nights, during prolonged hot periods it may endure for weeks or even months.

During hot days, particularly in the tropics, the surface temperature of the pond water may rise appreciably in excess of 30°C to produce a warm, non dense top water layer with a rapid thermocline just under the surface, to cooler denser layers of water. This is similar to the epilimnion and hypolimnion of deeper reservoir systems.



observed -

At the surface	317% supersaturation with d-o
50 mm depth	5% saturation " "
100 mm depth	0% saturation " "

While on a hot clear windy day

At the surface	185% supersaturation with d-o
100 mm depth	105% saturation " "

Ponds in parallel but not in series

As it is expected that under normal conditions the majority of the water in a pond will contain an appreciable quantity of dissolved-oxygen and that as a result the biological stabilization processes will be largely aerobic it is essential that the BOD of the pond water is not excessive. Normally it is considered that the BOD of a well-mixed facultative pond should not exceed 60 mg/l although there may be little harm in it exceeding this figure by even as much as 20 mg/l. It is also largely for this reason that facultative ponds are not built in series. If the BOD of the pond is appreciably in excess of 60 mg/l it will no longer be facultative but anaerobic, and if there is a pond in series with one producing a 60 mg/l BOD then it will nearly certainly be fully aerobic.

PAPER 13

PROCESS DESIGN OF WASTE STABILIZATION PONDS

by

J. Lumbers

Waste Stabilization Ponds: Design Considerations and Methods

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Paper presented at a meeting of the Midlands District Centre on 13th December 1978

INTRODUCTION

This paper attempts to bring together the many considerations necessary for the successful design of waste stabilization ponds. Design methods for anaerobic, facultative and maturation ponds together with aerated lagoons are discussed, including the facilities design.

DESIGN CONSIDERATIONS

The general design considerations for a scheme using waste stabilization ponds include the usual investigations required for any method of sewage treatment. However, the following factors are suggested as being particularly relevant to the choice of type and design of waste stabilization ponds in tropical and semi-tropical climates.

Treated Effluent Disposal Route

The effluent disposal route is a fundamental factor in the choice of any treatment method.

Where a perennial river is in the proximity of the pollution source this is generally the preferred disposal route as the self-purification capacity of the river may be utilized in addition to the basic dilution available.

Disposal to land irrigation may prove to be both a useful alternative avoiding long fluid transport distances and also providing increased crop yields due to regular irrigation. However, there are many factors which must be analysed prior to embarking on an effluent irrigation scheme these being principally

- salinity
- exchangeable sodium percentage
- sodium absorption ratio
- chemical content of effluent
- soil permeability
- soil drainage
- established irrigation methods, if any
- type of crop
- storage requirements
- alternative disposal route

Effluent disposal to the subsoil by artificial recharge can prove viable where an aquifer of suitable characteristics outcrops near the surface. It is necessary to maintain a low suspended solids content in the recharge water to avoid blockage of the aquifer, this may be achieved by lining the aquifer interface with a graded layer of sand and gravel.

Where the climate is sufficiently arid the effluent may be disposed of by a combination of evaporation and percolation.

In most instances the only viable route for effluent disposal is to a nearby perennial water-course.

River Uses

There are many possible uses for river water including

- irrigation
- wastewater disposal
- commercial fishing
- bathing
- commercial transport
- pleasure boating
- landscaping/river walks
- domestic water supply
- industrial uses
- angling

For example, in a recent case considered the uses of the river water were identified as

- domestic water source
- decorication of sial
- irrigation

These water rights formed the basis for the specification of river water quality requirements. In many developing countries untreated river water is used for domestic purposes and it is necessary to establish chemical and bacteriological standards with this fact in mind.

River Quality

A typical recommended standard for the quality of river water for use as a potable source to be given normal water treatment is given in Table 1. These figures may be compared with the World Health Organization standards for drinking water summarized in Table 2.

A knowledge of the existing water quality is essential to the establishment of realistic standards as in many instances in developing countries the river water may be contaminated already. In these circumstances it would be better from an overall economic viewpoint to invest in a water treatment facility for the affected population using the capital saving resulting from a works designed to achieve a lower treated effluent standard.

River Low Flows

For the determination of suitable effluent standards in the absence of any general legislation, the low flow river condition must be established for a given return period of say once in five years. The low flow to be considered may be the mean monthly, weekly or daily minimum. Where poor records exist it may be possible to in-fill the data by flow simulation methods. Where no information exists flow gauging over a short term coupled with general information obtained from the local inhabitants is the last resort.

Algal Toxic Tolerances

Algae are essential to the operation of waste stabilization ponds and it is advisable to check the levels of substances such as copper and cadmium which are algicidal at low concentrations. The LC 50 of copper for algae is quoted as between 0.02 and 2 mg/l and cadmium has been found to inhibit algae growth significantly at concentrations as low as 0.006 mg/l. Other toxic substances effecting a 50 per cent chlorophyll reduction are given by Gloyna³.

Details of Industries

Establishing the probable flows of effluent from industrial premises can be difficult, as although most industrialists are in favour of pollution control measures they are often reluctant to divulge specific information which may increase their pre-treatment requirements. Therefore all projected effluent flows and constituents must be compared with previous experience and should be based on a knowledge of

- product production
- the number of workers
- the number of shifts per day
- the factory processes and effluent sources
- seasonal effects
- cleaning methods
- product washing methods
- peak flow rates
- pre-treatment facilities and balancing

Nutrient Balance

The principal nutrients required for biological growth are carbon, nitrogen and phosphorus and the essential ratios of BOD : N : P are generally 100 : 5 : 1. The usual content of purely domestic sewage is about 350 mg/l BOD, 125 mg/l available carbon, 40 mg/l of N and 11 mg/l of P as reported by Oswald⁴. Oswald also reports that nitrogen often has been found to be the limiting factor for algal growth with essentially no growth occurring below 0.3 mg/l of nitrogen, whereas a maximum specific growth rate for *Chlorella* was observed at a background phosphate level of 0.05 mg/l.

Environmental Factors

The choice of the type of pond and the location should be made in the light of the possible adverse environmental impacts of such factors as odour, mosquito breeding, schistosomiasis (bilharzia), rodent breeding, flies, noise, etc. Odour problems often arise from the use of anaerobic ponds although this can be alleviated to some extent by recirculation.

reducing the areal loadings or pH elevation⁵. Mosquito problems have been shown to be countered by regular pond maintenance to restrict weed growth⁶ in particular this also assists in the control of schistosomiasis⁷.

TABLE 1
Standards for River Water Quality for Potable Use given Normal Treatment

Colour	Limiting value 300 Hazen units	
	Preferred level mg/l	Max level mg/l
Total dissolved solids	500	1500
Iron	0.5	5
Manganese (assuming ammonia 0.5 mg/l)	1.0	5
Copper	1.0	1.5
Zinc	5	15
Magnesium plus sodium sulphate	500	1000
Detergents ABS	—	1.0
Nitrate as NO ₃	—	45
Fluoride	—	1.5
Phenolic compounds	—	0.002
Arsenic	—	0.05
Cadmium	—	0.01
Chromium	—	0.05
Cyanide	—	0.20
Lead	—	0.10
Selenium	—	0.01
	mg/l	
Chemical oxygen demand (COD)	10	
Biochemical oxygen demand (BOD)	6	
Total nitrogen excluding nitrate	1	
Ammonia	0.5	
Organic pollutants	0.5	
Grease	1	
Coliforms	MPN/100 ml	
for disinfection only	0-10	
for conventional treatment of coagulation, filtration and disinfection	50-5 000	
for extensive treatment	5 000-50 000	
to be avoided	> 50 000	

Specific Design Factors

The specific design factors required for the various design methods include

- ambient temperature
- altitude
- wind velocity and direction
- solar radiation
- cloud cover
- strength of raw wastewater
- water temperature

DESIGN METHODS ANAEROBIC PONDS

The commonly used criteria for the design of anaerobic ponds are

- surface loading of BOD 1000-6000 kg BOD/ha d⁵
- retention 2-5 days^{8, 11}
- volumetric loading of BOD 0.19-0.24 kg BOD/m³ d⁹
- depth 3-5 m

The principal requirements for successful operation are

- exclusion of toxic and inhibitory substances
- approximately continuous loading
- temperature in excess of 20°C
- adequate mixing

Additional factors which may require monitoring are

- volatile acid concentration
- alkalinity
- pH range 6-8
- oxidation-reduction potential (ORP)

In addition to the design criteria above empirical relationships have been developed. Vincent *et al*¹⁰ give the following relationship

$$L_e = \frac{L_i}{k_n \left[\frac{L_e}{L_i} \right]^n R + 1}$$

TABLE 2
Drinking Water Standards World Health Organization 1971

Characteristic	Units	Preferred	Maximum
Colour	Hazen	5	50
Turbidity	Units	5	25
Total solids	mg/l	500	1500
pH	—	7.0-8.5	6.5-9.2
Anionic detergents	mg/l	0.2	1.0
Mineral oil	mg/l	0.01	0.3
Phenolic compounds	mg/l	0.001	0.002
Total hardness	mg/l CaCO ₃	100	500
Calcium	mg/l	75	200
Chloride	mg/l	200	600
Copper	mg/l Cu	0.05	1.5
Iron	mg/l Fe	0.1	1.0
Fluoride	mg/l F	—	0.8-1.7
Magnesium (sulphate-250 mg/l)	mg/l Mg	30	150
Manganese	mg/l Mn	0.05	0.5
Sulphate	mg/l SO ₄	200	400
Zinc	mg/l Zn	5	15
Arsenic	mg/l As	—	0.05
Cadmium	mg/l Cd	—	0.01
Cyanide	mg/l CN	—	0.05
Lead	mg/l Pb	—	0.1
Mercury (total)	mg/l Hg	—	0.001
Selenium	mg/l Se	—	0.01
Nitrate	mg/l NO ₃	—	45

95 per cent of samples throughout the year should contain less than one coliform per 100 ml and not exceed 10 coliforms per 100 ml

Source: International Standards for Drinking Water World Health Organization 1971

where L_e = effluent BOD (mg/l)

L_i = influent BOD (mg/l)

k_n = rate coefficient

n = constant

R = retention (day)

Work in Zambia produced values of k_n and n of 6.0 and 4.8 respectively for a pond temperature of 20°C

It is now generally accepted that the volumetric BOD loading is the most relevant design criterion^{5, 11} provided that the depth is sufficient to maintain a predominantly anaerobic regime. A retention period of 2 to 7 days¹² is generally required depending on the ambient temperature. Retention periods longer than about 5 days appear to have little effect on the removal of BOD for a given loading rate³ and 5 days appears to be the optimum for year-round operation⁸.

DESIGN METHODS FACULTATIVE PONDS

There are numerous design methods based on empirical formulae, statistical analysis and rule of thumb from which a wide range of designs can be obtained. A survey of design methods was undertaken by the author in 1975 as part of Howard Humphreys and Partners internal technological up-dating programme in which it was concluded that the McGarry & Pescod formulae⁵ give a reasonable design under most circumstances. This conclusion was confirmed independently by Mara⁸ in 1976.

The principal design methods in most common use are given below as named by the author

- Global Environmental Design Basis
- Gloyne's Empirical Formula
- First Order Kinetics
- Indian Approximate Procedure
- Solar Radiation Method
- Thirumurthi Method
- McGarry & Pescod Formulae

A comparison of designs for a particular location, using these methods, is given in Table 3, from which it can be seen that the McGarry and Pescod method provides average values

TABLE 3
Comparison of Facultative Pond Designs

Method	Retention (days)	Areal BOD loading (kg BOD/ha d)
Global Climate	33-17	150-350
Gloyne's Formula	43	96
1st Order Kinetics	13	319
Solar Radiation	11	373
Thirumurthi	19	217
McGarry & Pescod	15	267

Global Environmental Design Basis

Gloyna¹³ gives a table of acceptable organic loadings to facultative ponds which have been used successfully in various environments. The loadings given are based on the following assumptions

- the total evaporative and seepage loss is not greater than the influent volume
- the *per capita* BOD contribution is 50 g BODs per day
- the wastewater production *per capita* day is 100 litres

The surface loadings quoted are given in Table 4

TABLE 4

Global Environmental Design Basis		
Surface Loading (kg BODs/ha d)	Retention Period (days)	Environment
10	200	Frigid zones icy low water temperature, variable cloud cover
10-50	200-100	Cold seasons icy temperate summer season (short)
50-150	100-33	Temperate to semi-tropical No prolonged cloud cover Occasional ice cover
150-350	33-17	Tropical uniform distribution of sunshine and temperature No seasonal cloud cover

Obviously these generalized values should be used only where limited information precludes the use of more refined methods

Gloyna's Empirical Formula

An empirical formula based on the operational results from many small laboratory experiments larger pilot scale studies and several hundred full scale ponds, relating pond volume to temperature influent BOD and wastewater flow was first developed in 1958 and is given by Gloyna¹³

$$V = (3.5 \times 10^{-5}) N q L_0^{(15-T_m)}$$

where V = pond volume (m³)

N = contributing population

q = *per capita* wastewater flow (l/d)

θ = temperature reaction exponent = 1.085

T_m = average water temperature on the coldest month (°C)

L_0 = Ultimate Biochemical Oxygen Demand (mg/l)

This formula has now been updated³ to include factors for the toxicity content and sulphide content. The retention periods based on this equation are shown on Figure 1 for various influent organic loads assuming the ultimate BOD is 1.4 times the 5 day BOD

Gloyna³ recommends the following guidelines for the determination of depth

Depth in metres	Conditions
1.00	Generally ideal conditions very uniform temperature tropical to subtropical minimum settleable solids
1.25	As above but with modest amounts of settleable solids. Surface design based on one metre depth and 0.25 metre used for reserve volume
1.50	As for 1.25 m except for significant seasonal variation in temperature major fluctuations in daily flow. Surface design again based on one metre
2 and over	For soluble wastewaters which are slowly biodegradable and retention is controlling factor

First Order Kinetics

The formula for the first order reduction of BOD is

$$L_e = \frac{L_0}{k_7 R + 1}$$

where L_e , L_0 = influent effluent BOD (mg/l)

R = retention (days)

and k_7 is the BOD reduction rate constant in the pond environment k_7 varies with temperature according to the equation

$$k_7 = k_{35} \times 1.085^{(T-35)}$$

where k_{35} = 1.2 in southern Africa

and T = pond operating temperature, °C

This formula is based on the assumption of a completely mixed reactor whereas most pond systems are far from this condition. Therefore as would be expected the formula tends to over-estimate the BOD reduction¹⁴ although a judicious choice of the value of k_7 can counter this particularly when based on local operative experience in other pond systems

Indian Approximate Procedure

It has been found in India that the areal BOD loading to facultative ponds can be related approximately to the latitude. An increase of 25 kg BOD/ha d for every 4° change in latitude produces a range of loading rate from 325 to 150 kg/ha d for a latitude change from 8°N to 36°N. Extrapolations outside India are not recommended. The values are only approximate and are based on an altitude of sea level, clear skies for 75% of the year and a pond depth of 0.9 to 1.5 metres¹²

Solar Radiation Method

This method was originally developed by Oswald and Gotaas and is based on

- average monthly maximum and minimum solar radiation
- average monthly sky clearance factors
- algae energy requirement and utilization efficiency
- oxygen production rate by algae

Probable average values for visible solar radiation may be calculated from the information published by the United States Weather Bureau and typical derived figures for sea level are quoted by Arceivala¹². The average radiation is then calculated for the lowest month of the year

$$\text{Average radiation } S_0 = \frac{\text{minimum visible rad} + (\text{max} - \text{min}) \times \text{sky clearance factor}}{\text{total possible sunlight hours}}$$

where the sky clearance factor = $\frac{\text{actual hours of sunlight}}{\text{total possible sunlight hours}}$

The visible solar radiation values should be corrected for altitude

$$S_E = S_0 (1 + 0.00925 E)$$

where S_E = visible radiation at elevation E (g cal/cm² d)

S_0 = visible radiation at sea level

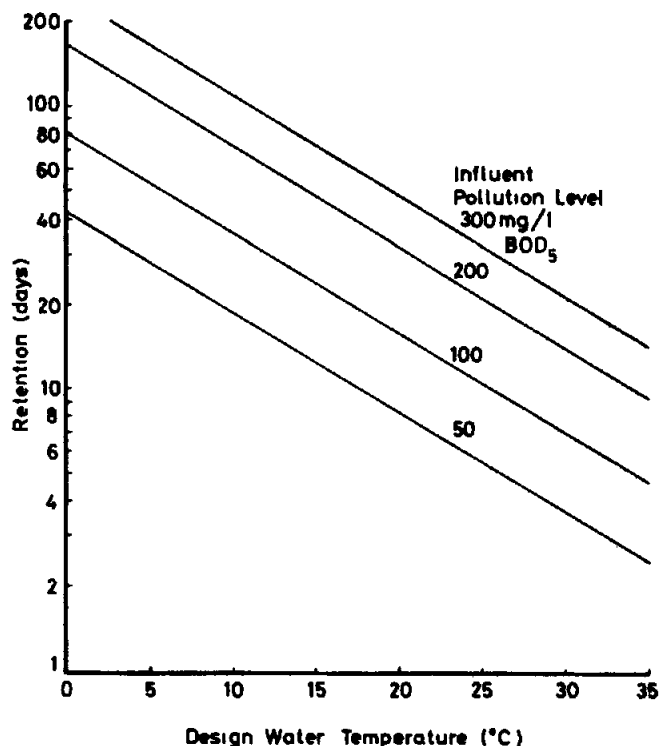
E = elevation above sea level (ft $\times 10^{-3}$)

The energy requirements for the formation of one gram of algae is in the region of 4 to 6 kilo-calories³ and the utilization efficiency of conversion of visible energy is 4 to 6 per cent

The mass of algae produced is given therefore by

$$\text{mass of algae} = \frac{0.06 \times S_E \text{ average}}{6 \times 10^6 \times 10^{-6}} \text{ kg/ha d}$$

The oxygen production by algae is estimated in the order of 1.25 to 1.75 g/g of algae produced¹². In warm climates oxygen production rates of 500 kilograms O₂ per hectare day have been attained although 200 kilograms per hectare day is an average realistic figure and a useful check on design¹⁵



$$\text{Retention} = 3.5 \times 10^{-2} \times 1.4 \times \text{BOD}_5 \times 1.085^{(35-T)}$$

Figure 1 Gloyna's Empirical Equation

There are different design approaches relating the oxygen produced by the photosynthetic activity of algae to either the BOD_s or ultimate BOD removed or applied to the pond system^{16, 17}. Siddiqi and Handa¹⁶ propose the use of a load factor defined as the ratio of BOD_s load to the oxygen produced by algae, although they agree it would be preferable to use ultimate BOD rather than the BOD_s value. They report that Oswald suggested a lower limit of 0.44 and themselves suggest a value of 8.0 as an upper limit to maintain facultative conditions. Arceivala *et al.*¹² state that the ultimate BOD load to the pond should not exceed the oxygen produced by algae and, although this ignores the reduction of BOD by settlement and anaerobic decomposition, it provides a safe design basis. Mara⁸ suggests a safety factor not greater than 1.07 times the oxygen produced to give the design load of ultimate BOD. This factor is included to allow for not all of the oxygen produced being utilized for organic stabilization, but again no account is taken of the BOD reduction by settlement. In addition to the settlement phenomena any theoretical relationship between load and algae oxygen should include natural surface reaeration as an oxygen source.

Thirumurthi Method

The Thirumurthi¹⁸ method is a rational approach based on a non-ideally mixed reactor in which the degree of mixing is represented by a constant d ranging from zero for plug flow conditions to infinity for completely mixed systems. The usual BOD decay rate constant, k_T , is determined from the influent BOD level, values of the product k_T times the retention time are related to the percentage BOD remaining by the equation solutions represented in Figure 2. The value of d is unlikely to exceed 1.0 for waste stabilization ponds but even between a d value of 0.25 and 1.00 the calculated retention can vary by 65 per cent. Thirumurthi suggests tracer studies or model tests should be undertaken to determine the appropriate d value for the pond configuration.

The determination of the BOD decay rate constant, k_T , is also discussed by Thirumurthi and relationships are proposed for a dependency on the organic loading rate and solar radiation levels in addition to temperature. A dependency of k_T on the organic loading rate has also been observed by Siddiqi and Handa¹⁶ reducing from a value of 0.22 in the first pond to 0.06 in the third pond.

McGarry and Pescod Formula

McGarry and Pescod⁵ analysed data from 143 primary facultative ponds and found a very definite relationship exists between the areal removal of BOD and the influent BOD value. The regression line takes the form

$$L_r = 10.37 + 0.725 L_u$$

where L_r = areal removal of BOD (kg BOD/ha day)

and L_u = areal loading of BOD (kgBOD/ha day)

The 95 per cent confidence limits for this regression were ± 32.8 kg/ha d.

A further equation is given describing the maximum feasible loading of a primary facultative pond related to the mean monthly ambient temperature taking the form

$$L_u = 11.2 (1.054)^{T_m}$$

where T_m = mean monthly ambient temperature °F

Mara¹⁹ proposes the use of a safety factor of 1.5 to allow this equation to be used for design. Using this factor the results were compared with those of the modified first order kinetics approach developed by Marais and Shaw and described by Gloyna¹³ as the South African procedure. However, this comparison is only valid if the temperature values used in the two methods are consistent. The McGarry and Pescod formula uses mean monthly ambient temperature whereas the BOD decay rate constant used in the first order kinetics method is usually related to pond liquid temperature. Marais²⁰ correlated monthly mean maximum and minimum air temperature with pond liquid temperature and produced an equivalent value of the decay constant based on air temperature extremes. Pond temperatures are usually a few degrees centigrade above the mean ambient temperature except in the hottest months^{12, 26}.

DESIGN METHODS AERATED LAGOONS

Aerated lagoons are generally designed on the principle of a completely mixed steady-state reactor using the first order kinetics theory³⁰

$$\text{Retention} = \frac{L_0/L_e - 1}{k_T}$$

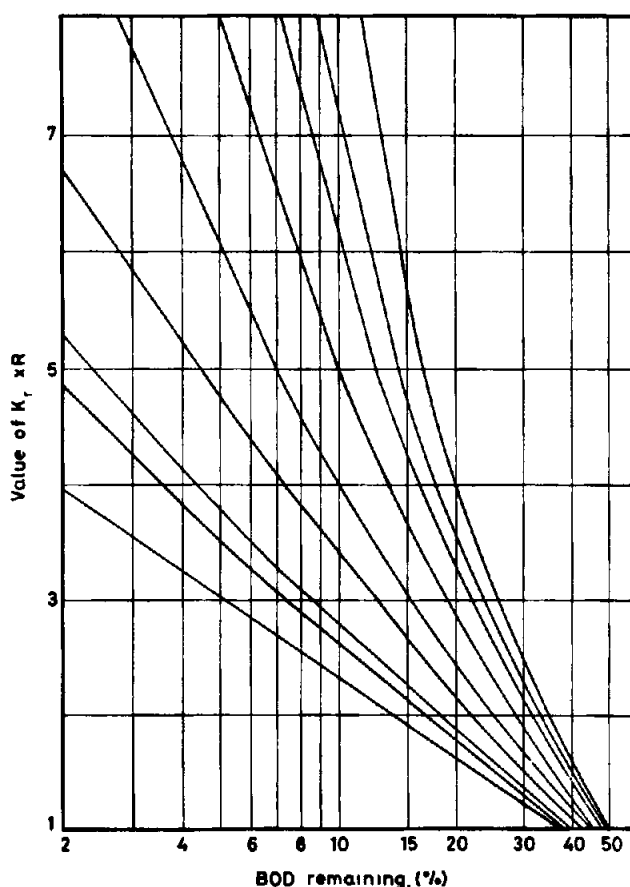
where L_0 , L_e = influent, effluent BOD (mg/l)

$$k_T = k_{20} \theta^{(T-20)}$$

$$k_{20} = 0.5$$

$$\text{and } \theta = 1.055$$

The oxygen input is related to the ultimate BOD removed and must be adjusted for temperature and atmospheric pressure (altitude) so that the aerator efficiency under standard conditions may be specified



K_T = BOD removal rate constant related to BOD load

R = retention period (days)

Figure 2 Design Chart Thirumurthi Method (The curves from left to right represent values of $d = 0, 0.06, 0.1, 0.25, 0.5, 1, 2, 4$ and ∞ respectively)

Pearson³¹ proposed the simplified equation

$$O_2 = \frac{O_{2u} C_1}{A ((C_{1u} \times P/P_s) - C_L) + 0.2(T-20)}$$

where O_{2u} = oxygen required under standard conditions

O_{2a} = oxygen required under actual conditions

C_1 = dissolved oxygen saturation concentration at 20°C

A = oxygen transfer efficiency

C_L = oxygen concentration to be maintained in lagoon at operating temperature

C_{1u} = dissolved oxygen saturation concentration at lagoon temperature

P_a = barometric pressure (mm Hg)

P_s = standard barometric pressure

T = temperature of lagoon

The atmospheric pressure may be related to altitude by the following formula

$$P = 760 \left(1 - \frac{0.13E}{1000} \right)$$

where E = elevation above sea level (m)

P = barometric pressure (mm Hg)

Typical oxygen transfer efficiencies in lagoons are in the order of 0.8 to 1.0 kg O₂/kWh. For facultative lagoon operation the power input should not exceed about 4 watts/m³, whereas for complete mixing power inputs in the order of 20 watts/m³ are necessary, depending on the shape. Typical depths range from 2 to 5 m and should be determined with consideration to the type of aeration system to be used. Retentions are typically between 3 and 6 days with overall BOD loadings in the order of 0.016 kg BOD/m³ d.

Mara⁸ proposes a refined design approach in which the ultimate BOD of the effluent is calculated from the incoming soluble BOD not stabilized in the lagoon plus the ultimate BOD due to the bacteria cells synthesized in the lagoon. This phenomenon was originally highlighted but not included by Eckenfelder³².

An alternative design approach would be to apply the basic activated sludge theory to determine the effective volume of a lagoon required assuming the F/M ratio appropriate for extended aeration. Consideration of this theoretical approach suggests that the performance of aerated lagoons could be optimized by adjusting the suspended solids content in the lagoon, perhaps by varying the degree of mixing of the contents or by returning a proportion of the settled sludge to the lagoon inlet. However, this does not take into account the ultimate oxygen demand of sludge deposits nor of the new cells synthesized and one is left with the conclusions of Bartsch and Randall³⁰ that, although good performance of aerated lagoons has been observed in many situations there is a need for additional information to determine in particular

- the validity of the first order kinetics theory
- the effects of basin geometry, aerator design and power input on the BOD removal coefficient
- temperature effects

DESIGN METHODS MATURATION PONDS

The principal purpose of maturation ponds is the removal of enteric bacteria and viruses, other purposes include the reduction of BOD and the removal of nutrients.

First order kinetics theory is generally used to predict bacterial die-off assuming complete mixing and steady-state conditions. For a number of ponds in series the effluent faecal coliform concentration is predicted by the following equation

$$N_e = \frac{N_i}{(K_T R_1 + 1)(K_T R_2 + 1) \dots (K_T R_n + 1)}$$

where N_e = effluent faecal coliform count per 100 ml

N_i = influent faecal coliform count per 100 ml

K_T = faecal coliform removal rate constant at temperature $T^\circ\text{C}$

R_n = retention of n th pond

K_T is very temperature-dependent and is calculated by the following equation³³

$$K_T = 2.6 \times 10^{-19} T^{-20} \text{ for } T \text{ between } 5 \text{ and } 21^\circ\text{C}$$

Mara³³ has shown that the most efficient system is composed of ponds having equal retention periods.

Gloyne³⁴ has observed that various bacteria die off at different rates and that the overall die-away is greater when a large variety of algal species are present. Bacterial removal was found to be greater with an initial anaerobic pond. Although some aftergrowth was observed in *E. Coli* and *Pseudomonads*, this was not observed for the pathogens.

Maturation ponds are generally designed to have a retention period of 3 to 7 days with a depth of 1 to 1.5 m. An increased depth at the outfall point allows the effluent take-off level to be selected to achieve a minimum algal content in the effluent.

Where the effluent is to be re-used for agricultural irrigation the effect of evaporation on the concentration of salts and toxics in the effluent should be checked, particularly in arid climates.

The theoretical reduction in BOD in a maturation pond is not dealt with separately in the literature but it is suggested that the design criteria for facultative ponds provide a safe design. Mara⁴ reports that provided the influent BOD is less than 75 mg/l the effluent from two maturation ponds in series each having 7 days retention should have a BOD of less than 25 mg/l.

Oswald⁴ discusses the design considerations of nutrient removal employing algal growth.

FACILITIES DESIGN

The facilities design should include at least the following factors

- precipitation
- percolation
- evaporation
- outflow analysis
- orientation, shape, configuration and position
- inflow and outflow structures
- embankments
- wave protection
- mixing
- groundwater level
- flood protection
- sludge accumulation and removal
- pond lining
- future expansion

Precipitation

It is necessary to have a knowledge of the average rainfall figures throughout the year so that the effects of dilution and reduced retention

period can be accounted for in the design. Precipitation records are available more often than those for evaporation and locally collected information should be sought wherever possible. Where few data are available a rough guide may be obtained of general world wide precipitation from the Handbook of Applied Hydraulics²¹.

Percolation

Percolation into the subsoil may be estimated from the type of soil and the depth of the water table below the base of the pond. For unlined ponds the initial percolation rate is usually very much higher than that finally achieved due to the clogging of the pond bottom by fine particulate matter and organic growth. In fine sand, for example, an initial rate of 15 cm/day was observed to decline gradually to 1 cm/day^{12, 22}.

An in-situ test which may be used to approximate the initial percolation rate and predict the final percolation rate has been proposed by Oswald²³ and is shown in Figure 3.

Knowledge of the likely percolation losses are required in order that the effective retention may be calculated and the desired water depth range maintained. Where the treated effluent is re-used for irrigation purposes for example it is desirable to minimize the seepage losses. Conversely, where the aim is the ultimate disposal of the effluent such measures as rotation of pond usage and bottom scarification should be taken to maximize seepage. Care must be exercised to avoid polluting an aquifer used for a potable water supply source. Additional measures to enhance the overall percolation rate may include the application of a sand/gravel layer on the surface of the exposed aquifer. Such a layer should be designed to achieve a maximum amount of infiltration prior to clogging while protecting the aquifer from solids intrusion²⁴.

Evaporation

In hot climates evaporation losses can significantly reduce the effective retention in a pond and can cause a drop in water level to below that

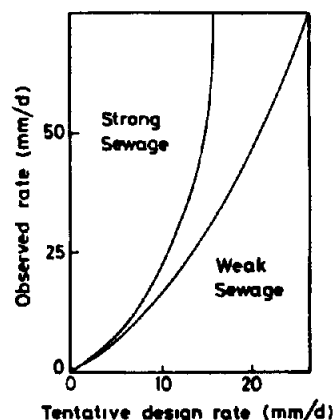
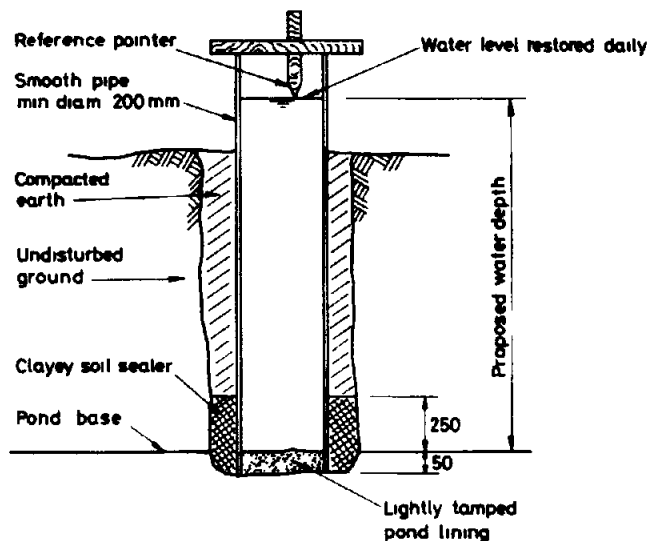


Figure 3 In Situ Percolation Test and Design Curve (After Oswald²³)

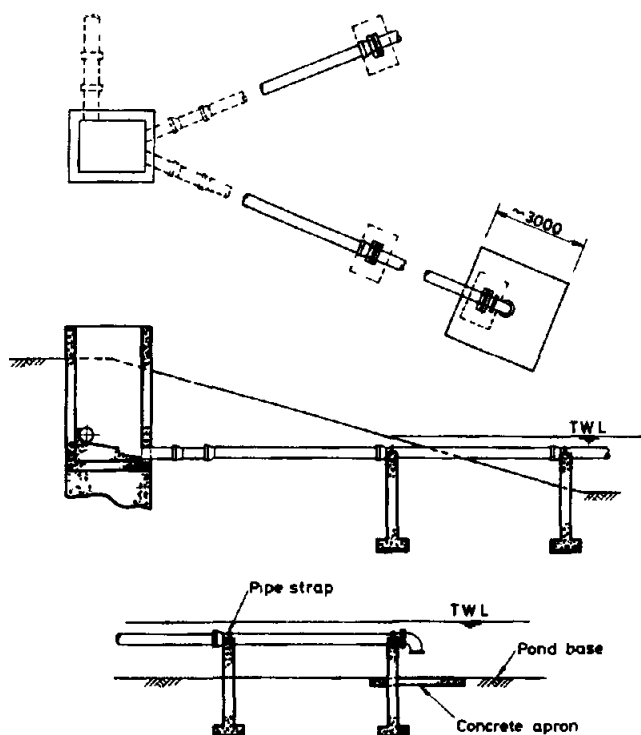


Figure 4 Pond Inlet

required for satisfactory operation. In addition the concentration of pollutants in the pond may be increased to a level which may cause operational difficulties and subsequent environmental pollution.

Evaporation data are not always available for specific locations and where no records exist a nomograph produced by Kohler *et al*²⁵, relating mean daily air temperature, mean daily dew-point temperature, wind velocity and solar radiation may be used to approximate "lake" evaporation. Such estimates should be confirmed by locally collected data whenever possible.

Outflow Analysis

An analysis of the combined effects of precipitation, percolation and evaporation on the overall flow through the ponds should be undertaken to check fluctuations in water level and the minimum retention likely to be obtained.

Low Initial Flows

The above outflow analysis is of particular relevance to the early stages of operation during which flow rates very much lower than the design flow may be encountered. Where possible initial low flows should be taken into consideration in the design of the size, number and configuration of the pond system so that maximum flexibility of operation is provided.

Pond Orientation, Shape, Configuration and Position

The directions of maximum wind-induced mixing and of maximum sunlight are the principal factors in the selection of the ideal orientation of a pond system, but in many instances site area and topographical considerations are predominant.

Rectangular ponds with a length to breadth ratio of 2-3:1 have been found to minimize short-circuiting and to be the most simple to construct²². Mara⁸ suggests that a deepening at the inlet end of a long narrow pond is advantageous in providing an anaerobic zone.

Generally it is preferred to divide a pond system into at least two parallel streams of ponds to facilitate draining down for maintenance and to allow flexibility of operation, particularly at low flows. The aspect of wave protection is discussed later but as a guide to the maximum size of any pond, areas less than 12 ha have been quoted as not usually giving wave erosion problems²².

A pond system should be positioned taking into account the probable environmental impact on local residents. There are a number of local requirements regarding the distances of ponds from residential areas, a typical figure being 0.5-1.0 kilometres for a facultative pond and more for an anaerobic pond.

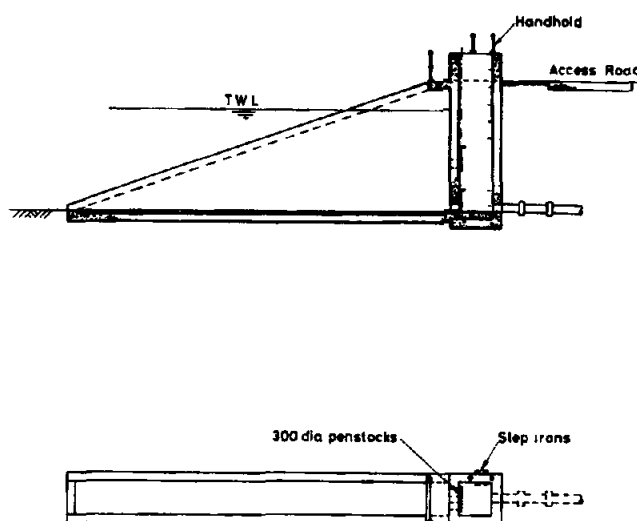


Figure 5 Drain-Down Chamber

The ponds should be located to avoid flood damage on a site having room for future expansion and of course as close as possible to the treated effluent disposal point while maintaining if at all possible, a gravity flow of sewage to the works.

Inflow and Outflow Structures

It is considered that multiple pond inlets should be used for all but small ponds^{12, 13, 22}. Inlets to small ponds and anaerobic ponds should be at the centre of the pond, whereas for larger ponds the inlet points should be positioned about 60 m from the nearest bank^{22, 13} and about 25 m for medium sized ponds⁸. Opinions differ as to whether the outlet end should be straight, down-turned or up-turned from a simple consideration of hydraulics and cost a straight pipe gently sloping towards the pond base would be preferred.

In all cases pond inlets should be submerged to encourage rapid mixing and reduce the probability of odour nuisance.

A bar screening facility for the raw sewage usually should be provided to remove large floating objects and debris from the main flow.

Flow measurement by flume or V-notch is desirable both for performance monitoring and also for future extensions design.

A typical inlet to a primary pond is shown in Figure 4.

Drain-down chambers should be provided to allow emptying for maintenance and sludge removal. The outlet arrangement should facilitate removal of top waters only if required, this may be achieved simply by two penstocks at different levels as shown in Figure 5, for example or by a bottom penstock and floating arm draw off facility. A less sophisticated design would comprise a single valved pipeline through the base of the embankment.

Interpond connections may be in the form of a single pipe but should always be valved¹³. Mara⁸ suggests that where the water level differential between ponds exceeds 1 m then an inlet chute should be used. To obtain maximum operational flexibility a central feeder collector and by-pass pipe can be incorporated as illustrated in Figure 6 for a six-pond installation.

Pond outlets usually consist of an overflow weir and a scum board. Both the weir and scum board should be adjustable so that the water depth of the pond may be controlled and the depth of extraction varied to meet seasonal changes in the water quality at different depths. A typical weir outlet chamber is shown in Figure 7.

Embankments

The height of an embankment above the pond base is based on the design maximum water depth, the freeboard requirements for storm-water flows, spare capacity, an allowance for sludge accumulation and the expected wave height.

The embankment slope is not usually steeper than 1:3 depending on the soil stability and the crest width between three and five metres, the latter being preferable for easy access and equipment handling. The size of embankments is often dictated by the general objective to balance the earthworks cut and fill. Organic material should be excluded from the material used to form embankments.

Embankment erosion, other than at the water-edge, is reduced by good compaction and a surface layer of stones on the inside slope, and the planting of a hardy grass species on the outside slope.

Typical embankment designs are shown in Figure 8.

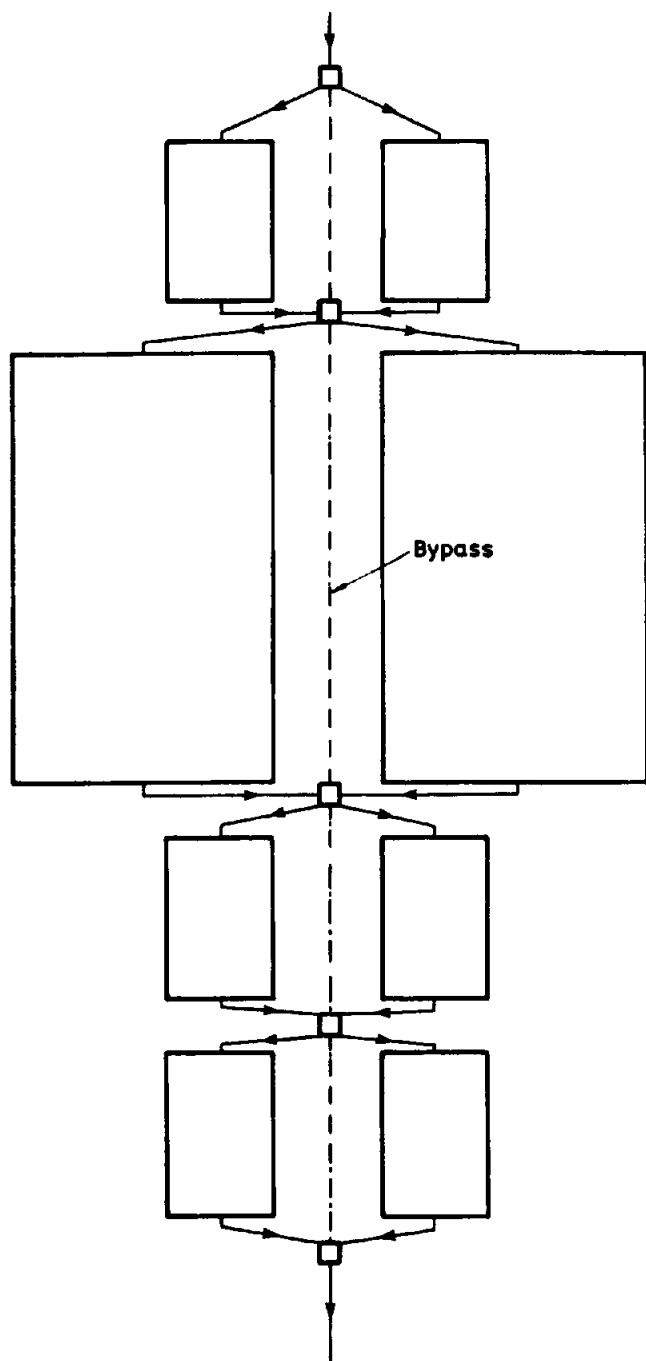


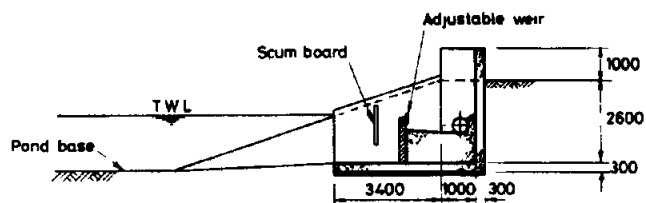
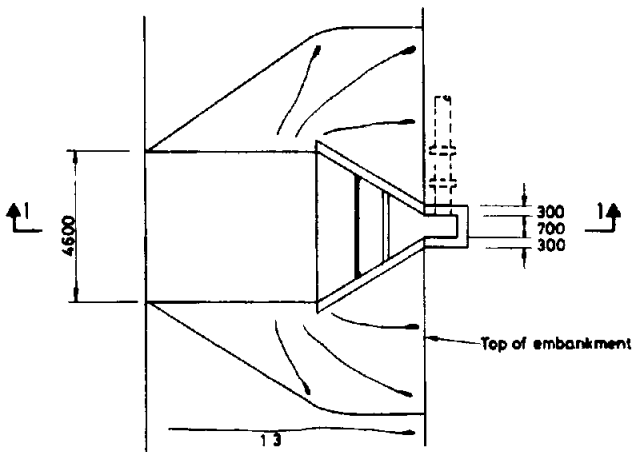
Figure 6 Flexible Operation Pipework

Wave Protection

Inadequate protection of embankments at the water-edge appears to be one of the most unsatisfactory aspects of waste stabilization pond design²⁴. Concrete paving slabs are in common use but in some cases subsidence and lifting have arisen probably due to the soil type. A stone layer can prove a successful measure provided it is formed of stones of sufficient size and placed in a layer of adequate depth retained if necessary. Bank protection should extend at least 300 mm above and below the maximum and minimum water levels.

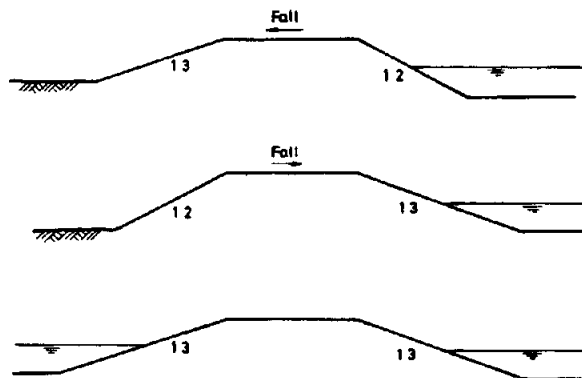
Other methods providing protection are principally applied to seal the pond base and include bentonite stabilized soil, asphalt and plastics linings.

Wave erosion is minimized by using pond areas less than 12 ha²². Various formulae have been proposed to predict wave height most resulting in a minimum value of about 0.70 m. Oswald²³ proposed the following formula for freeboard calculation

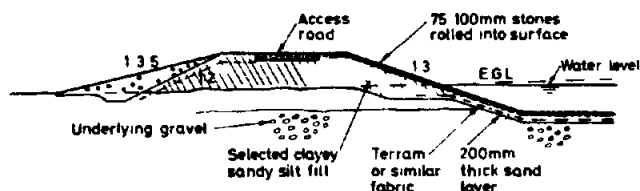


SECTION 1-1

Figure 7 Pond Outlet



TYPICAL SECTIONS



SPECIAL SPECIFIC DESIGN

Figure 8 Embankments

$$f = (\log A)^{1/2}$$

where f = freeboard (m)

A = area (m²)

Mixing

Wind-induced mixing is essential to satisfactory pond operation as it increases the natural surface reaeration, reduces short circuiting (provided the wind direction is not from inlet to outlet) and maintains an

improved algal population and distribution of pollutants. The degree of mixing is largely dependent on the fetch or length of open water and the surrounding obstructions such as vegetation and buildings. Ideally a wind rose should be obtained for the site so that pond orientation is best suited for wind induced mixing. A clear distance of about 100 metres around the pond should eliminate the effects of surrounding obstructions.

A fetch of 100 to 200 metres is reported as being sufficient for adequate mixing^{8, 12, 22}. Where wind speeds do not commonly exceed about 20 km/hour wind induced mixing can not be relied upon and artificial mixing should be considered.

Thermal mixing on a diurnal basis usually occurs due to stratification and subsequent turn-over resulting from water density changes. Stratification is not desirable and therefore this form of mixing is of little use.

Where insufficient natural mixing occurs artificial means may be used such as surface aerators (which moves the design into the realm of aerated lagoons), simple 'bubble guns', rotary brushes and horizontal screw pumps.

Groundwater Level

A knowledge of the groundwater level is important to ensure the pond base is above the water table so that construction work is made very much easier and groundwater infiltration into the ponds is avoided.

Flood Protection

Often pond sites are located in a natural topographic low area to permit gravity sewage flow to the works. Such sites may be prone to flooding during storm periods and therefore should be adequately protected from flood water erosion.

During flood conditions it is also desirable to provide some sort of emergency overflow from the ponds so that the embankments are protected. Emergency overflows may take the form of simple spillways.

Sludge Accumulation and Removal

The accumulation of sludge in a pond depends on the type of pond and the grit and settleable solids content of the raw sewage. Accumulation rates in India and the USA are discussed in detail by Arceivala *et al.*¹². The nett accumulation of sludge deposits *per capita* decreases with time due to the effects of anaerobic digestion and compaction. Typical accumulation rates in India range from 0.08 to 0.3 m³ per head per annum¹² and the resulting desludging frequencies normally range from about 5 to 12 years depending on the depth of pond considered and the climate.

Sludge removal should be effected once the minimum desirable water depth for the type of pond has been reached for example the minimum liquid depth for a facultative pond is generally one metre. Sludge will be in a dried form after withdrawing the particular pond from operation and draining down and then may be removed by a mechanical shovel or by hand barrow. Care must be taken to avoid damaging the pond base construction. The typical moisture content of accumulated sludge ranges from an initial value 80 to 94 per cent which will reduce with the period of drying. The reduction in moisture content with time has been examined by Pescod²⁷ and a graph was produced relating the total solids content to cumulative solar radiation. The final disposal of sludge may be to agricultural land (landfill) or tip depending on the circumstances and sludge constituents.

Pond Lining

Where it is desired to avoid excessive percolation into the ground in areas of limestone or gravel formations for example, bottom sealing in some form will be necessary. Bottom sealing may be achieved simply by a layer of clayey soil or bentonite if this is available locally at reasonable cost.

Other methods include soil stabilization with cement, sprayed or rolled asphalt and plastics lining materials. Plastics lining materials are usually placed on a prepared sand layer and it is advisable to protect the membrane with a similar sand layer on top. The cost of lining usually precludes its application except where the potential pollution of a water source for example justifies its use.

Future Expansion

As indicated above the selection of a site for a pond system should take into account the land requirements for future expansion. Where land area is not available to expand the system can be modified to include aerated lagoons by deepening some ponds. The addition of anaerobic lagoons for the primary treatment of stronger wastes has been shown to reduce the total land area required by a factor of up to two times in theory²⁸ but this is not always realized in practice.

Other methods of upgrading pond systems through process modification have been suggested²⁹ including

- organic load adjustment
- recirculation
- feed and withdrawal facilities
- mixing
- aeration

CONCLUDING COMMENTS

Despite the large amount of research and literature on the subject of waste stabilization ponds the various design methods can result in widely differing results. No one method appears to cover the whole story for example for facultative pond design the McGarry and Pescod method is based on ambient temperature only, whereas a check on the available solar radiation level is advisable.

Ponds are the first choice of treatment method in hot climates as they are simple to construct, cheap to run and can produce an effluent having minimal risk to the environment.

This paper represents the views of the author and not necessarily those of Howard Humphreys.

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DISCUSSION

Dr R J Martin (Lecturer, University of Birmingham)

Q 1 Is there a danger of waste stabilization pond design being unnecessarily complicated and theoretical particularly in view of the poor level of maintenance often encountered?

A 1 This question is best answered in three sections

- Although the methods presented may appear complicated, in fact they are based on very simple concepts. No one method is completely satisfactory and it is therefore important to take into account the factors included in a number of approaches. In general terms the more theory used in a design the more likely a successful design will result
- Most of Howard Humphreys work is for international funding agencies who require full justification for a proposed scheme
- Adequate maintenance is essential to any scheme of sanitation and wherever possible training schemes are offered as part of the project implementation

Q 2 What sort of population may be served by waste stabilization ponds?

A 2 Waste stabilization ponds are generally used for small rural communities of a few thousand. However the largest pond system to my knowledge is in Auckland, New Zealand serving an equivalent population of over 500 000 and occupying 525 ha

Mr J P Arthur (Research Student, University of Birmingham)

Q 3 In view of the many factors which must be considered in the design of waste stabilization ponds, is it not desirable to construct pilot plants to observe their behaviour before full scale ponds are constructed?

A 3 Pilot scale studies have been undertaken to examine the effects on performance of different configurations^{2, 34}. However scaling up the performance criteria may not prove to be reliable in view of the changed influence of factors such as wind action

Q 4 What constitutes the limit to loading of facultative ponds and could there be some reduction in retention periods and some increase in the loading rates suggested in the paper?

A 4 The maximum areal loading rates for facultative ponds before the onset of anaerobic conditions was related to mean monthly ambient temperature by McGarry and Pescod⁵. A factor of safety of 1.5 has been suggested by Mara⁶ for use in design

For a given surface area the retention period is a function of depth only, normal depths of facultative ponds range between 1.0 and 1.5 m depending on the sludge storage requirements and climatic conditions

Q 5 How do the costs of waste stabilization ponds compare with other treatment systems?

A 5 Any cost comparison of treatment methods must include both the capital costs, recurrent costs and operating costs. Ponds are simple to construct but where the majority of the pond volume is below ground level the rate for excavation will be the predominant cost factor

A recent comparison between a waste stabilization scheme and an oxidation ditch scheme revealed the following capital cost comparison for a three stage construction

	Stage I	Stage II	Stage III
Oxidation Ditch	10 000	8 600	9 900
Waste Stabilization Pond	10 300	8 900	13 600

The Stage III of the pond scheme involved a conversion to an aerated lagoon system

From the above capital cost comparison one might be tempted to adopt the oxidation ditch scheme but when the operating costs and recurrent costs are taken into consideration the pond scheme has a discounted cost of 29 600 compared with 32 500 for the ditch scheme and would therefore be selected on the basis of *least discounted cost*

Further factors affecting the choice of process include

—the removal efficiency of pathogenic bacteria and viruses helminths etc

—the level of local operative skill

—the proportion of foreign currency required

Gloyna¹³ gives the following cost comparison of construction costs

Construction Costs per Head (US\$)			
	Waste Stabilization Ponds	Activated Sludge	Percolating Filters
1 000	9-29	45-98	50-110
10 000	4-13	23-52	23- 51
100 000	1.75- 6	12-27	11- 23

Mr W M Waterson (A H S Waters & Partners)

Q 6 Is ammonia removal as complete as would be expected from a long retention system?

A 6 Some researchers have reported ammonia removal efficiencies of between 56 and 84 per cent whereas an extensive investigation in Sunnyvale California¹⁵ found that no appreciable nitrification occurred with or without surface aeration. No inhibitory substances were detected in the waste and although the nitrifying bacteria *Nitrosomonas* and *Nitrobacter* were present in the pond the lack of aerobic surfaces for attachment and growth was suggested as being the cause of the lack of nitrifying activity

Q 7 Is there experience of removing sludge from a pond without emptying i.e. using a mud-cat type vacuum cleaner?

A 7 I do not know of any successful desludging methods used without first emptying the pond

Q 8 Is microstraining an effective method of removing the type of algae that predominate in pond systems?

A 8 Probably no other system has been tried so often and failed so consistently as has microstraining in the attempt to upgrade pond effluent¹³⁶. The commonly occurring species of algae such as *Chlorella* and *Scenedesmus* are often smaller than the smallest commercially available microstrainer aperture of 23 μ

Mr C Macrorie (MSc Student Water Resources Technology University of Birmingham)

Q 9 Are ponds advised for combined sewer systems i.e. foul sewage and stormwater and if so how is the problem of surge dealt with when effluent guarantees have to be given?

A 9 Ponds are probably more able than most treatment processes to accommodate storm water arising from a combined system. However in some instances a storm water overflow prior to the ponds may be advantageous although generally a pond system should be designed to accommodate storm water flows. Overflow weirs should be included in each pond to prevent damage to the embankments. The reduced retention times during storm flows are offset by the additional dilution available

PAPER 14

DESIGN AND CONSTRUCTION OF POND SYSTEMS

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DESIGN AND CONSTRUCTION DETAILS OF WASTE STABILIZATION PONDS

1. General

This paper is concerned with the design of pond facilities, their interaction with the process design and the details of construction. Included in the facilities design are such factors as the net evaporation, precipitation and percolation and mixing. Both these factors have a direct influence on the process design.

A retention time based on the incoming design flow may be very different from the actual due to the combined effects of evaporation, precipitation and percolation. There have been cases where ponds have failed to fill due to over-design resulting from not accounting for these factors. Such problems can be aggravated by initial flows being very much less than expected due to over-ambitious projections of water usage, rate of house connections to the system or similar factors built-in to a design. The policy in New Zealand is never to allow designs to cater for population increases of more than 25 percent, EDEN (1978).

The degree of mixing within a pond is important for three reasons:

- o firstly, the photosynthetic activity of the mixed algal population is greatly increased when adequate mixing is provided. Mixing may be wind induced or mechanical. High fences or other wind breaks too near to ponds cause poor performance.
- o secondly, the bacterial breakdown of the waste is enhanced by maximum contact with the substrate ensured by adequate mixing.
- o thirdly, the degree of mixing is important as this affects performance. THIRUMURTHI (1969) developed a design approach accounting for the degree of incomplete mixing using the dimensionless dispersion number. TIMPANY et al (1971) emphasized the importance of a relatively square shape to achieve good mixing. MURPHY and WILSON (1974) report that the degree of mixing is more dependent upon the pond shape than the artificial

aeration input over a given range for an aerated lagoon IWUGO and WINNICKI (1978) undertook tracer studies of the mixing characteristics and found that the mean residence time of a rectangular pond was significantly greater than the volumetric retention

The various factors included in the facilities design are listed by LUMBERS (1979) as follows

- | | |
|-----------------|-----------------------------------|
| o precipitation | o inflow and outflow structures |
| o infiltration | o embankments |
| o evaporation | o wave protection |
| o orientation | o mixing |
| o shape | o groundwater level |
| o position | o sludge accumulation and removal |
| o configuration | o lining/sealing |
| | o future expansion |

2 Precipitation, Infiltration and Evaporation

The combined effects of precipitation, evaporation and infiltration must be accounted for in design so that complete pond filling is ensured and the maximum and minimum retention periods over the year estimated. Exact estimates of these parameters and, of course, also the actual variations in the flow of waste water itself, are difficult. Indeed a precise design would call for consideration of long-term rainfall and evaporation statistics in order that specified return-periods may be used in design.

2.1 Precipitation

Local rainfall records are not always available, however reasonable data for most locations may be obtained from the UK Meteorological Office or alternatively the US Weather Bureau. A rough guide to world-wide precipitation may be found in the Times Atlas of the World, or the Handbook of Applied Hydraulics, DAVIS (1965).

2.2 Infiltration

The rate of percolation into the subsoil depends on the type of soil and the depth of the groundwater table. In addition the infiltration rate declines with time as the soil surface at the pond bottom becomes progressively clogged with fine particulate matter and organic growth. In fine sand for example, an initial infiltration rate of 15 cm/d may gradually decline to 1 cm/d. The frequency of desludging and the accompanying scarifying of the pond bed will have an influence on the overall cumulative loss.

OSWALD (1975) quotes typical losses in California as being approximately $1.27 \text{ cm/d} \pm 0.63$ due to the combination of infiltration and evaporation. Highest rates were observed on coarse shale at 6.4 cm/d, in fine sand at 3.3 cm/d and in heavy silty clay at 1.0 cm/d.

Oswald also summarises the general aspects of an in-site percolation test reported by McKINNEY (1970) shown here in Figure 1.

2.3 Evaporation

Evaporation data are often difficult to obtain. A knowledge of evaporation loss over the year is important as it affects:-

- o the net retention
- o the maintenance of water depth
- o the concentration of pollutants in the pond contents e.g. nitrates and phosphates, metals

Evaporation pans may be used and the results corrected to estimate lake evaporation.

3 Embankments

There are many factors which affect the design of embankments

- o height
- o soil stability
- o access requirements
- o minimisation of earthworks
- o wave protection
- o cross-section

3.1 Height

Embankment height is a function of the

- o maximum design water depth
- o freeboard
- o sludge accumulation provision

The maximum water depth may be dictated by a storm water overflow weir above the normal water level.

The freeboard requirement is determined by the predicted wave height in order to prevent over-topping and consequential embankment damage. A minimum freeboard of 0.7 m is recommended and 1.0 m has been used frequently.

The provision for sludge accumulation is typically 200-300 mm and this should be added to the design normal operating water depth.

3.2 Soil Stability

Generally embankment designs are very conservative in the side slopes and crest width. However it is advisable to establish the characteristics of the material to be used regarding its slope stability, attainable compaction, subsidence potential, etc. The guidance of a soil mechanics engineer is

preferable particularly where embankment stabilising techniques are likely to be needed or where seepage problems are possible. In difficult soils stabilisation techniques such as, "stabilized earth", "reinforced earth" or the use of fabric membranes may be appropriate.

3.3 Access Requirements

For vehicle access a minimum width crest width of about 3 m is necessary, 4 m is desirable and 5 m preferable for easy access and equipment handling.

3.4 Minimization of Earthworks

It is highly desirable that the excavation of material is approximately matched to that required for constructing embankments. Removing excess material from a site for disposal elsewhere could be extremely costly.

3.5 Wave Protection

Bank slopes are typically 1 in 3 to 1 in 5 and this encourages dissipation of wave energy. It is important to protect the embankments from erosion by wave action. Protection should extend at least 0.3 m above and below the maximum and minimum water levels, taking into account wave action. Pronounced wave action may occur in down-wind corners and here the bank protection could be required to extend full height.

The most common protection methods are stone pitching with 75-150 mm stones rolled into the surface or precast concrete slabs. Concrete slabs cast in-situ have been used but can be expensive. Asbestos cement sheets have been found to crack or move. Other methods, which are simply extensions of bottom sealing, include asphalt, stabilized earth, and plastic or butyl rubber sheeting. Slabs must be keyed in to prevent slipping and this is difficult to achieve adequately with small slabs. Weed growth in joints is a problem and must be controlled.

3.6 Embankment Cross-Section

A variety of shapes are possible according to particular needs, Figure 2 gives some examples. A cross-fall on the crest is needed to provide surface drainage. LUMBERS (1979) gives an example of a specific design of an embankment using a fabric layer for a pond overlying gravel and embankments of clayey sandy silt.

4 Embankment Construction

Embankments should be constructed on existing ground stripped of trees, shrubs and vegetation. The ground should be prepared to accept embankment fill by watering and harrowing.

The embankments should be constructed using selected cohesive fill material from the bulk excavations and should be free from organic matter and large stones of 150 mm or more in diameter.

The selected fill material should be placed on successive layers about 200 mm thick, watered by bowser and then harrowed to mix the fill and water together. Watering and harrowing should be sufficient to raise the moisture content to within 2 percent of the optimum for the compaction of the particular material. Prior to the application of further layers, each layer should be compacted by roller to obtain an in situ density of 95 percent of the maximum attainable in the standard compaction test Test 12, BS 1377 : 1975. The in situ density of representative samples should be measured in accordance with Test 15(A) BS 1377 : 1975 or by other approved methods. The surface of the embankment during construction should be watered to prevent drying and keep the same moisture content as at the time of placing.

At the beginning of constructing the embankments trials should be made in order to determine the best method of operation with respect to placing, watering, harrowing and compacting (number of roller passes required) in order to achieve the specified density of the compacted material.

Where pipes are constructed through embankments particular care should be taken to ensure thorough compaction on back-filling the trench or around the pipe as the embankment rises.

5 Sealing

The requirements for sealing the base of ponds should be considered in the light of the potential groundwater pollution and the importance of percolation losses to the operation and performance of the system. A knowledge is required of the subsoil, the groundwater level and its fluctuations and the locally available sealing materials such as clay. In areas where the ground water level can rise rapidly above the level of the pond base, sealed ponds must be filled quickly and maintained full.

The entry of saline water into ponds can cause the reduction of sulphates to sulphides if more than 5% salt is present. Saltwater can infiltrate into pond systems in coastal areas or through the sewer system and justify bottom sealing.

5.1 Clay Layer Sealing

The cost of synthetic sealing materials is prohibitive in many countries and clayey soil material may be used where available. Such material should be laid in layers of 200 mm up to a thickness of 1000 mm as necessary. The pond base may be rendered more durable by rolling on stones of a similar size used for protecting embankments, but this is not essential. Where exposed gravel underlies the pond then it may be necessary to use a fabric membrane on top of the gravel cover by a sand layer prior to placing sealing material to prevent particle migration. The fabric should be a synthetic permeable membrane such as "Terram" and should be man-handleable and stored out of direct sunlight.

Bentonite is a form of clay which expands when wet and fills the voids in a coarse soil. Where it is locally available it may be economically attractive used as a 25-50 mm skin or mixed with soil in a 5-10% mix.

5.2 Soil Stabilization

Soil stabilization or stabilized earth can be one of the most economic alternatives where sealing or lining is essential. A cement mixture with soil of about 5-10% is mixed to form a surface layer of 150-300 mm thick. The layer is then rolled and the natural ground moisture sets the mixture. The actual proportion of cement required should be determined by site trials and in some cases it may be necessary to import suitable soil for the mixture. The resulting lining is permanent and facilitates sludge removal without damage.

5.3 Asphalt Sealing

Asphalt may be applied either as a layer containing aggregate (perhaps from road repairs) or as a spray onto fine grained soils. Spray applications are much less expensive but do not offer the strength of a conventional paving layer. In hot areas problems with softening and consequent damage are to be expected.

5.4 Synthetic Linings

These linings include polyethylene, PVC, fibre-glass mats and butyl rubber. All these are expensive and should only be considered where efficient sealing is essential and the soil conditions rule out other choices of sealing methods. Butyl rubber has been successfully used for a variety of wastes.

6 Inlets, Outlets and Transfers

6.1 Inlets

For small ponds of say less than 3 ha an inlet pipe about 30 m long on piers is commonly used. The end of the pipe is often fitted with a downward pointing bend directed onto a concrete plinth in order to disperse the settled sludge as evenly as possible. The design of outlet bends has not been thoroughly evaluated and a simple horizontal or slightly inclined outlet would probably be just as adequate. The cost of pipe bends can be very significant and from a process point of view unjustified.

Larger ponds tend to use multiple inlet arrangements with pipes laid on the pond base.

Inlet pipes should be possible to drain by gravity and would benefit from a slight gradient of 1 in 200-300. The pipes should be designed to achieve a self-cleansing velocity of at least 0.6 m/s and preferably 1.0 m/s at least once or twice a day.

The pipe material should be selected to be resistant against internal and external corrosion, with suitable protective surrounds as necessary.

6.2 Outlets

Outlets may either be a weir chamber constructed in the embankment or a manhole within the pond itself. An adjustable weir is preferable in order to give operational flexibility and a low level drain-down facility is required for all ponds in which desludging is likely to be required.

Flexibility in the level of outlet draw-off is desirable as the level with minimum algal content may be selected.

Scum boards are essential to protect outlet weirs and should be about 500 mm deep.

Pond outlets should be sited at the upwind side of a pond if possible to help avoid short-circuiting and scum accumulation at the outlet.

The use of penstock should be avoided if possible, especially screw penstocks.

6.3 Transfers

At their most simple transfers from one pond to another are open ended pipes laid through the dividing embankment. Alternatively channels with the facility to place stop-logs may be used which have the advantage of being able to control water level in the previous pond and also shut-off flow for maintenance. Scum boards are desirable to prevent carry forward of floating material.

7 Shape and Configuration

Rectangular ponds with rounded corners provide the best mixing and reduce short circuiting. Ponds over 12 ha are not preferred due to the excessive wave action likely to be encountered.

Two separate streams of flow are desirable for maintenance and interconnections provide operational flexibility.

8 Maintenance

"Ponds are an economic means of providing sewage treatment for small and medium sized communities. The requirements for good workmanship in construction must not be underestimated. They should be treated as permanent units which required regular, intelligent maintenance"

Ministry of Works & Development, Wellington, New Zealand

Ponds should be fitted as soon as possible to prevent weed growth and water depths of at least 1.0 m to avoid tall rooted plants establishing themselves.

Tree and shrub growth in the vicinity should be kept low to avoid a reduction in wind-induced mixing.

Most ponds should be visited at least once a week and the following activities undertaken:

- o remove floating debris
- o keep structures (inlets, outlets and transfers) clean and free from growth
- o remove debris washed-up on banks
- o repair bank erosion
- o control vermin (rat) infestation
- o check and repair fences
- o clear banks of excessive weed growth
- o maintain any buildings
- o grease mechanical equipment
- o paint machinery as necessary
- o keep effluent channel clear

It has been shown that keeping the banks and shallow areas free from weed growth reduces the potential mosquito breeding problem to acceptable levels if not completely, LEODOLFF, C.J (1963)

9. Siting

As a general rule ponds should not be sited nearer than 300 m from housing areas and where anaerobic ponds are used considerably more than this depending on the sensitivity of the area. Ideally ponds should be downwind of populated areas to reduce smell nuisance which is inevitable occasionally.

Flat exposed sites are preferable to obtain maximum sunlight and wind-mixing.

10. Miscellaneous

10.1 Fencing

Pond sites should be securely fenced to prevent access by children or livestock.

10.2 Warning Notices

Warning notices must be erected at regular distances stressing the danger of the contents for any kind of domestic or uncontrolled irrigation.

10.3 Flood Protection

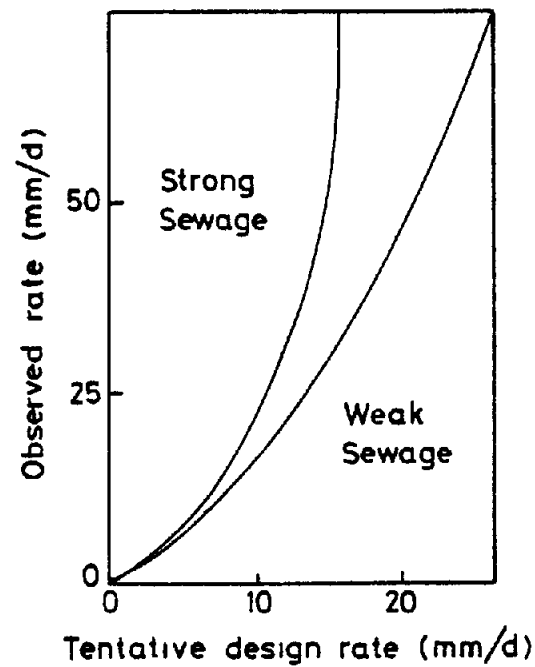
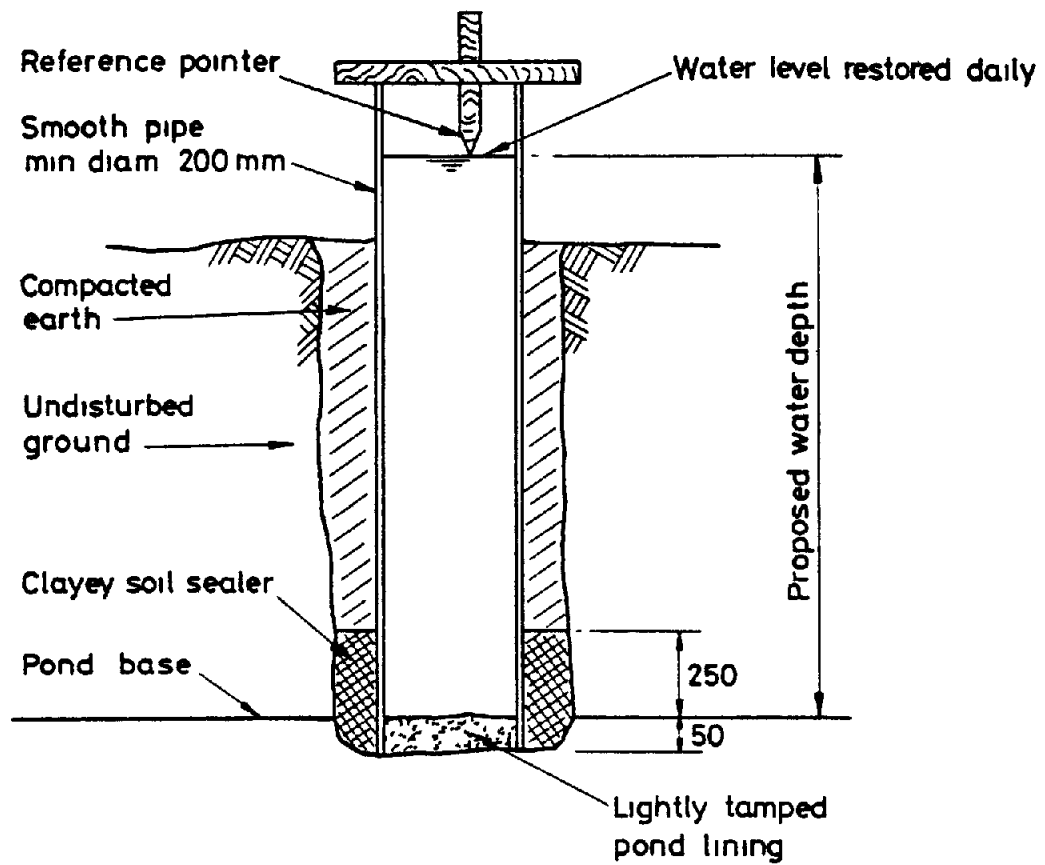
Where ponds are sited in low lying areas or wadis, care must be taken to protect against potential flood damage.

10.4 Flow Measurement

Some form of flow measurement is highly desirable to assist in operational management and forward planning. 'V' or rectangular notch weirs provide a simple means or alternatively flumes may be used but the liners are expensive and prone to damage. Inlet and outlet flows should be measured to assess losses.

10.5 Staff Facilities

A building should be provided to store tools and equipment, keep plant records, carry out routine tests if required and provide a washing facility for staff



IN-SITU PERCOLATION TEST & DESIGN CURVE

FIGURE 1

after McKINNEY (1970)

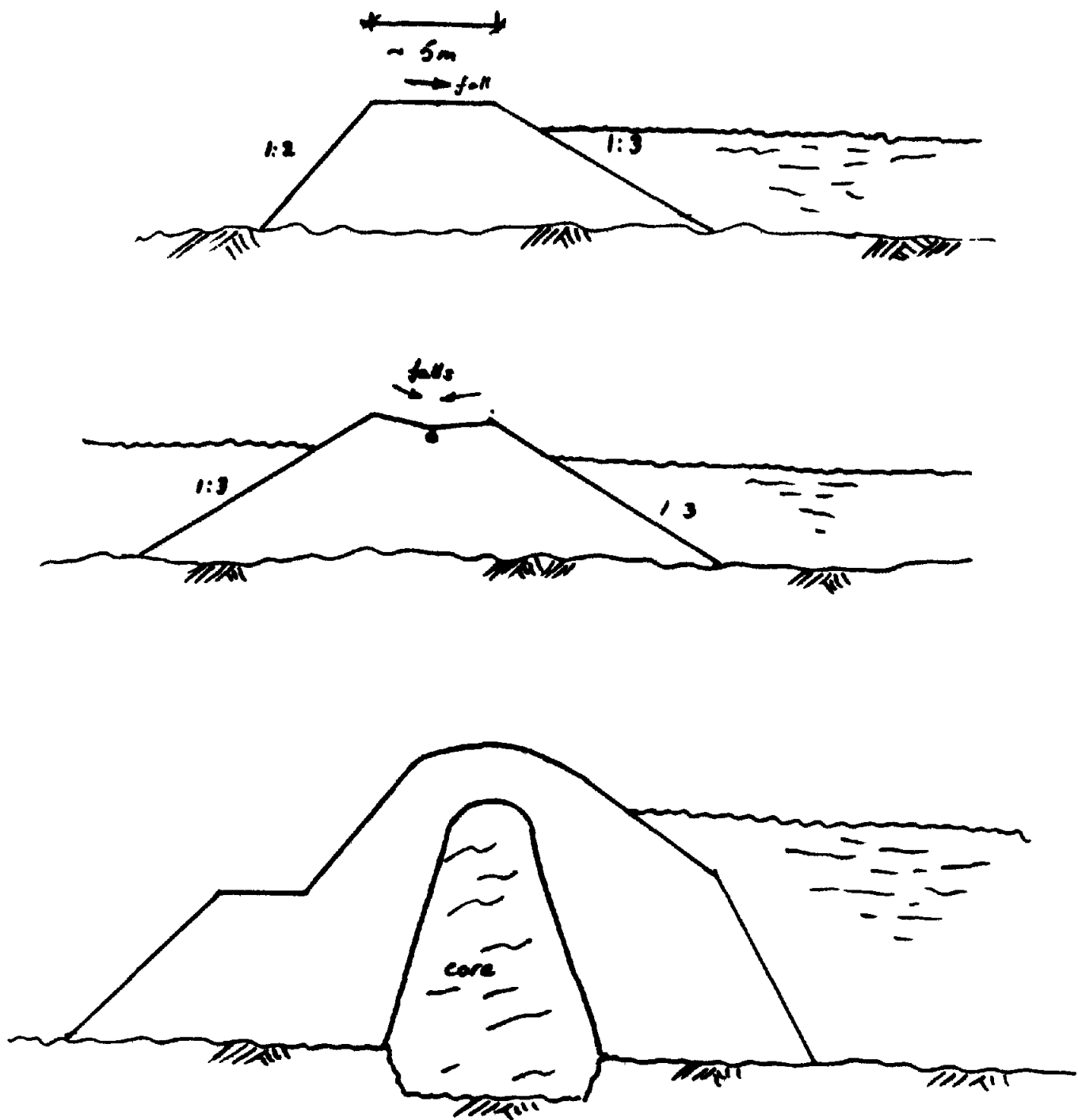


Figure 2
Examples of Embankment Cross-sections

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PAPER 15

FULLY AEROBIC (MATURATION) PONDS

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FULLY AEROBIC (MATURATION) PONDS

These ponds are either used as the final stage in a series of anaerobic, facultative, aerobic ponds or to improve the quality of the effluent from a conventional wastewater treatment works. They are used

- (i) for further lowering the BOD of the effluent;
- (ii) for reducing the bacterial count in the effluent;
- (iii) for further recovering suspended solids from the effluent of a conventional treatment unit;
- (iv) for oxidising ammonia to nitrate (nitrification);
- (v) for recovering the nutrients, phosphates and nitrates from the effluent.

When employed to further reduce the BOD in the effluent maturation ponds continue the work of the facultative ponds. The first-order, complete-mix equation

$$\frac{L_e}{L_i} = \frac{1}{1 + k \cdot t}$$

can be used to calculate the BOD reduction to be achieved (1.2.3.4.5.). As maturation ponds are usually built in series this equation can be modified to

$$\frac{L_e}{L_i} = \left(\frac{1}{1 + k \cdot \frac{t}{n}} \right)^n$$

where L_e = BOD of effluent from the maturation pond (mg/l)
 L_i = BOD of influent to the maturation pond (mg/l)
 k = reaction coefficient (d^{-1})
 n = total number of ponds
 t = total nominal retention time in the ponds (d_1)

The results for BOD removal can be complicated by the presence of algae in the effluent. The results calculated are only for 'soluble' BOD. The presence of algae can increase this result greatly.

Removal of bacteria

The lowering of the bacterial count in the wastewater is achieved as a result of

- (i) the bactericidal effect of the U.V. light in the sun's radiation;

- (ii) the presence of natural predators in the water (particularly protozoa);
- (iii) the result of holding intestinal bacteria in a foreign and unfavourable environment.

The die-off of intestinal bacteria is also supposed to follow first-order kinetics (4.5.) and hence

$$\frac{N_e}{N_i} = \frac{1}{1 + k_b \cdot t}$$

where N_e = bacterial count in the effluent from the ponds
 N_i = bacterial count in the influent to the ponds
 k_b = the first-order rate constant for bacterial die-off

Maturation ponds can be very effective at reducing the numbers of intestinal bacteria. The rate of removal is greatly affected by an increase in temperature. In a series of four ponds with a total nominal retention period of 86 hours in the limited temperature of an English summer the E. coli count was reduced from $2 \times 10^4/100$ ml to 800/100 ml and the faecal streptococci from 8000/100 ml to 10/100 ml (6). This reduction would be greatly increased in a hotter climate.

Reduction of suspended solids

Maturation ponds can be used for reducing the concentration of suspended solids in the effluent from a conventional sewage treatment works. If employed for this purpose the nominal hydraulic retention time will be limited to a few hours - perhaps 24 at the outside - to prevent the development of algae which would increase rather than reduce the suspended solids in the water.

Oxidation of ammonia

Ammonia in an effluent can be dangerous when discharged to surface water. Ammonia is very toxic to fish-life, even in very low concentrations. For this reason ammonia in wastewater is normally oxidised to nitrate during treatment. This is known as nitrification and is brought about by bacterial action. In most conventional treatment works, and in facultative ponds, in warm climates this oxidation should be complete. However, if the works are overloaded or if the temperature is low there might still be an appreciable concentration of ammonia in the effluent discharged. If this is so then maturation ponds can be employed to increase the total nominal

retention period, to allow time for the nitrifying bacteria to develop and operate and hence to achieve the required nitrification.

Removal of nutrients

Both phosphates and nitrates are removed from the water in maturation by the development of algae. These nutrient salts are used in the creation of new algae cells. This is called an anabolic process. However, most of the nitrate removed during maturation pond treatment is the result of 'denitrification', and most of the phosphate is removed by precipitation. Denitrification occurs when bacteria in the water employ the oxygen contained in the nitrate ion (NO_3^-) for their metabolic (living, feeding, growing) processes. Nitrogen gas is then given off to the atmosphere. Precipitation of the phosphate occurs as the hydroxyapatite (7) as the pH in the water increases daily under the influence of the photosynthetic action of algae. In addition to these methods there can be a drastic reduction in the phosphate and nitrate content of pond water if water-hyacinth (*Eichornia crassipes*) is grown in the maturation pond.

Mosquitoes

The development of mosquitoes can be a nuisance and danger in maturation ponds. This is particularly so if malaria is endemic in the region. In these situations extreme care must be taken that the risk of breeding mosquitoes in the ponds is minimised. This risk can be reduced by

- (i) placing concrete slabs around the water's edge (1.0m above and below the water-line) to prevent the growth of weeds and reeds in which mosquito larvae can exist;
- (ii) using synthetic membranes, or other sealing devices, above and below the water-line around the pond;
- (iii) continually cutting and removing weeds and reeds that grow;
- (iv) stocking the ponds with fish that will eat the larvae and the weeds;
- (v) spraying with herbicides (not generally recommended).

Several species of fish can be employed to prevent the development of mosquito larvae in ponds. Perhaps the most commonly used is the species *Gambusia affinis* (the mosquito fish) which eats the larvae. Several types of carp have also been employed but the best species to use can often be the *Tilapia* which eats the weeds and has been successfully used for keeping irrigation canals and ditches in parts of the southern United States free

from weeds (8.9.). *Tillapia* is also a good source of food protein and has been known to produce a nine-fold increase in weight in only three months (8.).

Depth, number and size

As maturation ponds should be fully aerobic and as there is no intense oxygen demand, comparable with that of a facultative pond, the ponds could be constructed with a greater depth. However, because it is necessary to make use of the Ultra-Violet light in the solar radiation as a disinfecting medium, it is usual to retain the same depth of 1.5m which is common for facultative ponds.

Unlike facultative ponds it is usual to build a number of maturation ponds in series. The greater the number of ponds in series the closer the flow pattern approximates to the efficient plug-flow regime, and the better the results achieved. It is, however, more costly to build a number of small ponds rather than one or two large ponds so a compromise of three or four ponds in series is normally selected. This results in efficient operation at not-too-great a cost.

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PAPER 16

ALGAE, HIGH-RATE TREATMENT, ALGAE REMOVAL AND HARVESTING

By

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Algae, High-rate Treatment, Algae Removal and Harvesting

Algae are nearly invariably present in facultative and aerobic stabilization ponds. They add large quantities of dissolved-oxygen to the water during the day as they respond photosynthetically to sunlight, but at night they withdraw oxygen from the water. They also encourage thermal stratification.

In stabilization ponds the development of algae is the reverse of the normal biological breakdown - or purification - of complex organic materials.

With the normal aerobic biological stabilization of large, complex, unstable organic molecules of pollution (the process of purification) these organic materials are broken down into simple, stable inorganic compounds such as carbon dioxide, water, sulphates, phosphates, etc. The development of algae is the reverse of this process in that the simple, stable inorganic compounds such as nitrates, phosphates and carbon dioxide are used to form the complex organic materials that make up algal cells.

In the effluent from a stabilization pond the algae represent an additional and unwanted source of BOD. As the algae die the cells will be decomposed by other aerobic microorganisms. This will create an additional oxygen demand that is difficult to quantify. As a result algae in the effluent from a stabilization pond are a liability to the receiving stream.

As algae in the effluents do represent a major disadvantage various techniques have been considered for removing them. These techniques include several types of sand filtration (rapid, slow, intermittent), continuous belt filtration, microscreening, chemical coagulation followed by dissolved-air flotation, and the application of chlorine.

Rapid sand filtration is only effective for algae removal if the water is pre-treated either with alum or with magnesium hydroxide (1.2.). Slow sand filters quickly block, microscreening has proved to be largely ineffectual, continuous belt filtration is relatively costly and complex (3), the use of chlorine can be dangerous as well as costly (4), chemical coagulation followed by dissolved-air flotation is effective but complex and costly. Fish are only partially effective.

On the whole, because of the cost and complexity of the available techniques for algae removal, it is normally considered to be not worth the effort. It is normally concluded that algae in the effluent from stabilization ponds are an evil which must be lived with.

There are however a few simple techniques for the removal of algae which on occasions can be effective. The first of these is to use a variable level discharge so that if algae are present in the final pond at only one definite level then the effluent can be withdrawn from a different level. Then, in some situations, it is possible to retain the effluent in the final pond until the algal bloom dies off. This technique can be useful if there is an appreciable period of cold weather during the year and if there is excess capacity in the final pond.

The other, most effective, means of reducing the concentration of algae in an effluent is to allow a growth of water hyacinth (*Eichhornia crassipes*) in the last pond of the treatment series (5,6). Water hyacinth is the fastest growing plant known and will quickly cover the whole pond area if the water temperature is above 21°C, although it is sensitive to high salinity water. Reduction of the algae concentrations of over 90% have been recorded. However, *Eichhornia crassipes* is an environmental hazard in many parts of the world and should not be introduced into areas in which it does not already exist, but in those countries in which the water hyacinth is already present it can conveniently be used to remove algae.

High-rate ponds and the culture of algae

In several parts of the world (Florida, Thailand, Israel, The Philippines, Singapore) investigations have been carried out into the possibility, not of discouraging the growth of algae but of encouraging their growth so that they might be eventually separated from the effluent, dried and used as a source of protein for cattle, poultry or fish food.

This has been done by using shallow ponds (200-600mm) with limited retention periods (1-3 days). In this way the mass of algae produced has been greatly increased up to the range of 20 to 40 g./m².d (7), although the effluent has not necessarily been of a good quality. Trouble has been encountered due to the presence of predators such as the cladocera *Moina* which have the ability of clearing a pond completely of algae in a very short time. One of the problems currently is the rapid rate of change of the predominant species of algae which can lead to harvesting difficulties. In Singapore investigations are now in progress (1981) into the possibility of producing a single, constant dominant strain of algae, such as *Microcystis*, which will be readily harvested.

Separation of the produced algae from the effluent (harvesting) is readily achieved employing the coagulant alum, together with acid to reduce the pH to between 6.0 and 6.5, followed by dissolved-air flotation. Other coagulants such as magnesium hydroxide or the natural flocculant chitosan (9) have been successfully used.

Dissolved-air flotation together with a suitable coagulant will result in the production of an algal sludge containing 7% to 9% solids (7). This is then best treated by centrifuging to produce 15% solids and then by drum drying to produce 90% solids. Alternatively the 7% to 9% float can be solar dried by spreading on porous cloth and covering with a plastic sheet which permits the circulation of air while allowing the temperature of the mass to rise. Unfortunately solar drying does not rupture the tough walls of the algal cells as effectively as drum drying. Cell rupture allows the proteinaceous contents to escape and become available for animal nutrition. If the cells are not ruptured then monogastric animals are not able to utilize efficiently the algae as a food supply.

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PAPER 17

DESIGN EXERCISE

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Design Exercise

In the development of a new town (A) a site of approximately 26 hectares has been set aside for the treatment of all the wastewaters produced from the residential and commercial areas and from the industrial complex. This site is of a rectangular shape, being about 867 m in length by about 300 m, and is immediately adjacent to the northern boundary of the middle density housing estate. The contours of the site are shown in figure 1.

Two distinct streams of wastewater will be received at this site. One stream will be from the residential and commercial areas and the other from the industrial development. Details of flows and of water qualities are given in table 1.

The effluent from the wastewater treatment works will be discharged into the local river. Because of the relatively low flow in the river, and as a result of the demands on it for drinking water, irrigation and industrial abstraction the conditions of discharge are strict (See table 2).

The town water supply will have a relatively high salinity of 600 mg/l of which 200 mg/l will be sulphate. Amoebic dysentery, human round-worm (*Ascaris*), malaria and schistosomiasis are endemic amongst the local population.

There will be electrical power available at the site but it is expected that this will not be completely reliable and will be subject to frequent breakdowns which might last for up to four hours. Initially there will be little skilled labour available locally.

Produce a design for a suitable wastewater treatment works which can consistently achieve the standard of effluent required. This will include all necessary details of flow, retention period, tank or pond sizes, depths, shapes and orientation, purification expected in each section, pre-treatment (if required), simple construction details, inlet and outlet designs, but will neglect structural considerations of any concrete or other structures considered necessary.

Table 1

The sewage flow from the residential and commercial areas can be separated into three sections.

Low density housing

2,500 persons
350 litres/person.d water consumption
77 g/person.d BOD

Medium density housing

7,500 persons
190 litres/person.d water consumption
55 g/person.d BOD

High density housing

40,000 persons
50 litres/day sewage flow
42 g/person.d BOD

Concentration of Coliform bacteria in combined residential plus commercial flow = $12 \times 10^7/100$ ml.

Industrial wastewater

Daily flow	350 m ³
BOD	2675 mg/l
COD	5400 mg/l
Suspended solids	800 mg/l
Volatile suspended solids	600 mg/l
pH	4.0 to 9.5
Combined nitrogen	67 mg/l
Total phosphorus	26 mg/l

Concentration of Coliform bacteria in industrial flow = $1.05 \times 10^6/100$ ml.
Soluble sulphide - up to 25 mg/l

Table 2

Wastewater Treatment Works
Maximum Discharge Conditions

Biochemical Oxygen Demand	15 mg/l
Suspended solids	10 mg/l
Oxidised nitrogen	15 mg/l
Ammoniacal nitrogen	2.0 mg/l
Dissolved oxygen	5.0 mg/l
pH	6-8.5
Coliform bacteria	100/100 ml.

Figure. 1.

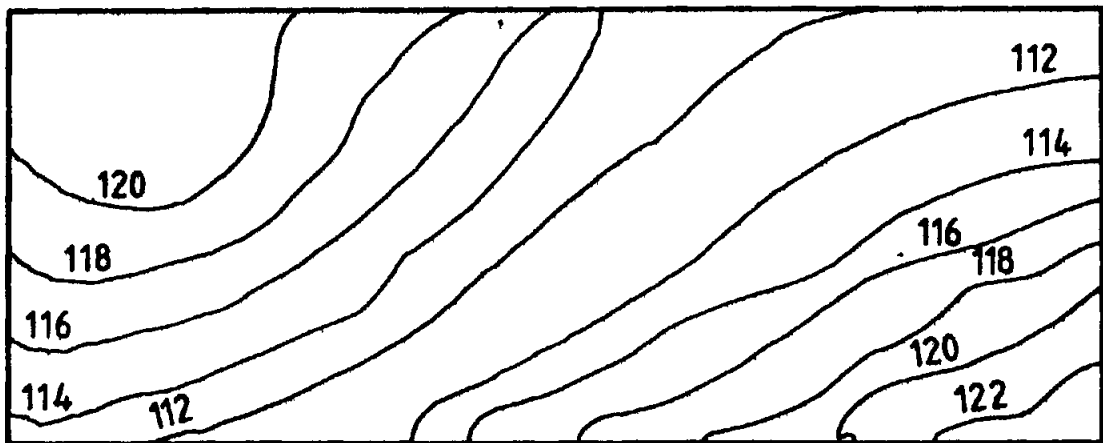


Figure. 2.

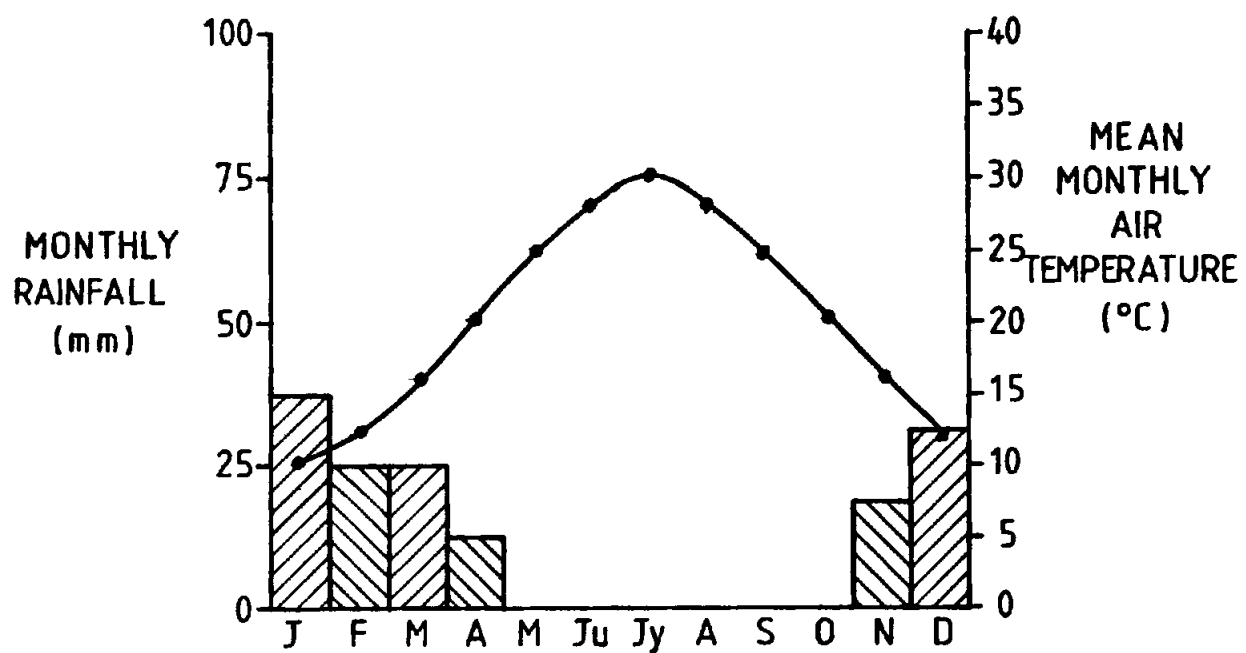


Figure. 3.

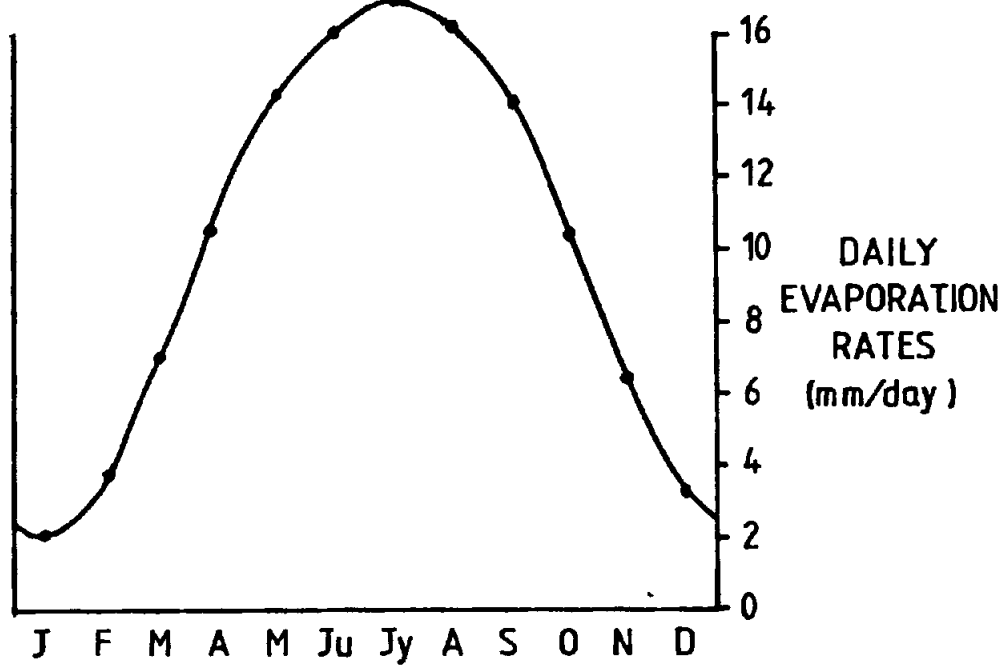
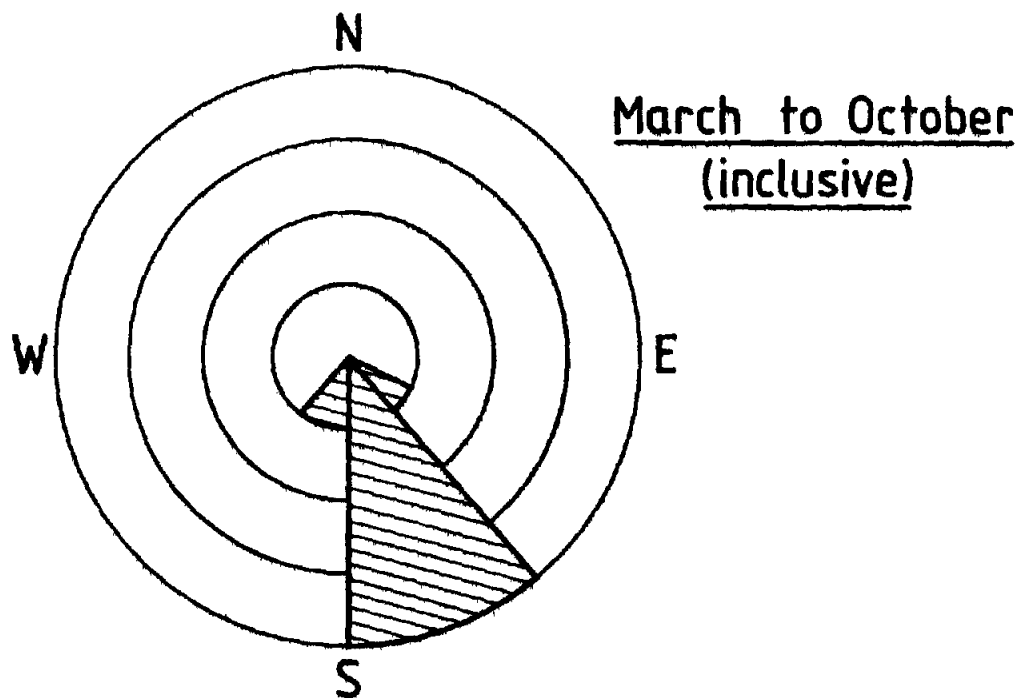
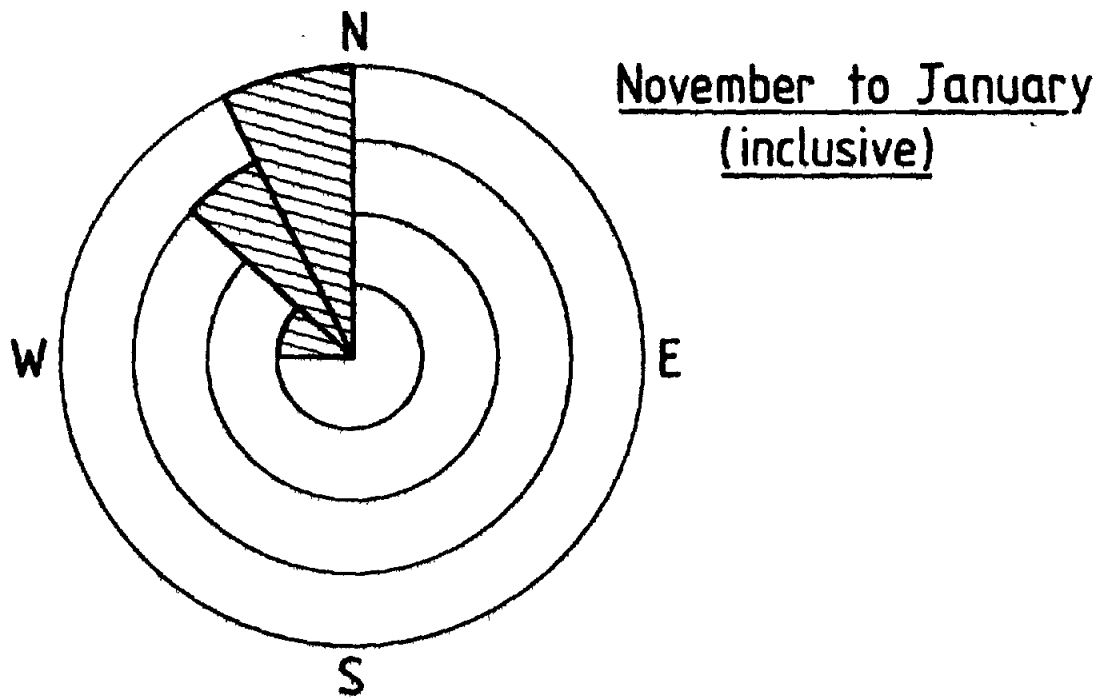


Figure.4.

WIND DIRECTION - as a proportion of total time



PAPER 18

OPERATION AND MAINTENANCE OF STABILIZATION PONDS

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OPERATION AND MAINTENANCE OF STABILIZATION PONDS

One of the principal advantages of stabilization ponds is the very limited amount of operation and maintenance required. Once the pond system is in use, there is very little that the operator can do to alter or improve the situation. There is also very little general maintenance that is required.

If the pond system possesses preliminary treatment processes, i.e. grit removal and screening, then much of the operators time will be involved in cleaning these units and keeping them free from blockages. Grit-channels, which are always constructed in parallel so that one may be readily taken out of operation for cleaning, will require regular de-gritting. The period between grit-channel cleaning will depend on local conditions and will only be learnt from experience. Usually the grit removed will be clean enough for disposal as land-fill, but occasionally it will be necessary to bury it.

Inlet screens, when employed, will almost certainly have to be cleaned by manual raking. Screens can block quickly and hence a close watch must be kept on them. The number of times in a day that screens have to be cleaned will again depend on local conditions of waste water flow and quantity, but often this will have to be carried out every hour. With extensive pond works dealing with flows from large towns an operations staff may have to be employed at night, if manually cleaned screens are installed. This is not a good idea and should be avoided if at all possible. All screening must be buried. The digging of suitable pits can be the most labour intensive job associated with pond operations and one which, if not adequately supervised, will be most disregarded. However, it is essential that all removed screenings be properly buried and covered with at least one-metre depth of earth to prevent access by rodents and flies.

If screens are not employed, and particularly if the first pond is a facultative pond, then floating debris and scum are likely to accumulate on the pond surface and must be frequently removed. Removal is essential to prevent fly and odour nuisance and to allow for the maximum possible re-aeration of the pond directly from the atmosphere. Removal is best achieved by employing long-handled rakes to reach floating material as it floats near to the banks. Occasionally a boat can be used to direct the scum and floating detritus towards the banks. Again, as with screenings, all floating detritus removed must be adequately buried.

Where grit-removal channels are not incorporated as a preliminary process, the grit and sand will settle out immediately under the inlet pipe. In facultative ponds a shallow pit is frequently dug immediately under the inlet to receive this grit and to delay the formation of an island on which reeds and weeds can grow. However, periodically this accumulation of grit must be removed. This is best accomplished by draining the pond and manually removing the deposited grit. If possible all stabilization ponds should be designed to be readily drained. Grit removal will be necessary perhaps only every two years, or even longer, but it will become necessary.

Draining the ponds is also of importance if accumulated sludge has to be removed. This will be necessary, infrequently, with both anaerobic and facultative ponds. Initially the supernatant water has to be drained off or pumped off and then the sludge layer allowed to dry before being removed either by mechanical means or, preferably, manually. If the sludge layer in an anaerobic pond is deeper than about 1.0m difficulty can be encountered in drying it out adequately. Frequently a thick dry layer is produced on the surface, but with semi-liquid sludge still existing beneath. This can make manual removal very difficult. In such a situation it is often better to pump out or bucket out the liquid sludge into a shallow drying area immediately following the removal of the supernatant water. The final disposal of the sludge should be carefully controlled, especially in areas in which parasitic diseases such as amoebic dysentery, human round worm or hook-worm are endemic. Storage of the sludge for an appreciable period, which should be at least six months and preferably twelve months, is essential to prevent re-cycling these parasites to the population if the sludge is going to be finally disposed of for agricultural purposes.

The presence of weeds and reeds in the shallow water along the banks can lead to the production of mosquitoes and flies. Such vegetation should be regularly and frequently cut and removed. This is perhaps the principal maintenance task necessary at many pond sites. The removed vegetation is dried and burned, if possible, or it can be composted.

Grass cuttings along the banks will also be an essential task during the wet season (if any). Long grass makes inspection of the banks difficult and laborious and encourages the presence of burrowing animals and of snakes. In some situations animals such as sheep or goats may be employed to keep the grass down. Again any grass cuttings must be dried and burned or removed and composted. No grass cuttings should be allowed to blow into the ponds.

Daily inspection of the banks and ponds is essential. The operator should walk around all the ponds every day, particularly to check on the state of the banks and to ensure that no damage is being done either by burrowing animals or by erosion. Any damage should be immediately repaired. Sand bags are useful for repairing such damage to the banks. Burrowing animals can be discouraged by, if it is possible, periodically varying the level of the water in the ponds.

In order to guide the operator, a daily check list should be compiled. Should the operator be illiterate this can be produced using symbols instead of words.

An example of such a daily check list could be:-

Walk around all the pond and check for

1. growth of water weeds
2. signs of burrowing animals
3. damage to the banks (erosion, leakage)
4. any build-up of scum on the ponds
5. signs of anaerobic conditions
6. any damage to the boundary fence
7. any evidence of short-circuiting of flows

In addition

1. check all valves, channels and pipes for blockages and clean if necessary
2. check flow meters or weirs
3. clean inlet screens and grit channels and dispose of detritus.

Also

Check condition of the effluent

Check condition of the receiving stream

Record

Rainfall

Temperature (air and water)

Sunshine

and finally

Take any necessary samples.

Approximate maintenance required is one man for every 20 hectares of pond surface.

PAPER 19

AERATED LAGOONS

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AERATED LAGOONS

Aerated Lagoons

1. The aerated lagoon represents a definite increase in sophistication, and in efficiency - if measured in terms of organic removal per unit area or volume - over the stabilization pond and at its most developed is equivalent to a simple activated-sludge plant in which there is no sludge return system. Over recent years aerated lagoons have been found frequently to be an economical alternative to other more conventional treatment processes. Unlike stabilization ponds for which there is only a very limited potential in this country there could be an important future for aerated lagoons in the treatment of industrial wastewater. Many industries which turn naturally to activated-sludge or percolating filters could well benefit by considering the possibilities of aerated lagoons.

There are basically three types of aerated lagoon

2. Aerated stabilization pond - This is merely a facultative stabilization pond with a water mixing device installed. The device does not add an appreciable amount of oxygen to the system but merely brings the oxygen demand, represented by the near anoxic water close to the pond bottom, into contact with the oxygen at the pond surface. There is still a substantial layer of anaerobic sludge at the bottom of the pond, no further settlement is required for the pond effluent and there is still a large algal content in the discharge. This type of pond is normally only suitable for hot country situations in which stratification is liable to decrease substantially the efficiency of the stabilization ponds

3 Facultative aerated lagoons - These facultative aerated lagoons are sometimes referred to as 'partially suspended' lagoons as a certain concentration of volatile solids are continually held in suspension and represent an embryonic bio-mass or mixed-liquor. A low rate of atmospheric oxygen is added mechanically and although there is increased agitation in the pond sludge settlement - and potentially digestion - also occurs. Normally no further settlement of the pond effluent is required but there is still likely to be an appreciable growth of algae in sunny situations, although this is less than with the aerated stabilization pond.

It has been suggested that the power requirement for this type of lagoon is between 1.0 w/m^3 and 2.0 w/m^3 , and that 1.0 w/m^3 represents a solids suspension of about 20 mg/l (3). Solids suspensions of 50 mg/l to 100 mg/l have been suggested but this might involve an additional settlement unit (5). In the summer, if the temperature is sufficient to support active sludge digestion, the injection of the products of partial anaerobic breakdown into the aerobic liquid phase can represent an increase in effluent substrate concentration of 50%. This is known as seasonal feedback.

4. Completely mixed aerated lagoons - In these all the atmospheric oxygen is supplied mechanically and all the solids are in suspension. The suspended solids operate as an immature activated-sludge floc and the nominal retention period must be related to the sludge growth rate so that the removal of solids with the lagoon effluent does not deplete the active mass. These are essentially activated-sludge units but without sludge return systems. The MLSS concentration is in the range of 0.2 to 0.4 kg/m^3 . A final settlement tank or pond is essential. There are no algae in the effluent.

As with all aerobic systems relatively large amounts of additional sludge are produced as the substrate is stabilized. The sludge yield coefficient (Y) varies between 0.5 to 0.8 kg sludge produced per kg BOD destroyed.

Relating this to a critical retention period, to prevent wash-out, it has been estimated that (5)

$$t_{rc} = \frac{1}{0.33Y \cdot k \cdot L_1}$$

where k is the specific substrate removal coefficient ($l \cdot mg^{-1} \cdot d^{-1}$) and L_1 is the BOD concentration of the influend (mg/l)

As there is no sludge settlement or sludge digestion in these lagoons there is no storage of BOD in winter and no seasonal feed-back in summer. The lagoons are therefore designed for the coldest month of the year

5 Starting with the assumption that the rate of BOD removal is proportional to the concentration of microorganisms (X) and to the BOD present (L_e)

and that, of necessity, the lagoon must be a complete-mix system then by taking a mass balance.

$$\begin{array}{ccccc} Q \cdot L_1 & = & V \frac{dL}{dt} & + & Q \cdot L_e \\ \text{BOD in} & & \text{BOD removed} & & \text{BOD out} \end{array}$$

one arrives at

$$L_1 - L_e = \frac{V}{Q} \frac{dL}{dt}$$

and then

$$L_e = \frac{1}{1 + k \cdot X \cdot t}$$

and

$$A = \frac{Q}{D \cdot k \cdot X} \left(\frac{L_1}{L_e} - 1 \right)$$

Then by assuming that the rate of synthesis of bacteria is the product of the Yield Coefficient and the rate of organic removal

$$\text{i.e. } \frac{dX}{dt} = Y \frac{dL}{dt}$$

an additional expression for the retention period in the lagoon can be arrived at

$$t = \frac{1}{Y \cdot k \cdot L_e - b}$$

It is claimed (5) that there are definite advantages in having a facultative lagoon - or a series of facultative lagoons - following a completely mixed aerated lagoon in that less power is required for the facultative lagoon in which all settlement is also accomplished.

For this the total retention period (t_t) in both the lagoons is suggested as being

$$t_t = \frac{1}{Y \cdot k \cdot L_e - b} + \frac{L_e - L_f}{X_f \cdot k \cdot L_f}$$

6. The BOD of the effluent from the lagoon consists of two fractions: the content of unstabilized influent BOD together with the BOD exerted by the organic matter represented by the bacterial cells synthesised during oxidation (X) so that the total BOD of the effluent can be represented by

$$\text{Total BOD} = L_e + 0.54.X$$

7. The pond depth of the completely mixed lagoons is usually between 3 m and 5 m and it is common to line the lagoons with either butyl rubber or polyethylene sheeting to prevent seepage and erosion. A rule-of-thumb for the power required is approximately 6w/m^3 of lagoon capacity, with most power being expended on maintaining the solids in suspension. Retention periods will normally vary from two to six days (1.3) although retention of less than one day will be possible for some easily degradable wastes.

With facultative ponds the rate of sludge deposition can be considerable and the necessity for periodic desludging must be taken into account during the lagoon design. Balanced sludge digestion will still leave about 2/3 of the deposited solids at the pond bottom.

Aeration devices for aerated lagoons may either be of the diffused-air type or surface aerators, although surface aeration will frequently be more acceptable due to the relative ease of running power cables rather than compressed-air lines.

8. For the seasonal industry aerated lagoons can possess a real advantage in that they are easily taken in and out of service with little or no maturation period required. This advantage is also of value in countries with very definite wet and dry seasons and in which the quality and quantity of industrial discharges is only controlled during the season of high flows in the receiving stream.

Aerated lagoons are suitable for treating most types of organic industrial discharges and are widely employed in North America for the effluents from abattoirs and meat processing plants and for the wastes from the fruit and vegetable processing industry. One example shows the reduction of a strong effluent (3000 to 4000 mg/l BOD) from a pea-processing plant, by 76% during a 5.5 day retention period. As the removal rate coefficient is claimed as being 0.044 this indicates a low rate of solids suspension in the system. The removal rate coefficient varies from about 0.02 to 0.04 $\text{mg}^{-1}.\text{d}^{-1}$ for domestic wastes and might fall as low as 0.001 for some intractable organic wastes, although 0.01 to 0.021 $\text{mg}^{-1}.\text{d}^{-1}$ is more common. Obviously this coefficient is greatly effected by temperature and it has been suggested (1) that it will increase according to

$$k_T = k_{20} \cdot 1035^{(1-20)}$$

The rate of auto-oxidation, b, is a first-order reaction coefficient and seems to vary between 0.2 and 0.3 days

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AERATED LAGOONS

SYMBOLS

- Y - Yield coefficient, (kg cells produced per kg BOD consumed)
- k - BOD removal rate coefficient ($l \cdot mg^{-1} \cdot d^{-1}$)
- L - BOD concentration ($g \cdot m^{-3}$)
- Li - Influent BOD concentration ($g \cdot m^{-3}$)
- Le - Lagoon and effluent, BOD ($g \cdot m^{-3}$)
- Lf - Lagoon or effluent, BOD - facultative Lagoon ($g \cdot m^{-3}$)
- X - Concentration of micro-organisms in Lagoon ($g \cdot m^{-3}$)
- t - Retention period (days)
 - t_R - fully suspended lagoon
 - t_f - facultative lagoon
- V - Volume of lagoon (m^3)
- Q - Flow rate ($m^3 \cdot d^{-1}$)
- A - Surface area (m^2)
- D - Depth (m)
- b - Rate coefficient for bacterial auto-oxidation ($days^{-1}$)

PAPER 20

**AERATION OF WASTEWATER:
THEORY, DESIGN AND EQUIPMENT**

By

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AERATION OF WASTE WATER THEORY, DESIGN AND EQUIPMENT

1 Oxygen Requirements

Oxygen is used in aerobic waste water treatment in the reactions which degrade the substrate to yield compounds for cell synthesis and also for respiration. The typical requirement in solution is 0.5-2 mg/l of dissolved oxygen which is sufficient to prevent the process becoming oxygen limited. It is important to note that simply increasing the dissolved oxygen beyond that required to prevent limitation will not increase the rate of biochemical oxidation which has a maximum under ideal conditions which may not be exceeded. Indeed there is evidence to show that the nitrifying bacteria are inhibited at very high levels of dissolved oxygen of about 16 mg/l. This concept should not be confused with increasing the rate of treatment which entails increasing the numbers of micro-organisms in a reactor by settlement and recycling and thereby allowing a higher input load to the same treatment volume i.e. the BOD removed per m³ reactor per hour is increased. When the rate of treatment is thus increased the oxygen requirement per m³ per hour is also increased. In the activated sludge process increasing the Mixed Liquor Suspended Solids (MLSS) will increase the possible BOD removal rate per unit volume, but the limitation on increasing the MLSS is the performance of the secondary settlement tank which depends on MLSS settleability and the rate of recycle.

The daily total oxygen requirement is a function of

- o mass of BOD removed
- o mass of Amm N. removed
- o endogenous respiration rate
- o the mass of micro-organisms
- o denitrification

Most treatment processes are classified directly or indirectly by the F/M ratio. This ratio of food (incoming mass of BOD) to micro-organisms (mass of micro-organisms in the treatment tank) is sometimes known as the sludge loading rate. In the activated sludge process the mass of micro-

organisms is measured indirectly by the MLSS whereas in a biological filter or rotating biological contactor the total surface area of media is the indicator

The choice of F/M ratio determines the operating position on the bacterial growth curve, Figure 1. The operational influences of selecting a particular range are shown in Figure 2, where it may be seen that high F/M ratios produce more surplus sludge than low F/M ratios. This difference reflects the relative significance of endogenous respiration.

If carbonaceous oxidation only is occurring then the oxygen requirement for a high sludge loading or F/M ratio of say $1 \text{ kgs BOD kg MLSS}^{-1}$ might have an oxygen demand of 0.8 times the BOD load compared to 1.6 times the BOD load for an F/M ratio of $\approx 0.1 \text{ kgs BOD kg MLSS}^{-1} \text{ day}^{-1}$. Early work by ECKENFELDER and O'CONNOR (1954) gave a formula for the oxygen requirements

$$O_2 \text{ required} = a \text{ BOD}_{\text{removed}} + b \text{ MLSS}$$

This equation may be rewritten in terms of the F/M ratio and plotted in graphical form as shown in Figure 3 (ASCE, 1977). From this it may be seen that for conventional sludge loading rates (F/M's) of about $0.2 \text{ kg BOD kg MLSS}^{-1} \text{ d}^{-1}$ the oxygen required is $0.9 - 1.2 \text{ kg O}_2 \text{ kg BOD}_{\text{rem}}^{-1}$, whereas at high sludge loading rates of $1.0 \text{ kg BOD kg MLSS}^{-1} \text{ d}^{-1}$ it is about $0.5 - 0.8 \text{ kg O}_2 \text{ kg BOD}_{\text{rem}}^{-1}$.

When conditions are created to ensure complete nitrification (if it is required), the oxygen required will be increased by about 4.2 times the concentration of Ammoniacal Nitrogen. This additional oxygen requirement may be of the same order of magnitude as that for carbonaceous oxidation for an Amm N load of 30 mg/l the O_2 demand is 126 mg/l (over whatever time period) compared to say a settled sewage of 150 mg/l O_2 demand.

Where it is essential to achieve nitrification it is worthwhile designing and controlling an activated sludge system to achieve denitrification. Denitrification is the mechanism under which the end product of nitrification, nitrate, liberates oxygen under zero oxygen conditions in what is

called an anoxic zone. Much of the nitrate can be utilized in this way with a resultant saving in energy costs of about 12 percent. The conditions for nitrification and denitrification are often achieved in oxidation ditches and are possible in activated sludge processes having relatively long periods of aeration i.e. 12 hours or more. The oxygen demand for oxidation ditches is given in the notes on this subject.

In aerated lagoon design it is common to calculate the oxygen required as being equal to the ultimate BOD removed, which is typically about 1.2 times the 5 day BOD. ECKENFELDER et al (1972) propose a relationship for aerobic lagoons based on the BOD removed and the endogenous respiration of the active biomass in the system.

Where there are significant variations in load over the day then the maximum oxygen requirement over a period equal to the retention time in the system should be used in sizing the aerators.

Oxygenation Theory

Several mathematical models have been developed to describe the transfer of gases to liquids. MARAIS (1975) proposes that three stages are involved:

- I - O_2 molecules are transferred from air to the liquid interface and a saturated layer is quickly established
- II - O_2 then passes through the liquid film by diffusion, a relatively slow process
- III - O_2 is then transferred to the bulk liquid by mixing and further diffusion

There are two main groups of aeration equipment:

- o mechanical surface aerators
- o diffused air sub-surface aerators

Mechanical aerator performance is governed by surface renewal whereas diffused air is governed by molecular diffusion.

For surface aerators the rate of transfer of dissolved oxygen is given by

$$\frac{dc}{dt} = k_L a (c_s - c) \quad (1)$$

where

c = concentration of dissolved oxygen

c_s = saturation concentration of dissolved oxygen in water

$k_L a$ = oxygen transfer coefficient (time^{-1})

[k_L = mass transfer or exchange coefficient]

$a = \frac{A}{V} = \frac{\text{interfacial contact area, air/water interface}}{\text{volume of aeration tank or basin}}$

t = time

For diffused air systems the value of c_s varies from the surface to the bottom of the tank due to variation of pressure with depth and also the partial pressure of oxygen within the gas bubbles themselves. A modified equation is proposed by LISTER and BOON (1973)

$$\frac{dc}{dt} = k_L a \left[\frac{1}{1 + k_L a V H / \rho Q} \right] (c_{s1} - c) \quad (2)$$

where,

V = volume of aeration tank

H = Henry's constant

ρ = density of gaseous oxygen

Q = air flow rate

c_{s1} = saturation concentration for water brought into equilibrium with the inflowing air

The usual method of testing and comparing the oxygen transfer capabilities of aerators is to conduct a test in clean water. The water is deoxygenated using sodium sulphite plus a cobalt catalyst. The rate of increase of dissolved oxygen with time is then measured. The method is known as the UNSTEADY-STATE METHOD.

For surface aerator tests a plot of $\ln(c_s - c)$ versus t gives a straight line of slope $k_L a$.

For diffused air aerator tests a plot of $\ln(c_{s1} - c)$ versus t gives a straight line of

$$\frac{k_L a}{1 + k_L a V H / 2 \rho Q}$$

There are two principal performance criteria for the evaluation of aeration equipment, namely.

OXYGENATION CAPACITY (OC)
and OXYGENATION EFFICIENCY (OE)

Oxygenation Capacity is defined as the rate of transfer of oxygen into a completely deoxygenated liquid at a given temperature per unit volume or to a known volume. Temperatures of 10 or 20°C are commonly specified

For $c = 0$ therefore, in equation we obtain

$$\begin{aligned} \text{OC} &= k_L a c_s \quad \text{kgO}_2 \text{m}^{-3} \text{h}^{-1} \\ \text{or } \text{OC} &= k_L a c_s V \quad \text{kgO}_2 \text{h}^{-1} \quad \text{for volume } V \end{aligned}$$

Oxygenation Efficiency is defined as the Oxygenation Capacity per unit of power supplied (wire to water not shaft)

The above measures of performance are obtained under conditions which do not represent those in actual operation for a number of reasons:

- o the real conditions are pseudo-steady-state where there is simultaneous oxygenation and oxygen utilisation
- o the criteria do not account for the mixing resistance in the bulk liquid -see HOYLAND (1979)
- o the real conditions of polluted water result in a different oxygen transfer coefficient ($k_L a$) and also a different oxygen saturation concentration due to waste water constituents and also local atmospheric pressure
- o tank shape, depth and diffuser configuration all influence the performance
- o the required dissolved oxygen concentration in the aeration tank or basin will affect efficiency i.e. if DO 20 percent of saturation OC is 80 percent of maximum potential and if DO 60 percent OC is only 40 percent of maximum
- o $k_L a$ does not vary significantly for MLSS of 2-7 kg m^{-3}
- o measurement of the respiration rate of activated sludge systems has shown that the oxygenation efficiency increases with the rate of respiration - possibly because of direct oxygen transfer or higher concentrations of bacteria at the water/air interface

The above factors are accounted for, in part, by the use of α and β constants and correcting the required aerator capacity for variations in atmospheric pressure

α is the ratio of the $k_L a$ under operating conditions to that in clean water. The value of α is affected principally by the influence of pollution on surface tension and may thus be simulated under laboratory conditions by the addition of 5 mg/l of anionic surfactant. The influence of the α factor differs according to the type of aerator -

for surface aerators the α factor may be greater than unity due to reduction in droplet size and the increased oxygen transfer i.e. the liquid-air contact area is increased raising the $k_L a_{\text{actual}}$

for diffused air aerators the α factor may be as low as 0.3 due to the surfactant boundary at the surface of the bubble

Other influences on the α value are the type of system i.e. completely mixed or plug flow and the F/M ratio or degree of treatment. For plug-flow systems the value of α will vary along the reactor inlet to outlet for example from 0.4 to 0.9 for a diffused air system and from 1.2 to 0.9 for a surface aerator. Where the degree of treatment is high, as in extended aeration systems, then the value of α will be similar for either system of aeration.

Aerator efficiencies ($\text{kgO}_2 \text{kWh}^{-1}$) are quoted by manufacturers in clean water, whereas the important consideration is the efficiency under the actual conditions. However, it is possible to have the same field efficiency from very different oxygenation capacities and α factors as shown below

	oxygenation efficiency in clean water ($\text{kgO}_2 \text{kWh}^{-1}$)	α	actual oxygenation efficiency ($\text{kgO}_2 \text{kWh}^{-1}$)
surface aeration	1.7	1.2	2.0
diffused air	3.3	0.6	2.0

The value of β is very often assumed to be unity

The effect of tank depth, shape and diffuser configuration is assessed for diffused air systems by LISTER and BOON (1973)

The oxygenation capacity under the actual conditions compared with those under standard test conditions is given by

$$OC_{\text{actual}} = OC_{\text{standard}} \frac{\alpha}{c_s} \left[\left(\frac{p-e}{760-e} \right) \beta c_{s_T} - c \right]^{T-20} \quad (3)$$

where α, β, c_s are as already defined

p = atmospheric pressure (mm Hg)

e = saturated vapour pressure at site (mm Hg)

$c_{s_T} = c_s$ at temperature of waste water (mg/l)

T = temperature ($^{\circ}\text{C}$)

The above equation (3) is suggested for use by PEARSON (1970), BARTSCH and RANDALL (1971), ECKENFELDER et al (1972) and ARCEIVALA (1973)

Typical oxygenation efficiencies are

diffused air	- fine bubble	1.5 - 3.6 $\text{kgO}_2 \text{ kWh}^{-1}$
	- coarse bubble	0.9 - 1.2
surface aerators	- vertical	1.5 - 2.2
	- horizontal	1.2 - 3.4

BINGEL (1979) produced two interesting graphs one regarding the relationship between oxygenation efficiency and capacity shown in Figure 4, and the other showing the deviations from manufacturers guaranteed values of OE, Figure 5.

2 Aerator Design and Equipment

It has been stated already that there are two principal classes of aeration equipment surface aerators and diffused air aerators Any aeration

device must provide sufficient oxygen for organic degradation and bacterial respiration as well as adequate mixing without destroying the biological floc

Diffused air equipment includes

- o porous domes - fine bubbles
- o diffuser tubes - medium - fine bubble
- o spargers - coarse bubble
- o static tube mixers - coarse - medium bubble
(tubular diffusers) good for lagoon applications
- o jet aerators - fine - medium bubble

These systems are sketched in Figure 6

Surface aeration equipment includes

- o low speed radial flow - problems of noise, aerosol and odour
- o high speed axial flow - mainly for lagoon application
- o horizontal brush, cage or mammoth rotors

The above systems are sketched in Figure 7

3 Closure

From the foregoing it will be clear that the design and specification of aeration equipment is not as simple as perhaps would be expected. Equipment specifications do not apply to actual field operating conditions and many factors must be taken into account in aerator selection and design. Available equipment from the manufacturers and the recommended configurations and tank sizes should be considered in all designs.

Figures

- Figure 1 - Bacterial growth curve
- 2 - Influence of F/M on the activated sludge process
- 3 - Oxygen requirements and the F/M ratio
- 4 - Measured OE versus OC for different systems
- 5 - Deviation from guaranteed OE's for particular surface aerator
- 6 - Diffused air aerators
- 7 - Surface aerators

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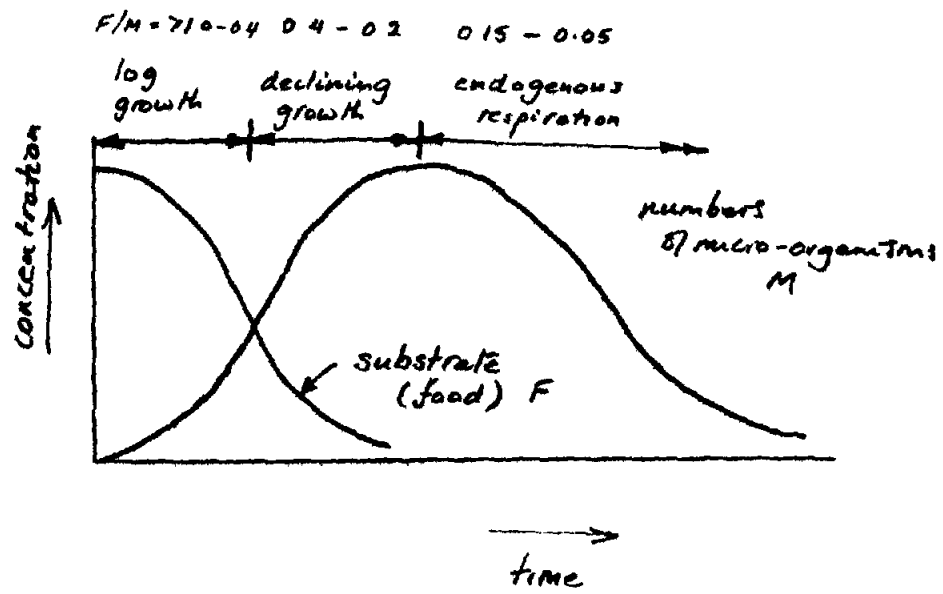
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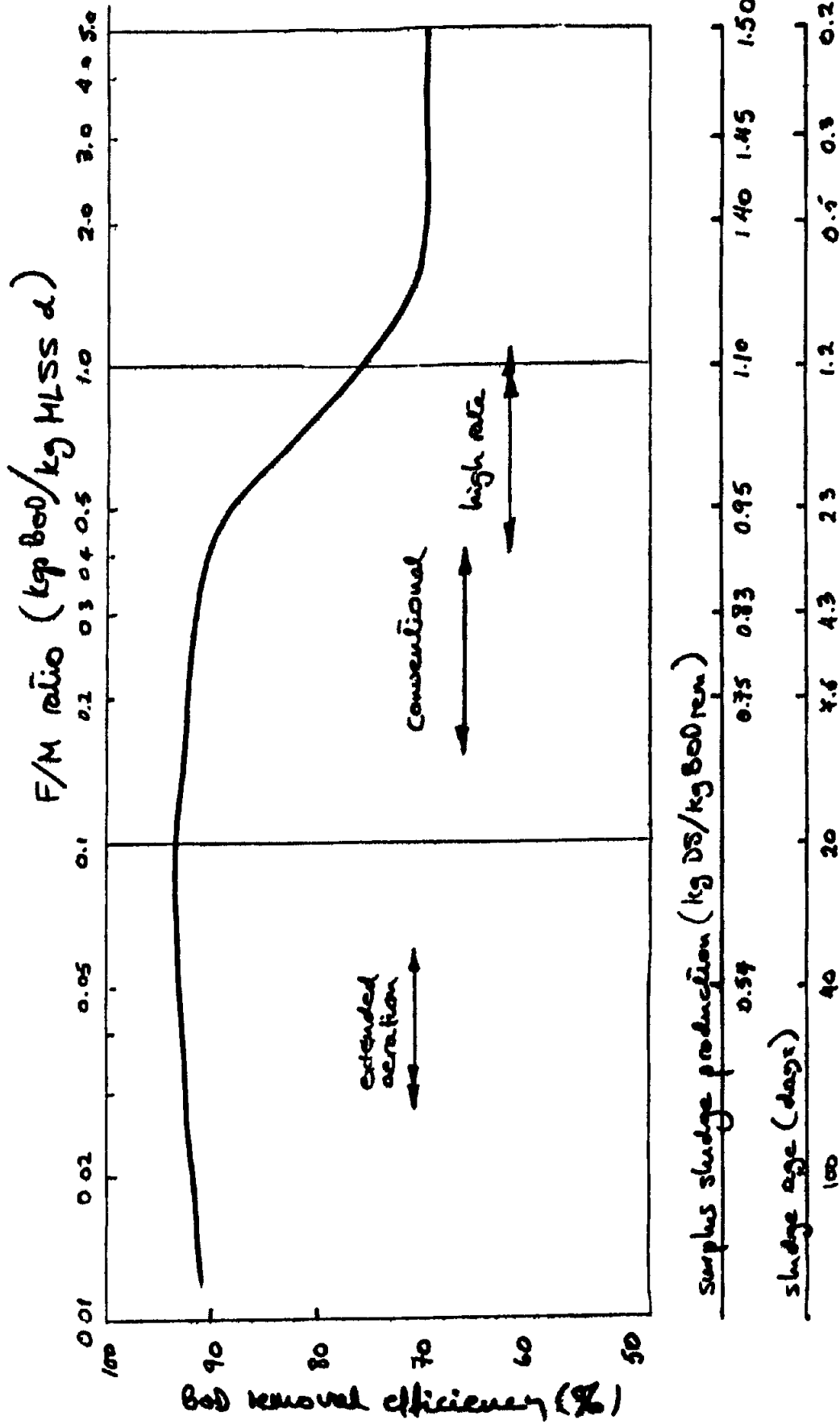


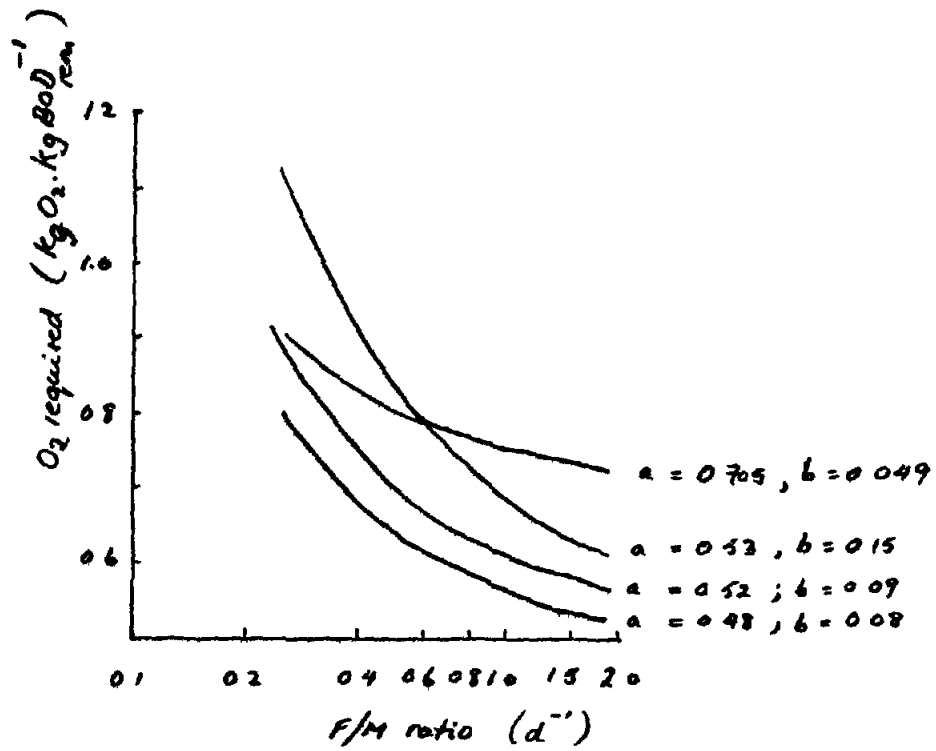
Batch Biological Growth Curve

Figure 1

Influence of F/M on the Activated Sludge Process

Figure 2

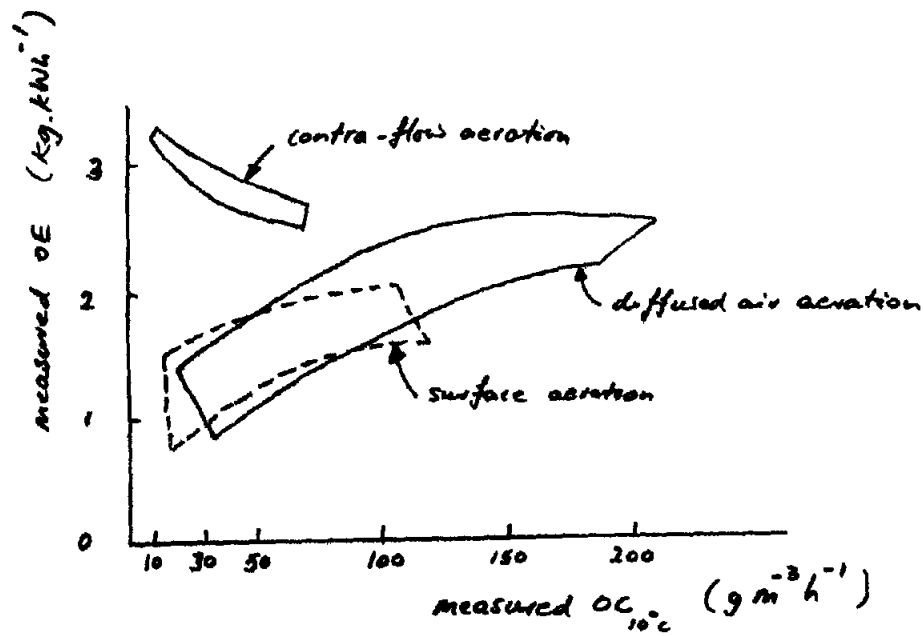




Oxygen Requirements and the F/M Ratio

Source: ASCE (1977)

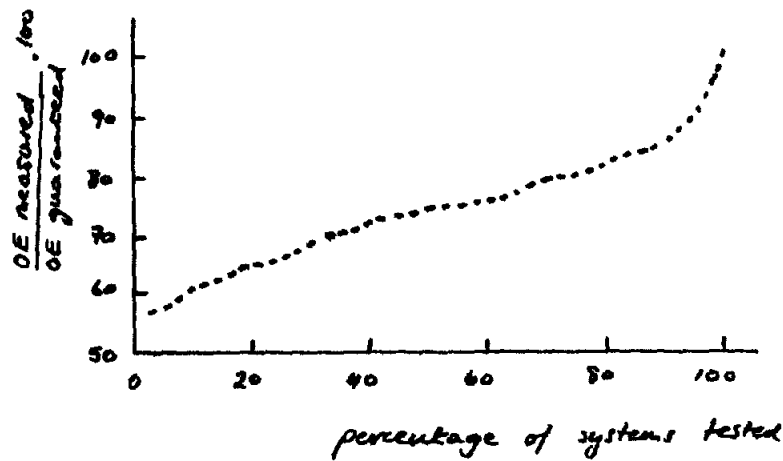
Figure 3.



Measured OE versus OC for different aeration methods

Figure 4

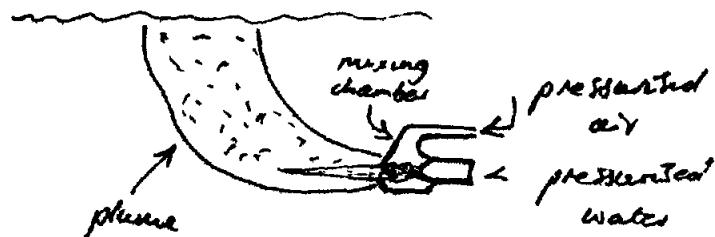
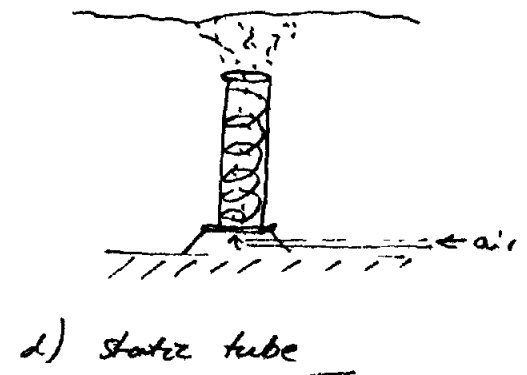
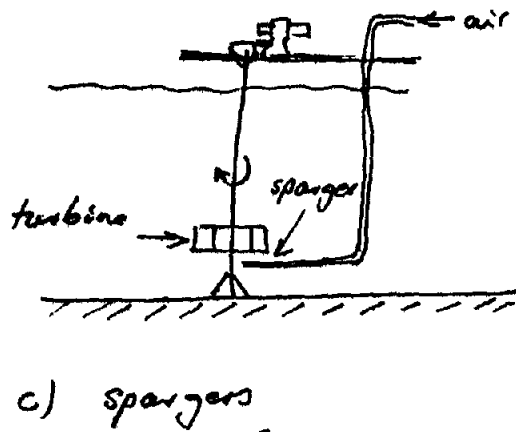
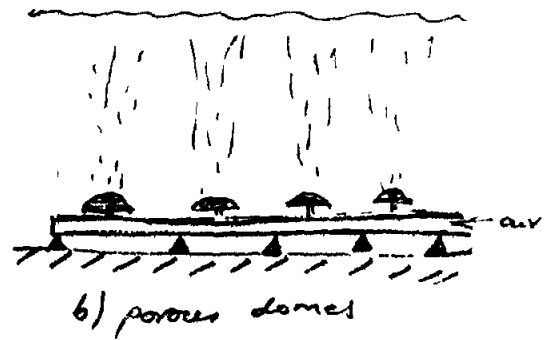
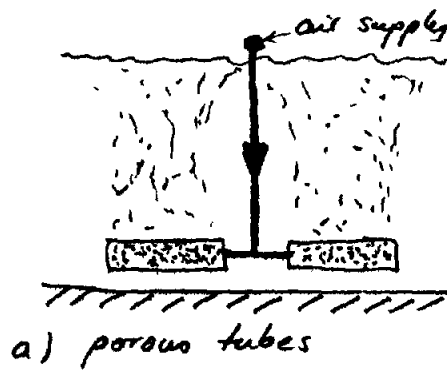
Source: BINGEL ()



Deviation of Measured Oxygenation Efficiency from that Guaranteed

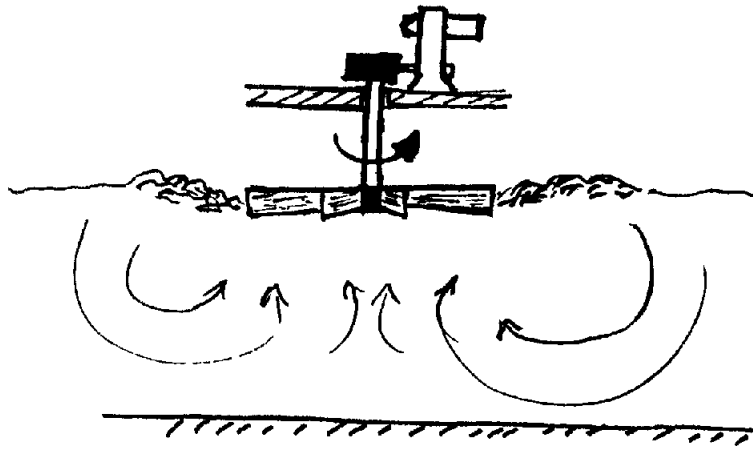
Figure 5

Source: BINGEL ()

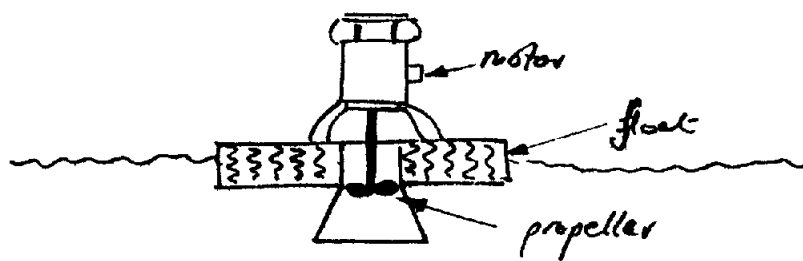


Diffused Air Aerators

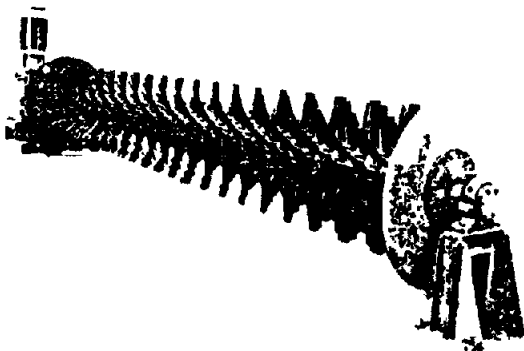
Figure 6



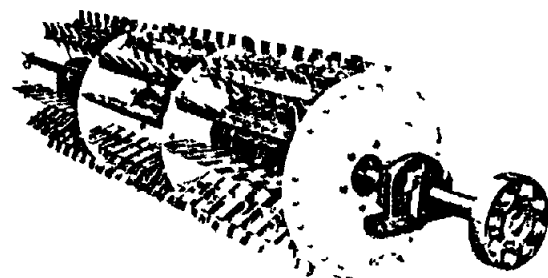
a) low speed radial flow



b) high speed axial flow.



G1) mammoth rotor



E11) TNO or cage rotor

ANNEX I

LIST OF PARTICIPANTS IN NATIONAL COURSE IN EGYPT 12 - 21 September 1981

Dr Salwa Shehata
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ANNEX II

WHO - FACULTY OF ENGINEERING AND ARCHITECTURE
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COURSE ON WASTE STABILIZATION PONDS
(DESIGN AND OPERATION)
KHARTOUM 12 - 21 DECEMBER 1981

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13. Abdel Gadir El Tahir El Tilib
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Sanitary Eng. Administration - Khartoum Province
15. Mohamed Mustafa Younis
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ANNEX III

COURSE PROGRAMME

Day 1	am	Review of wastewater treatment practices	KVE*
	am	Preliminary and primary treatment	JL**
	pm	Chemistry and parameters of wastewater	KVE
	pm	" " " " "	KVE
Day 2	am	Percolating filters and rotating disc units	JL
	am	" " " " "	JL
	pm	Microbiology (Introduction to)	KVE
	pm	" " "	KVE
Day 3	am	The concept of treatment by stabilization ponds	JL
	am	Anaerobic ponds	JL
	pm	Activated sludge	KVE
	pm	" "	KVE
Day 4	am	Extended aeration and oxidation ditches	JL
	am	Facultative ponds (function, advantages etc)	KVE
	pm	Process design of stabilization ponds	JL
	pm	" " " "	JL
Day 5	am)	SEMINAR/REVIEW	
	am)		
	pm	Details of design and construction	JL
	pm	" " " "	JL
Day 6	am	Maturation ponds	KVE
	am	High-rate ponds and algae recovery	KVE
	pm	Design exercise	
	pm	" "	
Day 7	am	Design exercise	
	am	" "	
	pm	Pond operation and maintenance	KVE
	pm	" " " "	KVE
Day 8		FIELD TRIP	
Day 9	am	Aerated lagoons	KVE
	am	Aeration - theory, design and equipment	JL
	pm	Design Exercise - Presentation of results	
		Design exercise	

* Mr K.V. Ellis, WHO Consultant
** Mr J. Lumbers, WHO Consultant

ANNEX IV

SELECTED BIBLIOGRAPHY FOR SEMINAR ON
WASTE STABILIZATION PONDS,
DESIGN AND OPERATION,
LAHORE, PAKISTAN

29 December 1979 - 5 January 1980

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ANNEX V
STATE-OF-ART REFERENCES ON
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