MODEL STUDIES OF WASTEWATER STABILISATION PONDS

BY

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DECLARATION

This Thesis is my original work and has not been presented for a degree in any other University.

CANDIDATE

This Thesis has been submitted for examination with my own approval as University Supervisor.

SUPERVISOR

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CONTENTS

SYMBOLS

A	Mid-depth area (m^2).
a	Ratio of overall transfer coefficient of waste water to overall transfer coefficient of tap water.
a'	BOD removed and used to provide energy for growth as a fraction of ultimate BOD or COD.
a"	Coefficient (See Section 3.1.)
B	Coefficient for unavailability of oxygeny in waste stabilisation ponds.
BOD_{ult}	Ultimate BOD applied per hectare per day.
b	Rate of cell decay.
C	Concentration of reactant/tracer after time t.
Ce	Concentration of the effluent
Ci	Concentration of the influent.
Co	Weight of tracer divided by volume of basin.
Cp	Level of dissolved oxygen in the pond (mg/l).
Cs	Oxygen saturation level of distilled water at 20^{o}C (mg/l).
Csw	Oxygen saturation level in pond at Temperature T (mg/l).
D	Axial dispersion coefficient (L^2T^{-1}).
Dp, Dm	Axial dispersion coefficient for prototype and model respectively.
d	Dispersion number (dimensionless).

F	Efficiency of light conversion.
F_A	Correction factor for altitudes above about 1200 m.
H	Depth of pond (m).
Hc	Heat of combustion of algal cells (=6 cal/mg).
I, I_2	Points of inflection for a tracer curve.
I_L	Intensity of light or solar radiation ($cal/cm^2/day$).
K	First order BOD removal constant (day^{-1})
K_b	Bacterial removal rate constant.
K_n	Design coefficient for anaerobic ponds.
Ks	First order rate constant for soluble BOD removal in aerated lagoons (day^{-1}).
K_T	First order rate constant at Temperature T.
L	Characteristic length or size.
Le	Effluent BOD (mg/l).
Li	Influent BOD (mg/l).
Lm	Size of the model.
Lp	Size of the prototype.
L'r	Areal 5-day BOD removal (Kg/ha/day).
Lr	BOD removed.
L_s	BOD_5 loading (Kg/ha/day).
m	desludging interval for anaerobic ponds (years).
N	Population served.
n	Constant for design of anaerobic ponds.
Ne	Number of faecal coliforms in the effluent.
Ni	Number of faecal coliforms in the influent.

O	Rate of oxygen production (Kg/ha day).
O'	Oxygen production (mg/cm^3).
Od	Oxygen required (Kg/day).
Om	Manufacturer's rating of aerator (Kg O_2/hph).
O_n	Oxygen required for complete mixed regime.
Os	Oxygen supplied by mechanical aeration (Kg O_2/hph).
P	Power of the mechanical aeration required.
Qe	effluent flow (l/day).
Qi, Q	Influent flow (l/day).
q	Per capita waste water flow (l/day).
R	Retention or detention time (days).
R_a	Retention time for anaerobic ponds.
R_f	Retention time for facultative ponds.
Rm	Retention time for maturation ponds.
R_n	Retention time in the $n^{\underline{th}}$ pond.
R_T	Retention time at temperature T.
S	Scale factor.
Se	Soluble BOD in the effluent of an aerobic lagoon (mg/l).
Si	Soluble BOD in the influent of an aerobic lagoon (mg/l).
So	Solids contributions per person per year (m^3).
$\hat{T}$	Theoritical detention time (days).
$\bar{t}$	Mean or actual detention time.
t_{r-1}	Time of maximum growth rate of the tracer curve.

t_1	Initial arrival time of tracer.
t_{max}	Maximum concentration time of tracer.
U	Fluid velocity (LT^{-1}).
V	Volume of reactor or pond.
V_L	Volumetric loading (g/m^3/day).
W	Production of algal cells (Kg/ha/day).
X	Cells in the aerated lagoon.
X_t	Volatile suspended solids in mixed liquor (mg/l).
Y	Yield coefficient of cells in aerated lagoon.
σ	Variance of the tracer curve.
σt	Standard deviation of the tracer curve.

SUMMARY

The performance of waste stabilisation ponds is affected by many factors among which is mixing. The mixing characteristics include short-circuiting and dead spaces, all of which can be represented by the dimensionless coefficient of diffusivity or dispersion number.

The above mentioned mixing characteristics will also affect the first order BOD removal constant, K. The constant K is an important criterion in the design of facultative ponds.

The report classifies waste stabilisation ponds and also discusses the factors which affect their performance. An outline of common flow patterns is also contained in the Thesis. The dispersion Index method, a statistical method discussed in Chapter Two, is used to calculate the values of dispersion number from the tracer study results. Chapter Three contains a review of some methods used in design of waste stabilisation ponds. Facultative ponds have been given some emphasis because they are widely used in Kenya.

The experimental studies consisted of tracer studies and BOD reduction experiments on four different sized perspex glass model ponds. The tracer studies were performed to establish the effect of hydraulic loading, length, breadth and inlet position on the dispersion number. For

a given hydraulic loading, increase in length to breadth ratio decreased the dispersion number; whereas for a given pond geometry increase in hydraulic loading decreased the dispersion number. A change of inlet position affected the dispersion number of a model pond.

In the BOD reduction experiments, synthetic chemical sewage, artificial illumination and an effluent seed from an existing pond for initial biological life were used. The values of K were estimated from the experimental results using the method suggested by Thirumurthi (1969). A mention on the usefulness of the experimental data to practicing engineer is also made.

CHAPTER ONE

INTRODUCTION

Conservation of the Environment in any community is man's prime need. The solid and liquid wastes from such communities which may be domestic, agricultural or industrial or a combination of the above, should be disposed of in a satisfactory manner so as not to render our surroundings unpleasant and unhealthy. Water is an important raw material for survival of man. The growth and operation of industries will also be affected by availability of water. For the above reasons, our waters should be kept unpolluted. Partly, to achieve this, the waste waters should be properly treated because the effluent of the waste water plants eventually finds their destiny to the stream.

The present energy crisis and inflation, has increased a need for developing countries to adopt low cost methods for treating waste waters. Also with inreasing urbanisation the traditional and conventional methods of waste disposal will create both economic and healthy problems.

Some form of biological treatment provides the most economical solution for handling domestic and most industrial waste waters - Gloyna (1971).

Waste Stabilisation Ponds, a system designed for biological treatment, as discussed in 1.2 are most suitable for developing countries. To achieve proper functioning of these ponds, adequate engineering, proper maintainance and supervision are necessary. Adequate engineering includes proper understanding of factors which affect the performance of Waste Stabilisation Ponds. A proper approach which ensures proper performance and ultimately long life is therefore necessary.

A number of design procedures have been developed as discussed in Chapter Three. However, a need to develop a new approach of design based on sound scientific and mathematical principles of Chemical Engineering Unit operations has been emphasized by Thirumurthi (1969).

1.1. CLASSIFICATION OF WASTE STABILISATION PONDS.

Waste Stabilisation Ponds are artificial ponds widely used and recognised as one of the methods of biological treatment of waste water. The classification can be based on the type of influent received into the pond which can be untreated, screened or pretreated. The pond overflow conditions may be a basis for classification. However, depending on the Environmental conditions and quantity of flow, ponds can be classified into non-existent, intermitent flow or continuous overflow types. If the mode of oxygenation is used as a basis for classification, photosynthetic, atmospheric surface transfer or mechanically aerated waste stabilisation ponds will be categorised. Widely used, is the classification based on biological activities under which are the following:

(a) Aerobic Waste Stabilisation Ponds

(b) Anaerobic Waste Stabilisation Ponds

(c) Factultative Waste Stabilisation Ponds

(d) Maturation Ponds.

1.1.1. Aerobic Waste Stabilisation Ponds.

In these type of ponds, aerobic bacteria break down the wastes with the help of algae photosynthetically providing enough oxygen to maintain the aerobic conditions. The extensive application in the tropics has not been realised yet and " ... have a place in planning of low cost waste water treatment systems" Gloyna (1971).

1.1.2. Anaerobic Waste Stabilisation Ponds

Anaerobic stabilisation of waste water is achieved by the action of anaerobic bacteria - those which do not require free dissolved oxygen. The source of anaerobic oxygen may be nitrate, sulphate or various organic compounds which are present in the waste water. The settlement of settleable solids at the bottom of the ponds results into intensive anaerobic digestion and ultimately a very high BOD removal is attained. The breakdown of wastes in aquaprivies, septic tanks and anaerobic ponds seems to be identical and will sometimes result into production of unpleasant and ordiferous compounds. Although most designers systematically use anaerobic ponds for pretreatment of domestic waste waters prior secondary treatment, they are very suitable for treatment of strong industrial wastes.

1.1.3. Facultative Waste Stabilisation Ponds

Facultative ponds are those in which the upper layer is aerobic, the central zone supports both aerobic and anaerobic bacteria (facultative) whereas the bottom sludge zone has anaerobic conditions. In most ponds, aerobic conditions are frequently maintained near the surface sometimes throughout most of the pond depth. But the presence of the settleable organic debris will always persist at the bottom resulting into anaerobic digestion. As a result of this situation, most waste stabilisation ponds for primary treatment develop into some type of facultative pond system hence similar to lakes and rivers.

1.1.4. Maturation Ponds.

Maturation ponds primarily function to reduce the number of pathogenic bacteria, viruses and the cysts and ova of intestinal parasites through extended detention time. This function is particularly vital in tropical countries where the waste water plant effluent is directly discharged into a stream downstream of which is used as a source of drinking water without treatment. Some maturation ponds have been advantageously used to rear fish.

1.2. ADVANTAGES OF WASTE STABILISATION PONDS.

Waste stabilisation ponds have been widely used in the world for treatment of waste waters. Gloyna (1971) discusses the extent of pond usage in the world. The suitability of use of waste stabilisation Ponds in developing countries such as Kenya, where tropical climate is favourable, funds and trained personnel are in short supply has been emphasized by Marais (1966), Mara (1975) and Report No. 9 (1973). Mara (1975) lists the advantages of the ponds which can be summarised as follows:-

1.2.1. Cost of Construction and Maintainance.

Waste stabilisation ponds can achieve any required degree of purification at the lowest cost and with minimum maintainance by unskilled labour. Land for construction of these ponds can be obtained cheaply within proximity for most developing countries. Holland (1973) observed that the unit capital cost for oxidation ponds in Kenya in 1972 was K.Sh. 120 per person served regardless of the total contributing population with a basic assumption that the cost of land is not more than K.Sh. 15,000 per acre. Also that the average running costs of these ponds is approximately K.Sh. 6 per capita per annum.

Compared to the conventional sewage treatment, the same author reported a cost of K.Sh. 400 per person served for towns of populations between 5,000 and 30,000 persons, costs being normally higher for towns with less than 5,000 persons. Table 1.1. shows a comparison between costs of waste stabilisation ponds and other possible alternative treatment methods.

Process	Annual Costs (Rupees/Person)
Conventional Treatment	3.5 to 13.2
Oxidation Ditch	3.8 to 6.0
Aerated Lagoon	2.8 to 4.8
Waste Stabilisation Ponds	0.9 to 2.3

Table 1.1. * COMPARISON OF COSTS OF VARIOUS WASTE TREATMENT PROCESSES.

* (Based on the data given by Central Public Health Engineering Institute in Technical digest No. 10 "Cost of sewage treatment" October 1970. The figures of cost include repayment of capital costs at 6% for 20 years.)

1.2.2. Efficiency.

Waste stabilisation ponds can effectively treat a wide variety of industrial and agricultural wastes. Toxic compounds present in Industrial wastes can be removed effectively through precipitation, absorption and sedimentation. This property makes the ponds able to withstand organic shock loads. They will also resist hydraulic shock loads. The removal of pathogens by maturation ponds is much superior to other methods of waste water treatment.

1.2.3. Economic Benefits.

The algae produced in waste stabilisation ponds are a potential source of high protein food which can be conveniently exploited by fish farming, chicken or duck feeding. The method of construction of the ponds is also such that should at some future date the land be required for some other purposes, it is easily reclaimed.

1.3. THE STABILISATION PROCESS.

The process begins immediately after the waste water enters the pond. The settleable solids, suspended matter and colloidal particles either settle by gravity or become precipitated by action of soluble salts. The soluble material is decomposed by the bacteria. The settled organic matter is also later decomposed to produce inert residue and soluble substances. The soluble substances later diffuse into the water above where further degradation occurs.

1.3.1. The Aerobic Process.

The aerobic conversion of the dilute liquid wastes occurs in the presence of free oxygen and bacteria but may also include fungi and protozoa. The decomposition results into bacterial sludge, carbondioxide and water. Much of the carbon functions as a source of energy for the organisms. The remainder of carbon, phosphorus and nitrogen are used by the same organisms to form new cells. The solids predominantly consist of Carbohydrates with varying quantities of protein, fat and ash. The classical biochemical reactions that occur in aerobic metabolism will include:

(i) Carbohydrate + Oxygen $\longrightarrow$ Carbon dioxide + Water + Energy + Unused Carbohydrate.

(ii) Protein (organic N) $\longrightarrow NH_3 \longrightarrow NO_2^- \longrightarrow NO_3^-$

(iii) Organic Sulphur $\longrightarrow$ Sulphate

(iv) Organic Phasphate $\longrightarrow$ Phosphoric Acid $\longrightarrow PO_4^=$

The quantity of oxygen required to stabilise the organic matter in the waste depends on BOD satisfied during the treatment. This BOD is mainly supplied by the photosynthesis or may be obtained from nitrates, phosphates or/and sulphates. Therefore in the presence of sunlight, algae grows in the ponds with conversion of carbon dioxide into organic compounds and free oxygen.

1.3.2. Anaerobic Process

In the absence of free oxygen, carbon, nitrogen, phosphorus, and other nutrients are converted into cell protoplasm. Oxygen is also required for anaerobic decomposition, but originates from chemical compounds - nitrates, sulphates. The breakdown of the material occurs in two main stages as below:

(i) Acid producing bacteria degrade the organic matter into organic acids, aldehydes, alcohols and other related compounds.

$$5(CH_2O)_x \longrightarrow (CH_2O)_x + 2CH_3COOH + \text{Energy.}$$

(ii) The growth of methane bacteria results into decomposition of organic acid into methane and carbon dioxide.

$$2\tfrac{1}{2}CH_3COOH \longrightarrow (CH_2O)_x + 2CH_4 + CO_2 + \text{Energy.}$$

Both aerobic and anaerobic waste stabilisation processes discussed above will take place in a facultative pond, as shown in Figure 1.1.

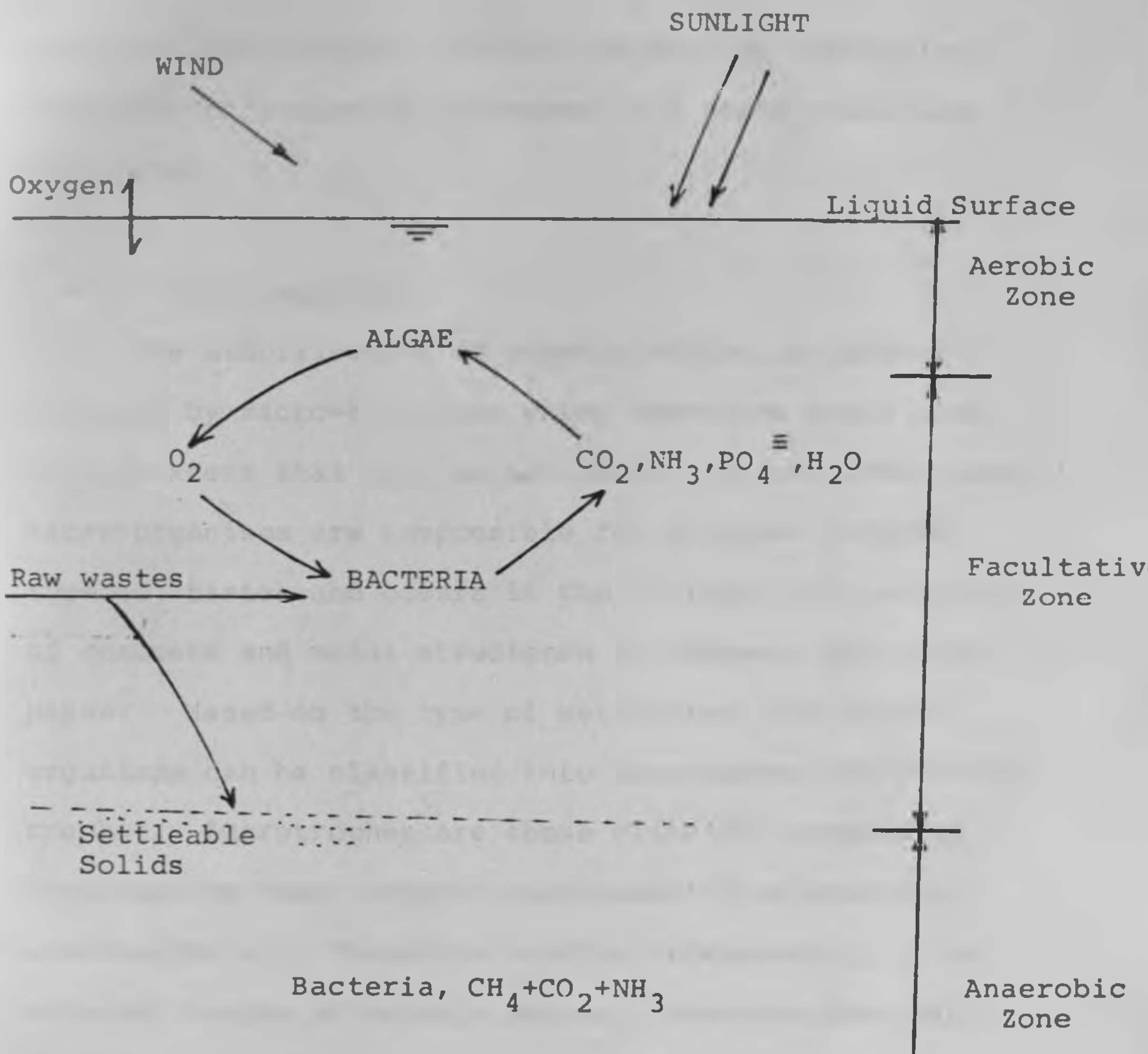


Figure 1.1. SCHEMATIC DIAGRAM OF STABILISATION MECHANISM IN A FACULTATIVE POND.

1.4. FACTORS WHICH AFFECT THE STABILISATION PROCESS

The success of waste stabilisation process is an indication of good performance of the waste stabilisation pond. Therefore factors which affect the performance of these ponds will have affected the stabilisation process. Canter et al (1969) lists the environmental factors which affect the pond performance. Oswald (1968) lists specific environmental factors which affect the pond operation and tabulates the minimum, optimum and maximum quantities required for oxidation processes in a waste stabilisation pond.

1.4.1. Micro-oganisms

The stabilisation of organic wastes is largely achieved by micro-organisms which reproduce their kind to the extent that food is available. On the other hand micro-organisms are responsible for diseases such as typhoid, tastes and odours in the effluent and corrosion of concrete and metal structures in channels and sewer pipes. Based on the type of metabolism, the micro-organisms can be classified into autotrophes and heterotrophes. Autrotrophes are those which are capable of synthesizing their organic requirement from inorganic sources and will therefore survive independently of an external source of organic matter. Heterotrophes will require an external source of organic matter. Saprophobes which obtain soluble organic matter from the surrounding directly or by extracellular digestion of insoluble material, paratrophes which are frequently pathogenic and obtain the organic matter from the tissues of living organisms and phagotrophes which utilise solid organic matter are all examples of heterophes.

Gann et al (1968) experimenting with model ponds found that the bacterial flora consisted mainly of saprophytic, gram negative, rod shaped bacteria. The most occuring genera being achromobacter, pseudomonas and flavobacterium which in all constituted over 80%

of the total plate count. His studies also showed that only commonly occurring gram positive rod shaped bacteria were spore formers of the genus bacillus present in small numbers.

Oswald (1964) reported that Daphnids were quite predominant in stabilisation ponds with detention time of more than ten days.

Many varieties of enteric viruses such as poliovirus, coxackievirus and echovirus have been reported to be present in waste stabilisation ponds - Bopardikar (1964).

Gann et al (1968) reported that the coliform reduction is associated closely with BOD reduction indicating that coliform reduction could be due to their inability to compete successfully for nutrients. They also reported that the principal site of the bacterial activity was located within the vicinity of the influent where 88 to 92% BOD ultimate removal and 88 to 93% coliform reduction occurred. Mc Garry and Bouthillier (1966) reported that the period of survival of salmonella typhi is dependent on the supply of the nutrient.

Gloyna (1971) writes that the survival of enteric micro-organisms depend on retention of sludges, detention of liquids, availability of nutrients, and numerous environmental factors such as toxicity, sunlight, temperature, dilution and mixing, aggregation and predation.

Reduction of faecal bacteria in pond was expressed in equation 1.4.1. by Marais and Shaw (1961)

$$\frac{Ne}{Ni} = \frac{1}{R.K_b + 1} \qquad 1.4.1.$$

Where

$\frac{Ne}{Ni}$ = bacterial removal

K_b = Bacterial removal rate constant

R = Detention time of the pond

Neel et al (1961) reported a removal of coliforms of the order 99.99% in ponds of raw waste water applied at organic loading of 22 to 112 kg BOD_5/ha/day in the temperature range of 2 to 33°C at an operating depth of 0.8m to 1.5m.

1.4.2. Light and Algae

Under natural conditions, the energy for photosynthesis is derived from the solar radiation. The light energy is absorbed by the photosensitive green pigment or chlorophyll of the green plants - algae in case of ponds.

Gloyna (1971) reports that though over 15000 species of algae have been catalogued (Plamer and Tarzwell, 1955) four groups are of importance to waste water treatment. They are:

(i) Blue green algae which are most associated with odours and other nuisance.

(ii) Non-motile green algae which usually form green floating mats in aerobic and facultative ponds

(iii) Pigmented flagellates.

(iv)

Eckenfelder (1961) enlists Chlamydomonas, Chlorella, Euglena and Scenedesmus as typical of the green algae in waste stabilisation ponds. Herman and Gloyna (1958) in their experimental investigations reported Chlorella, euglena, scenedesmus, phacus and oocystis to be predominant.

A useful but conservative estimate of energy use by algae was reported by Krauss (1956) as 20 to 25% efficiency in long term experiments. Oswald et al (1957) with Chlorella and Scenedesmus grown in sewage showed an energy use of 3.8 cal per mg oxygen produced. Further studies indicated that the conversion of energy seldomly exceeded 10% to 12% of the available light energy. The rest of the energy is reflected. Gloyna (1971) observes that the heat of combustion for sewage-grown algae is about 6 cal/mg. Therefore by calculating the visible solar energy, estimating the fraction of solar energy converted to cell material, measuring the heat of combustion of the cells, and establishing a detention time, the overall efficiency of the photosynthetic conversion of solar to algal energy can be determined. The same author noted that the usual efficiencies vary from 2% to 9% with 5% a common value.

Within limits, photosynthetic rate is directly proportional to the intensity of light. As intensity increases, photosynthesis increases proportionately until a saturation point. Beyond this point the rate remains constant until such a high light intensity is reached that photosynthesis is inhibitted - (Bartsch, 1961).

Experimental tests have shown that at the depth where 0.5% of surface light intensity occur, oxygen production is insufficient to meet the respiratory demand of the total biological life (Bartsch and Allum, 1957). The most photosynthesis effective layer is therefore the photic zone. Bartsch and Allum (1957) found that in

Dakota waste stabilisation ponds, 99% of incident light was absorbed in the upper 50 cm to 70 cm.

Krauss (1956) observed that light is not needed continuously but only for initial activation of the photosynthetic action. However, Bartsch (1961) noted that under idealised condition of high turbulence at full sunshine, increase in oxygen production is calculated to reach sevenfold. Naturally, such circulation could occur in sufficiently large ponds exposed to frequent winds.

Therefore direct utilisation of organic matter by algae is limited. The success of algae will be affected by a number of interacting environmental conditions such as deficiency of nutrient, temperature, quality, quantity and duration of light and density of algae.

Since the values of visible solar energy on earth are effected by latitude as well as season, the efficiency of algae will also be consequently affected. The type of algae will also affect the efficiency of algal activities. Canter et al (1969) observed that there was no obvious correlation between organic loading and the predominant algal species.

1.4.3. Nutrients and Toxicity

Domestic waste waters always contain all nutrients for proper bacterial and algal growth. Industrial waste waters, however, are frequently deficient in nitrogen or phosphorus or both and m contain toxic substances. Table 1.2. shows chemical requirements of certain microganisms.

Element	Algae	Fungi	Bacteria	Protozoa
C,H,O	+	+	+	+
N,P,S	+	+	+	+
Mg	+	+	?	+
Ca	+	?	?	+
Co	+	?	?	?
Cu	?	+	?	?
Fe	+	+	+	+
Mn	+	+	+	+
K	+	?	?	+
Zn	?	+	?	?
B	?	?	+	0
Ga	0	?	0	0
Mo	+	+	+	0
Si	?	0	0	0
Na	?	0	+	?
V	?	0	0	0
Sr	?	0	0	0
Rb	?			

* After Krauss (1961) p.45.
\+ Demonstrated to be essential.
? Uncertain whether essential, although a requirement has been demonstrated in some species.

Table 1.2. CHEMICAL REQUIREMENT OF CERTAIN MICRO-ORGANISMS.

Optimal growth of both algae and bacterial requires sufficient food supply containing adequate quantities of nutrients and micro-nutrients. For example the requirement for BOD: Phosphorus: Nitrogen ratio is approximately 100:5:1.

Metal ions such as Sr, Na, Ca, Mg, Mn, Pb, K, Al, Co, Cr, Ni, As, Zn, Cd, Au, Cu, Hg, Ag and other substances such as chloroamines, sulphides, ammonia and free chlorine are known to affect the growth of fish, algae and bacteria. The toxicity action may be due to interference with metabolic rate or unfavourable environment created by the presence of the substance in lethal quantities. Gikonyo (1978) investigating the effect of Cu, Zn, Cr on the performance of model batch waste stabilisation ponds found that the performance of the ponds in terms of BOD and COD removal was not effected for concentrations of the above of 0 to 10 mg/l. The model ponds succeeded in precipitating the metal thus reducing the effective concentrations in the upper water layer where algae and microorganisms predominantly occur.

1.4.4. Temperature.

Metabolic rates of algae and bacteria are influenced by temperature, hence affecting the rate of degredation of wastes. According to Vant Hoff's Theory for chemical reactions, the rate of reaction doubles for every 10°C rise. As the temperature increases to a certain limit, the total retention time required to achieve the same BOD reduction decreases.

Higher temperature make ponds more sensitive to shock loading. Higher, bacterial activity usually results which will result into higher oxygen uptake. Since oxygen production declines at higher temperature, there is an oxygen deficit rendering the pond anaerobic at temperatures usually higher than 35°C. Decrease in temperature slows down the rate of biochemical degredation of organic matter with the result that decomposition products suitable as algal nutrients are less abundant.

Optimum temperature will depend on the species of algae, for example Bartsch (1961) reported that growth of Chlorella (and perhaps many others) reaches a maximum between 25°C and 30°C. Gloyna (1971) observed that optimum oxygen production for some species of algae is obtained at about 20°C, the limiting levels being 4°C and 37°C. Some algae have been observed to grow well under ice cover and at higher temperature. Burlew (1953) reported strains of Chlorella to have fared exceptionally well at 39°C.

Seasonal variations of temperature in the tropical appear to have little effect on the reduction of BOD and high BOD loading rates are possible throughout the year without anaerobic conditions developing - (Marais, 1966).

1.4.5. Mixing.

The responsible factors for mixing patterns in a pond are wind and heat. Mixing by wind depends on the surface area of the pond, the path and the direction of the wind.

Under windy conditions, above the thermocline, a thin static layer of abrupt temperature change the top layers lose their heat more rapidly than bottom layer. The cooler top layers sink inducing mixing, resulting into temperature below the thermocline remaining approximately uniform but gradually decreasing. The thermocline then gradually sinks. When the temperature above and below become equal with further cooling, mixing is initiated and sustained throughout the pond depth. The absence of mixing will therefore result into thermal stratification. Non-motile algae also settles on the thermocline, acting as light barrier rendering the region below the thermocline anaerobic. Marais (1966) reported that under anaerobic conditions (absence of mixing) E. Coli has a very slow die off rate.

Mixing therefore has the following advantages:

(i) It provides a more uniform distribution of nutrient material, algae and BOD.

(ii) It is a quick means by which oxygen arising from both photosynthesis and surface absorption can be transferred to the lower depths.

(iii) Hydraulic short circuiting and formation of stagnant regions are minimised.

(iv) Non-motile algae is carried to zones of effective light penetration (photic zone).

1.4.6. p H.

In a waste stablisation pond, pH rises to a maximum in the afternoon and then decreases after dark reaching the minimum in the morning. It is likely that this variation of pH in the ponds may have a considerable effect on the growth and metabolism of the micro-organism. The high pH values in ponds indicate that the predominant organisms are able to produce high pH values and that the organisms can function actively under the high pH conditions.

Gann et al (1968) observed that the optimum growth of bacterial population predominated by pseudomonas - achromobacter-flavobacterium group, lies between 7.2 and 7.5; and will die off under pH of less or equal to 5.5.

Neilson (1955) cultured algae (of the type commonly present in waste stabilisation ponds) in a media with pH values of 3.0 to 11.0 without apparent inhibition of photosynthesis.

Experimenting with model ponds, Pipes (1962) concluded that the control of influent pH does not have a pronounced effect on BOD removal in the conventional type stabilisation pond, but seems to be an important factor in achieving high BOD removals in high rate type stablisation pond. He also observed that decreasing the pH of the influent waste does not increase the percentage BOD removal in a stabilisation pond with a short detention time.

1.5. THE FIRST ORDER BOD REMOVAL CONSTANT.

The BOD is a measure of biodegradable organic and inorganic compounds in a waste which is expressed in terms of the oxygen requirement for their biodgredation. Often the BOD removal follows the first order kinetics and the removal constant is desiginated the symbol K.

A chemical reactor with plug flow is characterised by the first order reaction formula:

$$Le = Li\,e^{-kt} \qquad 1.5.1.$$

Whereas the complete mixed flow regime modifies the above expression into

$$Le = \frac{Li}{1 + K.t} \qquad 1.5.2.$$

Where

Le = effluent BOD (mg/l)

Li = influent BOD (mg/l)

t = mean residence time or detention time (days).

Thirumurthi (1969, 1974) recommends that reactors which are neither plug nor complete mixed such as waste stabilisation ponds, be designed using Wehner - Wilhem equation for chemical reactors. As stated in Chapter two, equation 2.5.1 can be used to estimate the value of K.

Canter et al (1969) concluded that satisfactory performance of a waste stabilisation pond is dependent upon organic loading rates. Since organic loading affects removals of BOD, hence K, and pathogenic organisms, it can be considered a major criteria for pond design.

For a given hydraulic loading, the BOD removal constant decreases with increasing influent BOD, other environmental factors being kept constant. Similarly the BOD removal constant is a function of solar energy and increases with increasing available photosynthetic energy.

The first order BOD removal constant is a key to the design of waste stabilisation ponds and should be determined individually for each pond (Thirumurthi, 1974). He also recommends that the design value of K should be corrected for temperature, organic loading, industrial toxicity and incident solar energy. Although Parker and Skerry (1968) obeserve that sludge activity has no detrimental effect on algal activity provided that solids are not entrained in the lagoon contents, Thirumurthi (1969) recommends investigations to develop a new correction factor on design K value for benthal loads of sludge blankets.

CHAPTER TWO

HYDRAULIC PROPERTIES OF WASTE STABILISATION PONDS

The efficiency of most processes and operations in sanitary engineering is largely a function of the hydraulic properties of the system in which they operate. Therefore the measurement and analysis of the hydraulic efficiency of such systems are powerful tools in evaluating their performances. Basically description of detention time, distribution of the fluid and the flow regime in a system constitute the hydraulic efficiency.

With proper hydraulic design of the waste water treatment plants a number of objectives such as elimination of excessive turbulence and uniform withdrawal of effluent, in a manner which does not induce short-circuiting in the system, can be achieved. In this way an economical design can be arrived at, hence minimising both constructional and operational costs.

2.1. FLOW PATTERNS

Depending on entrance and exit arrangement, short-circuiting, rate of flow, velocity of flow and volume of tank, fluid passing through a tank or pond could result into several possible patterns of flow. Flow in a tank or pond can be classified into ideal and non-ideal flow. The ideal flow constitutes the plug flow and completely mixed flow whereas the non-ideal flow is one which is neither of the two extreme flows. It is an intermediate type of flow.

Metcalf and Eddy Inc. (1972) enlists the batch flow together with the above three mentioned. The batch reactor flow is characterised by the fact that flow is neither entering nor leaving on a continous basis. A complete mixed batch reactor is a closed system in which the composition and relative distribution of reactants and products are uniform (Weber, 1972). Since there is no inputs or outputs by bulk flow or diffusion in this type of reactor, the material balance equation can be written as:

Rate of accumulation or decay	=	Rate of reaction of the reactant within the reactor

$$\frac{dc}{dt}.V = V.k.c. \qquad 2.1.1.$$

Where

c = Concentration of reactant

V = Volume of reactor

k = Rate constant

The concentration of the reactants decrease with time. For constant V, equation 2.1.1. can be written as:

$$\int_{o}^{t} dt = \int_{c_o}^{c} \frac{dc}{kc} \qquad 2.1.2.$$

Integrating the above;

$$t = \int_{c_o}^{c} \frac{dc}{k.c} \qquad 2.1.3$$

Where t is the time required to change the concentration of the reactant from c_o to c. If kc is known, t and volume of reactor can be determined.

2.1.1. Plug Flow

Also known as piston, slug, tubular or non-mix flow, plug flow is one where the fluid in the reactor is orderly and fluid particles move in parallel paths as shown in Figure 2.1.

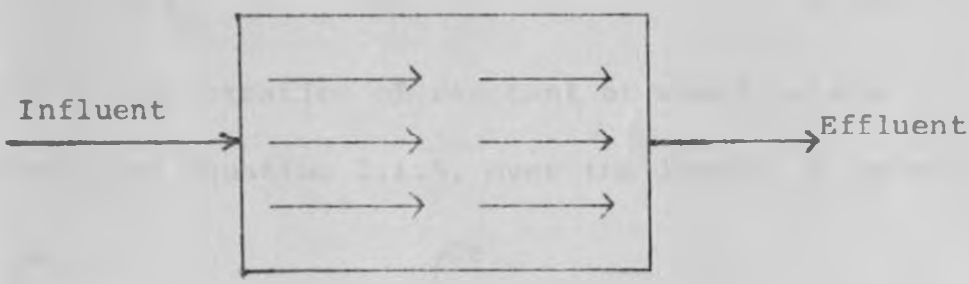


Figure 2.1. PLUG FLOW

The contents are uniform in the transverse direction, and there is no mixing due to concentration gradients in the longitudinal (flow) direction. Also, if two elements of flowing fluid are permitted, there is no velocity gradient because flow advances at a uniform and constant velocity. Each part of the fluid resides in the reactor for a period equal to the theoritical detention time. This type of flow is characterised by high concentration of reactant at the influent and does not respond well to hydraulic shocks. It has, however, an advantage that to accomplish a given extent of a particular reaction for which the rate increases with increasing concentration of reactant, plug flow reactor requires smaller volume than completely mixed reactor.

The dynamic behaviour of plug flow can be described by the differential equation:

$$\frac{\partial c}{\partial t} = -vx \frac{\partial c}{\partial x} + kc \qquad 2.1.4.$$

Where

$\frac{\partial c}{\partial t}$ = rate of change of concentration of reactant

vx = one dimensional velocity in direction x.

At steady state, $\frac{\partial c}{\partial t} = 0$, so that

$$vx.\frac{d\bar{c}}{dx} = k\bar{c} \qquad 2.1.5.$$

Where $\bar{c}$ = Concentration of reactant at steady state.

On integration equation 2.1.5. over the length, L, gives:

$$\int_{o}^{L} \frac{dx}{vx} = \int_{Co}^{Ce} \frac{d\bar{c}}{kc}$$

and

$$\frac{L}{vx} = \frac{L.A}{vx.A} = \frac{V}{Q} = \int_{Co}^{Ce} \frac{d\bar{c}}{kc} \qquad 2.1.6.$$

Equation 2.1.6. can be used to calculate the volume of reactor. For first order kinetics, the mean detention time, $\bar{t}$, for plug flow reactor is given by equation 2.1.7. (Weber, 1972).

$$\bar{t} = \frac{1}{k} \text{ Ln } Co/C. \qquad 2.1.7.$$

This type of flow is approximated in long tanks or ponds with high length to breadth ratio.

2.1.2. Completely Mixed Flow

Completely mixed flow is also known as total back mix or stirred tank. Figure 2.2. shows a completely mixed flow pattern in a reactor.

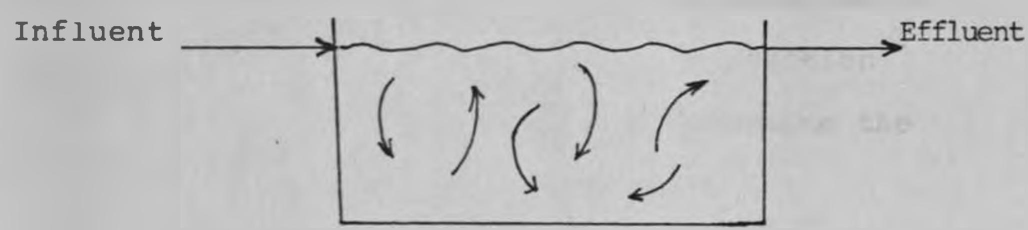


Figure 2.2. COMPLETELY MIXED FLOW

As can be deduced from Figure 2.2, this type of mixing will occur when particles of the fluid entering the reactor are immediately dispersed throughout the reactor with consequent result that there is homogeneuty of contents in the reactor. This type of reactor is resistant to shock loading hence responds well to time - variant input volumes and concentrations because influent reactants are rapidly diluted throughout the reactor. The material balance equation for entire contents of a completely mixed reactor is discussed in section 3.5.2. The derivation of detention time and consideration of competely mixed reactors in series are also given in section 3.5.2. This type of reactor is particularly useful for bench scale experimentation and testing (Weber, 1972).

2.1.3 Arbitrary Flow

Although many designs closely approximate to plug flow and completely mixed flow, these ideals are never fully realised in most full scale process applications.

Arbitrary flow represents any degree of partial mixing between the two extremes of flow discussed in sections 2.1.1. and 2.1.2. In this type of flow, detention times of various parts of the fluid are distributed over a wide range. The flow disturbances in the inlet and outlet zones, density and convection currents and the extent of dead spaces determine the detention time distribution and hence the hydraulic efficiency of the reactor. In this type of flow, it is usually necessary to experimentally determine the flow and mixing characteristics of the reactor.

2.2. THE MIXING PHENOMENA AND DISPERSION EFFECT

Presence of turbulent mixing and velocity gradients in the flowing medium results into axial dispersion. Figure 2.3. shows how dispersion of a dye tracer in a tank or pond occurs by velocity gradient.

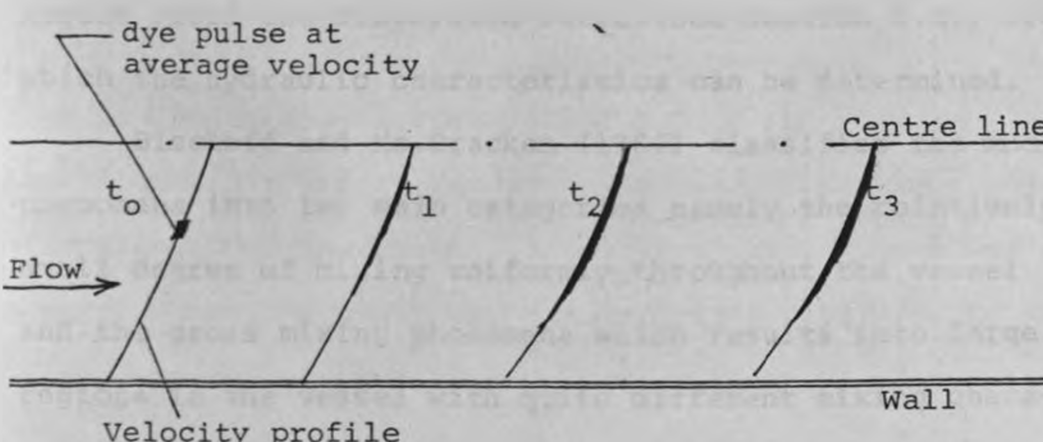


Figure 2.3. DISPERSION BY VELOCITY GRADIENT

(After Friedley, 1972).

Consider a pulse of dye placed in a stream at a radial position having an average velocity. With no diffusion, the dye would be carried downstream intact at the average velocity. However due to diffusivity, the dye tends to spread in the radial direction. (For simplicity the diffusion in the axial direction is neglected). The dye diffusing towards the wall moves into a more slowing liquid and is somewhat delayed. The dye moving towards the centre travels faster than the average velocity and will tend to arrive at the outlet before the mean time. The net effect is the dispersion effect.

The calculation of accurate local velocities of flow needed to form a net flownet in a tank or pond is not only unpractical but nearly impossible. Tracers which can be salt, radioactive substance or a fluorescent dye have been used to yield kinematic results that are functions of the general flow nets involved. (Rebhun and Argaman, (1969), Hirsch, (1969). The tracer techniques yield the dispersion curve (See Section 2.3.) from which the hydraulic characteristics can be determined.

Bischoff and Mc.Cracken (1966) classifies the mixing phenomena into two main categories namely the relatively small degree of mixing uniformly throughout the vessel and the gross mixing phonomena which results into large regions in the vessel with quite different mixing characteristics. Mixing problems that may result will include dead spaces, bypassing and non-uniform regions in the vessel. Non-uniform regions being a more complicated case than simple dead spaces or bypassing. Non-uniform regions are difficult to visualise from the tracer curves (Bischoff and Mc.Craken, 1966).

2.2.1. Dead Spaces

These are stagnant regions with no mixing or no fluid motion. Dead spaces result into loss of effective volume of the vessel and decreases the flow through time. The tracer therefore arrives at the effluent prematurely. Ideally there is no true dead space in a real system because even in a non-moving region, transport of matter would eventually occur by molecular diffusion. Regions whose holding time is between 5 and 10 or more times the holding times of the rest of the fluid, can be considered dead spaces. In real systems, corners and stagnation of upper layers form typical dead space regions.

2.2.2. Bypassing

A fluid can be said to have bypassed a vessel if it passes through a time 0.1 to 0.2 of the holding time of the main fluid stream (Bischoff and Mc Cracken, 1966). In a physical system, instantenous bypassing does not exist since all fluid takes a certain period of time to flow through the system.

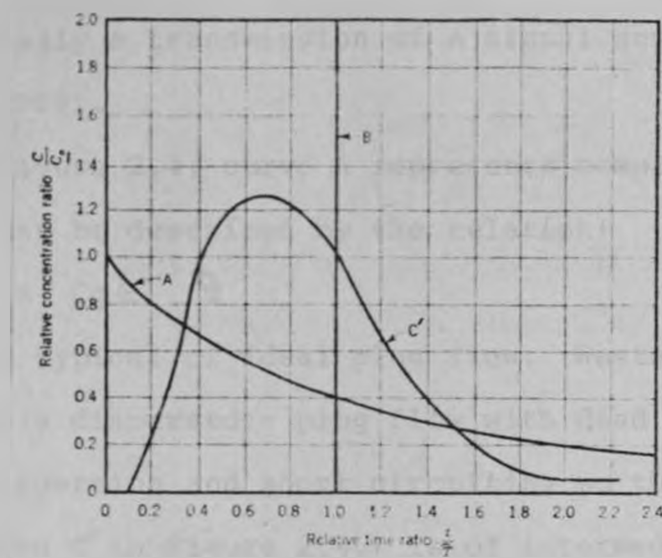


Figure 2.4. FLOW CURVE FOR VARIOUS BASINS
(after Rebhun and Argaman, 1965)

2.3. THE DISPERSION CURVE

The method of tracer technique used for measurement and analysis of hydraulic efficiency of ponds or tanks involves injecting a tracer at the influent and monitoring the concentration of the tracer as a function of time at the effluent. The results of the tracing test can be plotted as dimensionless values of $^{C}/_{Co}$ versus $^{t}/_{\hat{T}}$ as shown in Figure 2.4.

Co = Weight of tracer divided by volume of tank (basin)

$\hat{T}$ = Theoritical detention time

C = Concentration of tracer after time t.

Alternatively $\hat{T}$ and Co can be omitted to obtain a similar plot. The tracer curve or dispersion curve or flow curve obtained could be described as a distribution of detention time of the fluid in the basin (Rebhun and Argaman, 1965) or essentially a transmission of a signal across a basin (Hirsch, 1969).

In Figure 2.4, curve A represents completely mixed flow and can be described by the relation:

$$C = \underline{Co}\, e^{-t/\hat{T}} \qquad 2.3.1.$$

Curve B is typical of ideal plug flow. Waste stabilisation ponds have a dispersed - plug flow with dead spaces, lateral dispersion and short circuiting so that the flow curve (curve C′ in Figure 2.4.) is of intermediate shape.

2.3.1. Evaluation of Dispersion Curve

Once the tracer curve is plotted, the flow characteristics can be determined by evaluating the data obtained from the curve. The most commonly used approaches are the conventional and the statistical methods.

The conventional approach comprises of selection of specific points from the tracer curve as indices to describe the performance characteristics of the basin (Layla et al, 1977, Thirumurthi, 1969).

Different statistical approaches of analysis have been reported. Rebhun and Argaman (1965) proposed an intricate formula and graphical procedure to determine the partition of plug and diffuse (mix) flow. Wallace (1966) and El Baroudi (1966) critised their technique.

Probably the most widely used statistical approach is the dispersion Index method based on chemical engineering. The following expressions are used:-

$$\bar{t} = \frac{\Sigma tC}{\Sigma C} \qquad 2.3.2.$$

$$\sigma t = \frac{\Sigma t^2 C}{\Sigma C} - \left(\frac{\Sigma tC}{\Sigma C}\right)^2 \qquad 2.3.3.$$

$$\sigma^2 = \frac{\sigma t^2}{\bar{t}} \qquad 2.3.4.$$

Where

σt = Standard deviation

σ = The variance of the tracer curve

$\bar{t}$ = Mean or actual detention time

C,t as described in section 2.3.

The variance of a tracer curve and the dispersion number (see section 2.4.) have been recommended to be representative, consistent and reproducible parameters (Thirumurthi, 1969) because they consider all the points on the curve rather than only one or two points on the curve as in case of converntional method. Thus the dispersion number has the strongest statistical probability of correctly describing the hydraulic performance (Layla et al, 1977).

Hirsch (1969) analysed tracer curve using basic concepts of analytical geometry. Figure 2.5. shows his classification of flow on a typical tracer curve. Table 2.3.1. shows some faulty conditions and their effects as revealed by the shape of the tracer curve.

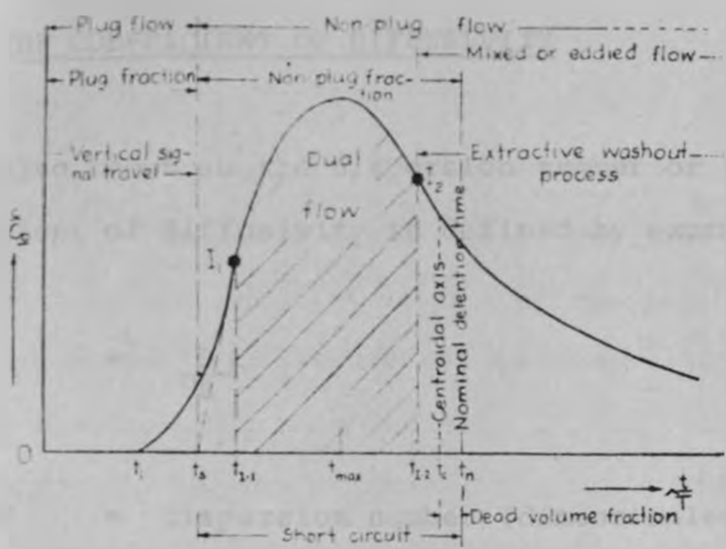


Figure 2.5. <u>TRACER TEST CURVE</u> : Significant Points, Interpretations and Partition of Flow (After Hirsch, 1969).

<u>Note</u>: On the graph,

t_1 = Initial arrival time

t_{max} = Maximum concentration time

t_{I-1} = Time of maximum growth rate of the curve

I_1, I_2 = Points of inflexion for the curve.

Fault	Tracer curve characteristic
Short circuiting	Early initial trace and plug point.
Scattering of tracer incomplete in dosing or lack of simultaneous release to settling zone	Wide interinflectional band, resembling a step type signal.
Recirculation	Bifurcated crest or hump in descending arc.
Dead spaces	Early position of centroid. Usually accompanies short circuiting.
Poor tracer technique and minor structural flaws	Presence of whisker and long smear before upturn; gently sloping ascent; wide angle of dispersion; and broad "interinflectum."

Table 2.3.1. <u>BASIN AND PROCEDURAL DEFIENCIES INDICATED BY TRACER TEST CURVES (AFTER HIRSCH, 1969)</u>.

2.4. THE COEFFICIENT OF DIFFUSIVITY

Also known as the dispersion number or index, the coefficient of diffusivity is defined by expression 2.4.1.

$$d = \frac{D}{UL} \qquad 2.4.1.$$

Where

d = Dispersion number (dimensionless)

D = Axial dispersion coefficient (L^2T^{-1})

U = Fluid velocity (LT^{-1})

L = A characteristic length or size (L).

In stabilisation ponds, L would represent the length of fluid flow in the pond. The values of d vary from zero for plug flow (no diffusion) to infinity for completely mixed flow. Metcalf and Eddy Inc. (1972) report that reactors with mechanical aerators designed to operate as completely mixed system, have probable values of d in the range of 4 to infinity, whereas most stabilisation ponds are somewhere between 0.1 and 2.0.

Levenspiel (1962) has suggested three different equations relating to dispersion Index with variance for open vessels, closed vessels and open-closed or closed-open vessels. For closed vessels as selected for this study,

$$\sigma^2 = 2d - 2d^2 \left(1 - e^{-1/d}\right) \qquad 2.4.2.$$

Hence from expressions 2.3.2, 2.3.3, 2.3.4 and 2.4.2. the value of d can be calculated by trial and error or computer.

The effect of temperature upon d was studied over the range of 12°C to 30°C (54°F to 86°F) by Murphy and Timpany (1967). They found that there was no significant difference of values of d for either short or long detention times.

For a given pond geometry, the increase in hydraulic loading would result into decrease in the value of d.

Since the length of travel path is longer in rectangular ponds of greater length to breadth ratio, the dispersion index would be lowered.

For a given hydraulic loading and the values of Kt, ponds with lower values of d would have higher BOD removal efficiency.

Murphy and Timpany (1967) suggests a scaling up factor of D values. Experimenting with aeration tanks and comparing the full scale prototype tank to 1/13th geometric scale laboratory model, they concluded that:-

(a) A scaled down laboratory model provided good approximation of the dimensionless curve obtained from a full scale aeration tank with the same detention time and air flow. For example D/UL of model was 7.2 whereas the prototype gave D/UL of 6.4 for the same detention time.

(b) The preliminary data suggests that the axial dispersion coefficient for the prototype is related to that of the model by the expression

$$Dp = \frac{Dm}{S^2} = \left(\frac{Lp}{Lm}\right)^2 Dm \qquad 2.4.3.$$

Where the subscripts m and p refer to model and prototype respectively and S is scale factor.

2.5. THE WEHNER AND WILHEM EQUATIONS

For a steady state chemical reactor of the first order reaction kinetics, Wehner and Wilhem (1958) derived the following expression from a reactor of axial dispersion and any kind of entrance and exit conditions.

$$\frac{Ce}{Ci} = \frac{4a\,e^{\frac{1}{2}d}}{(1+a)^2\,e^{a/2d} - (1-a)\,e^{-a/2d}} \qquad 2.5.1.$$

Where

a = $(1 + 4k\bar{t}d)^{\frac{1}{2}}$

Ci = Influent concentration

Ce = Effluent concentration

$\bar{t}$ = Actual detention time

k = First order reaction constant

Commonly known as Wehner and Wilhem equation for chemical reactors, expression 2.5.1. was originally proposed for use in design of chemical reactors with constant quality and quantity of influent and effluent stream with respect to time. The extensive use to incomparate the designs of waste stabilisation ponds has been advocated by Thirumurthi (1969 and Thirumurthi and Nashashibi (1967).

CHAPTER THREE

DESIGN PROCEDURES FOR WASTE STABILISATION PONDS

Early practice was to excavate a suitable area or locate a suitable existing depression and allow the waste water to accumulate. In this way the stabilisation ponds were a means of settling sludge, equalising flow or for odour control. Bacterial or organic removal were not a major concern as is the case for the current methods used in the design.

3.1. AEROBIC PONDS

Aerobic waste stabilisation ponds have not been widely used in Kenya. In practice, successful operation of aerobic ponds system will depend upon maintaining adequate mixing to keep essentially all the biological growth in suspension at all the time. Some aerobic ponds are aerated and mixed by recirculating oxygen-rich waters from the effluent of a facultative or maturation pond system to the influent area. Others are mechanically aerated. Compressed air can also be used in diffused aeration. The retention time can be estimated using expression 3.5.10.

Based on the production of algae, the rate of oxygen production for aerobic processes can be calculated from expression 3.1.1.

$$O = 0.22 \; F.I_L \qquad 3.1.1.$$

Where

O = Rate of oxygen production (Kg/ha/day)

F = Efficiency of light conversion (0.5-6.0%)

I_L = Intensity of light (Langleys/day or Cal/cm^2/day)

The production of algal cells, W, in Kg algae per hectare per day, is also related to the product of F and I_L by the relation:

$$W = 0.125\ F.\ I_L \qquad 3.1.2.$$

The highest percentage BOD removal occurs when the ratio of oxygen produced to the oxygen required is about 1.6. Using this concept, the energy balance equation becomes:

$$\frac{Hc.O'}{F.I_L.R.\frac{H}{10}} = 1.6 \qquad 3.1.3.$$

Where

Hc = Heat of combustion of cells (=6 Cal/mg)

O' = Oxygen production (mg/cm^3)

R = Retention time (days)

H = Depth of pond (m).

If induced mechanical mixing and aeration is employed, the design requires the systematic determination of oxygen supplied by mechanical aeration, oxygen requirement and the power requirement for maintaining completely mixed flow regime.

Gloyna 1971) gives the equation for estimating the oxygen supplied by mechanical aeration as follows:

$$Os = Om.\ \frac{Csw - Cp.}{Cs}\ .\ a\ .\ \theta^{(T-20)}\ .\ F_A \qquad 3.1.4.$$

Where

Os	=	Oxygen supplied by mechanical aeration (Kg O_2/hph)
Om	=	Manufacturer's rating of aerator(usually 1.8 to 2.1 Kg O_2/hph)
Csw	=	Oxygen saturation level in the pond at temperature T (mg/l)
Cp	=	Level of dissolved oxygen in the pond (mg/l)
Cs	=	Oxygen saturation level of distilled water at 20^oC (mg/l).
a	=	Ratio of overall transfer coefficient of waste water to overall transfer coefficient of tap water. (= 0.6 to 1.1)
θ	=	Temperature reaction coefficient (may be taken as 1.02)
F_A	=	Correction factor for altitudes above about 1200 m.

Under complete mixing, the oxygen required, is obtained using the equation 3.1.5.

$$O_n = a' (Li - Le) + b'.X_t \qquad 3.1.5.$$

Where

O_n	=	Oxygen required (mg/l)
a'	=	BOD removed and used to provide energy for growth, as a fraction of ultimate BOD or COD (a' varies between 0.5 and 1.1).
b'	=	Endogenous respiration rate per day, as fraction of ultimate BOD or COD (the values of b' vary from 9/100 to 10/100).
X_t	=	Volatile suspended solids in mixed liquor (mg/l).

For pretreatment purposes, the quantity of biological solids can be maintained at very low levels. In this case the oxygen requirement can be estimated from the modified equation 3.1.6.

$$Od = a'' \, Lr \qquad 3.1.6.$$

Where

Od = Daily oxygen required (Kg)

a" = Coefficient (varies between 0.7 to 1.4)

Lr = BOD removed (Kg/day).

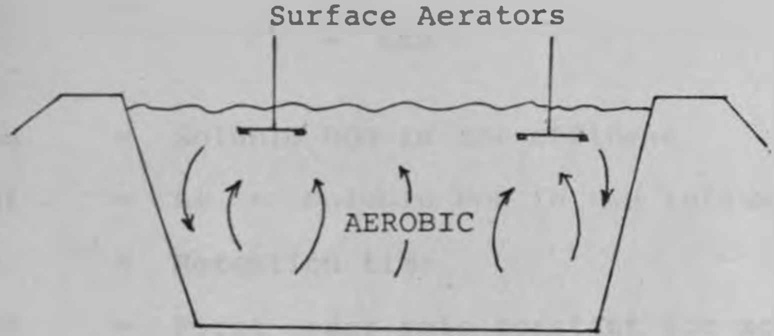
The power of the mechanical aeration equipment required, P, is obtained from the expression:

$$P = \frac{Od}{Os} \qquad 3.1.7.$$

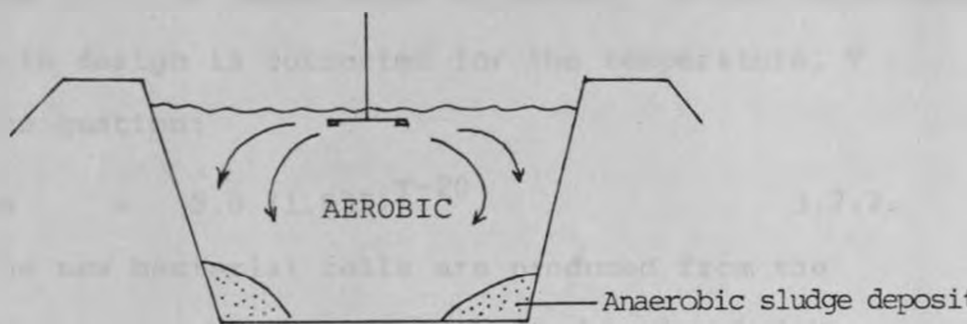
The power requirement will vary with the design of the aerator as well as the geometry of the pond.

3.2. AERATED LAGOONS

In an aerated lagoon, the waste water is treated on a flow through basis. Oxygen is supplied by means of aerators with or without algal photosynthesis. The rising air bubbles from the diffuser keep the contents of the lagoon in suspension. The amount of mixing in the lagoons will result into aerated aerobic lagoon (completely mixed) or aerated aerobic - anaerobic lagoon (with anaerobic sludge deposits) as shown in Figure 3.1.



(i)



(ii)

Figure 3.1. (i) AERATED AEROBIC LAGOON
(ii) AERATED AEROBIC-ANAEROBIC LAGOON
(After Metcalf and Eddy Inc. 1972).

Aerated lagoons have been described to posess microbial characteristics of an activated sludge system rather than those of stabilisation ponds. (Mc Kinney and Edde, 1961). The turbidity and turbulence prevent normal growth of algae.

The design may be based on first order removal of BOD in a completely mixed reactor. Because the aerated lagoons convert the waste water organics to new bacterial cells, it is appropriate in design to consider the soluble or non-bacterial BOD. The removal of BOD in a single aerated lagoon follows the expression:

$$Se = \frac{Si}{1 + Ks.R} \qquad 3.2.1.$$

Where

Se = Soluble BOD in the effluent

Si = Li - Soluble BOD in the influent

R = Retention time

Ks = First order rate constant for soluble BOD removal.

The value of Ks is temperature dependent. A good equation for use in design is corrected for the temperature, T using the quation:

$$Ks = 5.0\ (1.035)^{T-20} \qquad 3.2.2.$$

The new bacterial cells are produced from the oxidised organics according to the expression below:

$$\frac{dx}{dt} = Y.\frac{ds}{dt} \qquad 3.2.3.$$

Where

x = Cells in the aerated lagoon

s = Soluble BOD

Y = Yield coefficient.

Equation 3.2.3. can be rewritten on finite time basis to yield equation 3.2.4.

$$\frac{x.V}{R} = Y.(Si - Se).\frac{V}{R} \qquad 3.2.4.$$

In a lagoon, the synthesized cells will either die off in the lagoon or will be discharged in the effluent. Using this mass balance,

$$Y.(Si-Se).\frac{V}{R} = Q.x + b.x.V \qquad 3.2.5.$$

Where

b = The rate of cell decay

Q = Flow rate of the water

Rearranging equation 3.2.5, the cells in the lagoon are obtained from:

$$x = \frac{Y.(Si - Se)}{b.R + 1} \qquad 3.2.6.$$

But one gram of cells has a BOD of approximately 0.95 gm. Therefore the effluent from the lagoon exerts a BOD given by:

$$Le = Se + 0.95\,x \qquad 3.2.7.$$

From equation 3.2.7. the effluent of an aerated lagoon consists of the fraction of the influent which flows through the lagoon without undergoing oxidation and the cells leaving the lagoon. In practice, the latter can be removed by the polishing pond which usually acts as a settling basin for the solids accompanying the lagoon effluent.

Metcalf and Eddy Inc. (1972) recommends that the oxygen requirement be estimated from the relation:

$$\underset{\text{(lb/day)}}{\text{oxygen required}} = \underset{\text{per day}}{\text{food utilised}} - 1.42 \text{ (organisms wasted per day)}$$

The problem of aerated lagoons operating under extreme temperature conditions should not be neglected. The resulting temperature in the aerated lagoon can be estimated using the relationship suggested by Mancin and Barnhart (1968).

In design, a depth of 1.75 m is usually preferred. The retention time of 5 to 10 days is commonly used.

3.3. ANAEROBIC PONDS

Vincent et al (1963) showed that for tropical and subtropical conditons such as Zambia, the reduction of BOD in anaerobic ponds can be approximated from the expression:

$$Le = \frac{Li}{Kn.\left(\frac{Le}{Li}\right)^{n} .R + 1} \qquad 3.3.1.$$

Where

Li, Le as in section 1.5.

Kn = Design coefficient (Kn = 6 for Zambia)

n = Constant (n = 4.8 for Zambia)

R = Detention time for completely mixed system.

An influent and pond temperature of $20^{o}C$ is assumed.

Oswald (1968) recommended that a basis for scientific pond design (including anaerobic ponds) should select design parameters which will optimise the reactions and the interaction of these reactions so as to bring about optimum waste stabilisation.

Gloyna (1968) observes that the ratio between area of aerobic and anaerobic units appears to be an important design factor. Ratios between 10:1 and 5:1 seem to be fairly useful for purposes of design. Ponds with a low ratio of 3:1 will be sensitive to short term changes in BOD. This concept has been successfully used in Australia.

Mara (1975) reported an emperical design formula relating volumetric loading, V_L, influent flow, Q, the influent BOD and the volume of the pond, V.

$$V_L = \frac{Li.Q}{V} \qquad 3.3.2.$$

The mid-depth area, A, can be obtained from equation 3.3.3.

$$A = \frac{Q.R}{H} \qquad 3.3.3.$$

Where

H = Depth of pond

He also reported that for odour control, experience in Eastern and Southern Africa indicate that a volumetric BOD_5 loading of up to 400 g/m^3/day can be safely applied to anaerobic ponds provided the waste water does not contain a high concentration of sulphates. A concentration of not more than 500 $mgSo_4$=/l was given as optimum.

Gloyna (1971) recommends that desludging is normally necessary when the tank is half-filled with sludge. Based on this concept, the desludging interval m years will be given by the expression:

$$m = \frac{A.H}{2.N.So} \qquad 3.3.4.$$

Where

N = Population served

So = Solids contribution per person per year (0.03 to 0.05 m^3/person/year in Zambia - Vincent et al, 1963).

A liquid detention time of 5 days is recommended for tropical countries. Longer periods tend to render the anaerobic ponds facultative. Less than 5 days detention results into higher risks of odour release coupled with lower BOD removal and higher accumulation of solids.

Loss of heat from the surface and reaerration are restricted by the surface area. The cost of land, cost of construction of cuts and embarkments and topography will influence the depth of the pond. Mara (1975) recommends a depth range of 2m to 4m.

Odour is a common nuisance associated with anaerobic ponds. Odours are a result of escape from the pond surface of reduced volatile substances such as volatile organics, ammonia and hydrogen sulphide. Anaerobic ponds should therefore be designed to foster reactions which will prevent the formation of the reduced substances, to convert them to non-odourous substances as quickly as they are produced or prevent them from escaping (Oswald, 1968).

3.4. MATURATION PONDS

The design of maturation ponds is based on retention time to produce a final effluent with a BOD_5 of less than 25 mg/l and a bacteriological standard of less than 5,000 faecal coliforms per 100 mls. Field experience is that two maturation ponds in series each with retention period of seven days will produce an effluent of less than 25 mg/l (Marais and Shaw, 1961). The rate at which bacteria die off in stabilisation ponds has been given in Chapter One equation 1.4.1.

The same authors showed that the reduction in numbers of faecal bacteria through n ponds in series was given by the relationship:

$$Ne = \frac{Ni}{(1 + K_b R_1)(1 + K_b R_2) \ldots (1+K_b R_n)} \qquad 3.4.1.$$

Where

Ne = Number of faecal coliforms in effluent

Ni = Number of faecal coliforms in influent

K_b = Bacterial removal rate constant

R_n = Retention time in the n^{th} pond.

K_b = 2.0 for Central and Southern Africa.

For design purposes Ni was found to be 4×10^7 faecal coliforms per 100 mls .

Thus if a facultative pond preceeds two maturation ponds all in series, equation 3.4.1. becomes:

$$Ne = \frac{4 \times 10^7}{(1 + K_bR_f)(1 + K_bR_m)^2} \qquad 3.4.2.$$

Where

R_f and R_m refer to facultative and maturation ponds respectively.

Similarly for arrangement:

Anaerobic - facultative - Maturation - Maturation ponds

$$Ne = \frac{4 \times 10^7}{(1 + K_bR_a)(1 + K_bR_f)(1 + K_bR_m)^2} \qquad 3.4.3.$$

Where

Ra refers to aerobic Pond.

Most frequently the maturation ponds take the depth of the facultative pond associated with them (See Section 3.5.).

3.5. FACULTATIVE PONDS

The actual design method of facultative ponds and other types already discussed, will also depend on a great variety of local conditions such as topography, use of effluent, geology and regulations of local authorities. A number of precedures have been used to design facultative ponds. This section, however, discusses the following procedures:

(1) Gloyna's Emperical Procedure

(2) Marais and Shaw Procedure

(3) Procedure based on equating organic load removal and solar energy

(4) Asian Institute of Technology (A I T) Emperical Procedure

(5) Procedure suggested by Thirumurthi.

3.5.1. Gloyna's Emperical Procedure

Herman and Gloyna (1958) emphasized the influence of temperature on retention time to achieve 80% to 90% BOD reduction. An optimum pond operation temperature of 35°C was found to result into BOD reduction of 90% through retention time of 3.5 days. It was then postulated that at temperature, T, for the same reduction of BOD, the retention time, R_T, is given by Arrhenius law:

$$R_T = R_{35}\ \theta^{(35-T)} \qquad 3.5.1.$$

Where

R_{35} = Retention time at 35°C

θ = Temperature reaction constant (θ = 1.072).

Therefore equation 3.5.1. can be rewritten as:

$$R_T = 3.5\ (1.072)^{35-T} \qquad 3.5.2.$$

Statistical analysis of mean BOD for U.S.A. raw domestic waste water approximates to 200 mg/l. The 80% to 90% BOD reduction gives effluent of 20 to 40 mg/l. In order to keep the effluent BOD within these limits, retention time for the influent BOD greater or less than 200 mg/l would need to be adjusted by the factor $\frac{Li}{200}$ giving;

$$R_T = \frac{Li}{200} \cdot R_{35}\, \theta^{(35-T)}$$

$$= \frac{Li}{200} \cdot (3.5)\,(1.072)^{35-T} \qquad 3.5.3.$$

The volume of pond, $V = Q.R_T$ 3.5.4.

Where

$Q = N.q$

and q = Per capita waste water flow (litres/day)

N = Population served.

Substituting for R_T in equation 3.5.4.

$$V = N.q \cdot \frac{Li}{200} \cdot 3.5 \times 10^{-3} \cdot (1.072)^{35-T} \qquad 3.5.5.$$

The recommended depth was 0.61 m to 1.06 m.

Introducing a coefficient for toxicity, and recommending a depth of 1.82 m, Huang and Gloyna (1968) modified equation 3.5.5. into:

$$V = N.q \cdot \frac{Li}{200} \cdot 7.0 \times 10^{-3}\,(1.082)^{35-T} \qquad 3.5.6.$$

They also suggested that for tropical conditions the coefficient 7.0 should be reduced by a factor of 2.

Gloyna (1971) also recommends a value of 7 days for R_{35} and a temperature coefficient of 1.085 giving a metric expression of the form:

$$V = 3.5 \times 10^{-5} N.q.Li\,(1.085)^{35-Tm} \qquad 3.5.7.$$

Where

Li which appeared to be BOD_5 initially should be understood as the ultimate BOD for raw waste water. In case of settled waste waters Li may be taken as five-day BOD.

Tm = Avarage water temperature of coldest month (^{o}C).

Gloyna (1971) solves equation 3.5.7. and gives a graph as shown in Figure 3.1. The choice of depth is influenced by environmental conditions, type of waste to be treated, and by general safety factors desired. Table 3.1. gives the recommended depths.

Recommended depth (m)	Environmental conditons and type of waste-water.
1.0	Uniform warm temperature, presettled waste-water
1.0 to 1.5	Uniform warm temperature, untreated waste-water
1.5 to 2.0	Moderate seasonal temperature flactuations; raw waste-water containing settleable solds
2.0 to 3.0	Wide seasonal temperature variations, large amounts of settleable grit or settleable solids

Table 3.1. <u>RECOMMENDED DEPTH OF FACULTATIVE PONDS IN RELATION TO ENVIRONMENTAL CONDITIONS AND TYPE OF WASTE</u>. (After Gloyna, 1971).

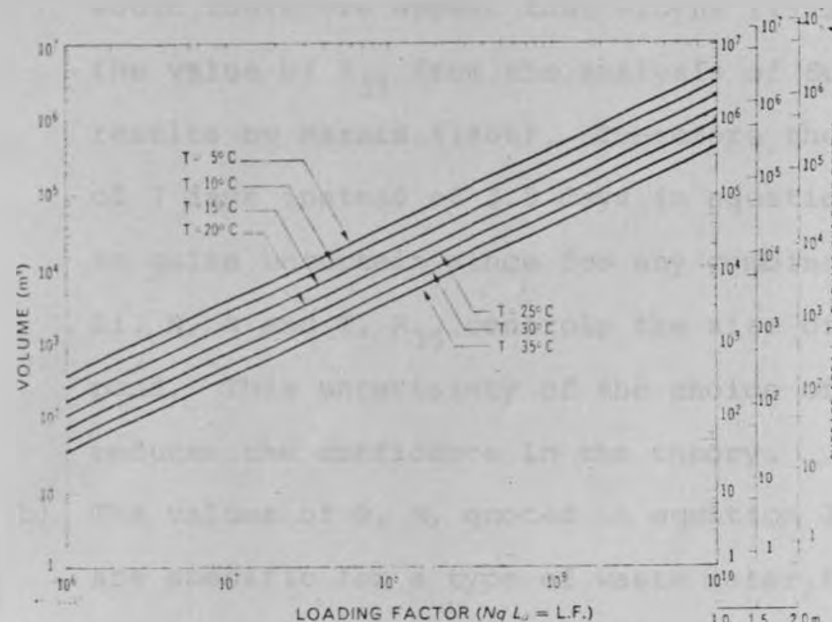


Figure 3.1. <u>GRAPH FOR CALCULATING THE REQUIRED VOLUME AND SURFACE AREA OF A FACULTATIVE POND.</u> (After Gloyna, 1971).

Use of Figure 3.1. should also observe the following:

(a) Increase of volume and surface area for wastes containing more than 500 So_4= mg/l.

(b) Increase of volume for some industrial wastes which may be toxic to algae or reduce chlorophyll

(c) Increase of surface area where there are prolonged cloudy weathers.

3.5.1.1. <u>Shortcomings of the Procedure</u>

The following disadvantages of Gloyna's procedure preclude its use.

(a) Marais (1966) analysed Suwannakarn's (1963) experimental results and gave the values of θ and K_{35} to be 1.085 and 1.2 respectively. It would therefore appear that Gloyna (1971) adopts the value of R_{35} from the analysis of Suwannakarn's results by Marais (1966). Therefore the choice of 7 days instead of 3.5 days in equation 3.5.7. is quite uncertain since for any combination of Li, N, q and T, R_{35} controls the size of the pond. This uncertainty of the choice of R_{35} reduces the confidence in the theory.

(b) The values of θ, R, quoted in equation 3.5.7. are specific for a type of waste water, temperature and mixing. They may not be reproducible for any change of the above conditions. For any combinations of N, q, Li and T, the volume of the pond will be controlled by θ. For this reason θ used should be very reliable for design conditions.

(c) The design method is mainly based on model test results where conditions are easily controlled. In the field, variation of local conditions and presence of sludge may present a different situation. The waste water received in the field may also vary in composition. All the above will affect the value of K as well as θ. The procedure does not seem to take into account possible effect of changing field conditions on K or θ.

(d) Gloyna (1971) does not give a justification for using ultimate BOD in some cases and BOD_5 in others for Li.

(e) The procedure neglects reduction of bacteria and prevention of possible odours from the ponds.

3.5.2. Marais and Shaw Procedure

Report No 9 (1973) recommends that facultative ponds in Kenya should be designed using the modified Marais and Shaw procedure.

Marais and Shaw (1961) advanced a method for design of waste stabilisation ponds in the tropics. They considered each pond in isolation taking into account influent and effluent flows, concentration of pollution in the pond and its degredation and the volume of pond. The following assumptions were made by Marais and Shaw (1961):

(a) that degredation of waste water organics occurs by first order reaction kinetics.

(b) that mixing is complete and instantaneous so that the contents of the pond equals the contents of the effluent.

(c) that the degredation of organics is not temperature dependent.

As reported by Marais (1966) the differential equation for the concentration of the organic matter used by Marais and Shaw (1961) was:

$$\frac{dL}{dt} + \left(K + \frac{Qe}{V}\right) Le = Li.\frac{Qi}{V} \qquad 3.5.8.$$

Where

Qe = Effluent flow per day

Qi = Influent flow per day

Le, Li as in Section 1.5. but can also be understood as the bacterial pollution.

For completely mixed flow, Qe = Qi so that $\frac{Qe}{V} = \frac{Qi}{V} = \frac{1}{R}$.

Equation 3.5.8. can be simplified into:

$$\frac{dL}{dt} + \left(K + \frac{1}{R}\right) Le = \frac{Li}{R} \qquad 3.5.9.$$

Where

R = Retention time for completely mixed conditions.

Under equilibrium, $\frac{dL}{dt} = 0$. For constant Le, R and K, solving equation 3.5.9. for $\frac{dL}{dt} = 0$ gives:

$$Le = \frac{Li}{1 + KR} \qquad 3.5.10$$

For series of ponds, the equilbrium concentration of BOD can be calculated progressively down the series as:

$$L_1 = \frac{Le}{1 + KR_1}, \quad L_2 = \frac{L_1}{1 + KR_2}$$

Thus generally

$$Le_n = \frac{Le_{(n-1)}}{1 + KR_n} \qquad 3.5.11.$$

For the same R for each pond

$$Le_n = \frac{Li}{(1 + KR)^n} \qquad 3.5.12.$$

From equation 3.5.12 a series of ponds is more efficient than a single pond for the same retention time. The efficiency would increase with the number of ponds until, with an infinite number, the upper limit of theoritical efficiency is reached. This limit is equivalent to piston flow where:

$$L = Lo\, e^{-KR} \qquad 3.5.13.$$

A similar expression was developed for bacterial removal.

The maximum value of Le consistent with the maintainance of aerobic conditions in ponds in South Africa was found to be related to the depth, H,in metres.

$$Le = \frac{1000}{2H + 8} \qquad 3.5.14.$$

Marais and Shaw (1961) later reduced the numerator to 750 for design purposes. Based on long term behaviour of ponds, Meiring et al (1968) reduced this value still further to 600. Marais (1970) later recommended 700. Report No 9 (1973) recommends that facultative ponds in Kenya be designed with depth of 1.75m (5.7 ft) using the denominator 600 hence

$$Le = \frac{600}{2H + 8} \qquad 3.5.15.$$

In South Africa, Rhodesia and Zambia, the value of K was found to be 0.23 day^{-1} but was later reduced to 0.17 day^{-1} for design purposes by Marais and Shaw (1961). Thus by setting K at 0.17 day^{-1} with Le, and Li known, R can be calculated by using equation 3.5.10. The mid-depth area, A, can then be obtained from equation 3.2.3.

Marais (1966) subsequently modified equation 3.5.10. by allowing K, at any temperature T, K_T, to be related to the value of K at $35^{\circ}C$ which was found to be 1.2 day^{-1}. The relationship being:

$$K_T = 1.2\ (1.085)^{T-35} \qquad 3.5.16.$$

This was an attempt to integrate the Herman and Gloyna (1968) and Marais and Shaw (1961) procedures by allowing:

$$\frac{K_{35}}{K_T} = \theta^{35-T} = \frac{R_T}{R_{35}} \qquad 3.5.17.$$

Substituting equation 3.5.16 into 3.5.10

$$Le = \frac{Li}{1 + 1.2\ (1.085)^{35-T}.R} \qquad 3.5.18.$$

Mara (1975) gives a variation of K with temperature of the form:

$$K_T = 0.30\ (1.05)^{T-20} \qquad 3.5.19.$$

and states that although the equation 3.5.19. is probably rather conservative, it nevertheless gives values which are very close to those used by design engineers in Kenya and its use is therefore recommended.

Mara (1974) showed that using expression 3.5.10. there is little variation in R for values of Le of 50, 55 and 60 mg/l. He also reasoned that since Marais and Shaw (1968) recommends Le of not less than 50 mg/l, a value of 55 mg/l was proposed for Le. Substituting V for A.D. Mara (1974) states the design equations for the depth range 1.0 to 1.5m as follows:

$$R = \left(\frac{Li}{55} - 1\right).\ \frac{1}{K} \qquad 3.5.20.$$

$$A = \frac{Q(Li - 55)}{55\ K.H} \qquad 3.5.21.$$

3.5.2.1. <u>Shortcomings of the Procedure</u>

(a) The assumptions made by Marais and Shaw (1961) cannot be fully justified for the field ponds. Mixing of the influent with the pond contents is inhibited by stratification thus inducing anaerobic conditions in the lower depths of the pond. However, as pointed out by Marais (1975), by assuming ponds to be completely mixed, provides a factor of safety in design because the pond volume for perfect mixing is larger than that of imperfect mixing found in practice. It should also be borne in mind, however, that unnecessary large volumes of ponds increases the total cost of construction.

In waste waters of complex composition it is unlikely that the breakdown rate remains constant. BOD removal as shown by laboratory tests occurs in two stages - the carbonaceous and the nitrogeneous. Gann et al (1968) found out that the principal site of bacterial activity was located within the vicinity of the influent. It would therefore appear that near the influent end, large amounts of compounds which are easily broken down will be decomposed quickly. The remaining matter which could be more resistant to biological degredation will be broken down more slowly. It would also appear that for a finite element of waste water, flowing through the pond, the rate of degradation decreases with time.

(b) The basis for modification of the procedure by Marais (1966) was through analysis of Suwannakarn's (1963) results. The derivation of equation 3.5.16. is therefore valid under the experimental conditions among which were:

(i) a water soluble milk compound as a degradable influent. Only insignificant portion of the waste settled out as sludge.

(ii) Constant temperature throughout the experimental period.

(iii) All ponds remained aerobic except one.

In practice, anaerobic decomposition occurs on the settled BOD and cyclic temperature variations affects the performance of the ponds. Under these deviations, it is unlikely that the field variation of K with temperature will follow the stated relationship.

(c) The geometry, shape, deadspaces, inlet and outlet arrangements of a pond will affect its hydraulic properties hence its performance. These parameters were not given a proper treatment in the design procedure.

3.5.3. Procedure Based on Equating Organic Load Removal and Solar Radiation

The basis of this procedure is the relation between daily BOD removal and the daily oxygen production by algae in a pond. As earlier discussed in Section 1.4.2. for a type of algae the amount of oxygen produced will largely depend on the intensity of solar radiation. About 6% of incident solar energy is utilised by algae, while for each kilogram of algae produced, about 6×10^6 calories are utilised. Hence based on the above assumptions the weight of algae produced can be approximated from:

$$W = \frac{I_L \times 10^8 (\text{cal/ha/day}) \times 0.06 \text{ (efficiency)}}{6 \times 10^6 \text{ (cal/Kg)}} \qquad 3.5.22.$$

Where

W = Weight of algae (Kg/ha/day)

I_L = Intensity of solar radiation (langleys/day or $\text{cal/cm}^2\text{/day}$).

With further assumptions that for every kilogram of algae grown, about 1.6 Kg. of oxygen are photosynthetically produced, the amount of oxygen produced in a pond will be given by:

$$O = 1.6 \cdot I_L \qquad 3.5.23.$$

Where

O = Rate of oxygen production (Kg/ha/day).

Joyangoudar et al (1970) equates the oxygen production and the removal of ultimate BOD in a pond. Oswald and Gotaas (1957) uses a relatively conservative criterion of equating the oxygen production with the ultimate BOD applied (rather than removed) per ha/day. Thus equation 3.5.23 can be written as:

$$BOD_{ult.} = 1.6 . I_L \qquad 3.5.24.$$

Where

$BOD_{ult.}$ = Ultimate BOD applied per hectare per day.

Rewriting equation 3.5.24 in terms of BOD_5 loading (Kg/ha/day), L_s.

$$L_s = 1.065 . I_L \qquad 3.5.25.$$

The mid-depth area, A, can be calculated from equation 3.5.26.

$$A = \frac{10\ Li \cdot Q}{L_s} \qquad 3.5.26.$$

Where

A is in m^2, Li in mg/l, Q in m^3/day and L_s in Kg/ha/day.

3.5.3.1. <u>The Shortcomings of the Method</u>

The oxygen produced in a pond by algae may be used by bacteria to stabilise the waste while some of the oxygen may be lost to the surroundings. Equations 3.5.24.

and 3.5.26. assume that all the oxygen produced by algae is available for stabilisation of the influent waste. To account for some of the oxygen lost to the surroundings, perhaps a modification would be to introduce a coefficient β to take care of unavailability of oxygen for waste stabilisation.

Then

$$L_s = \beta I_L$$

Where

$$\beta < 1.065.$$

Unfortunately there has been conflicting results of the coefficient showing that both β and I_L vary from place to place (Abstract, 1966). Therefore the procedure may not be reliably used for the design of facultative ponds unless I_L and β have been correctly determined for the particular place in consideration.

In waste waters of appreciable undissolved solid content, sludge formation at the bottom of ponds is prominent. In such a case some BOD is removed through anaerobic digestion in the sludge zone. Some BOD is also lost in the facultative zone. Equation 3.5.24., however, assumes homogeneous aerobic conditions which do not occur in facultative ponds.

3.5.4. The Asian Institute of Technology (AIT) Emperical Procedure

This procedure relates areal five-day BOD removal with areal five-day BOD loading both expressed in Kg/ha/day.

Mc Garry and Pescod (1970) working at the Asian Institute of Technology analysed operational results of various lagoons in different climates and deduced an emperical relationship:

$$L'r = 0.725 L_s + 10.25 \qquad 3.5.27.$$

Where

$L'r$ = areal five-day BOD removal

L_s = areal five-day BOD loading

Based on the lower 95% confidence limit of the data,

$$L'r = 0.725 L_s - 23.15 \qquad 3.5.28.$$

They also found that the maximum loading that a pond could operate under before reaching failure was controlled by temperature of the air according to the relation:

$$Ls = 11.2 (1.054)^T \qquad 3.5.29.$$

Where

T = Air temperature in ^oF.

The mid-depth area of a pond would be obtained using equation 3.3.3. after obtaining Ls from equation 3.5.29. The effluent BOD_5 would be calculated from equation 3.5.28.

The main disadvantage of this procedure seem to lie in the way the relation 3.5.29. was obtained. From Figure 3.2. it is evident that there is somehow lack of comformity to the fitted curve for the areal BOD of less than 200 lb/acre/day (224 Kg/ha/day). As can be deduced from equation 3.5.28, Ls will influence L'r and subsequently the volume of the pond. Therefore an error in Ls will affect the design volume of the pond.

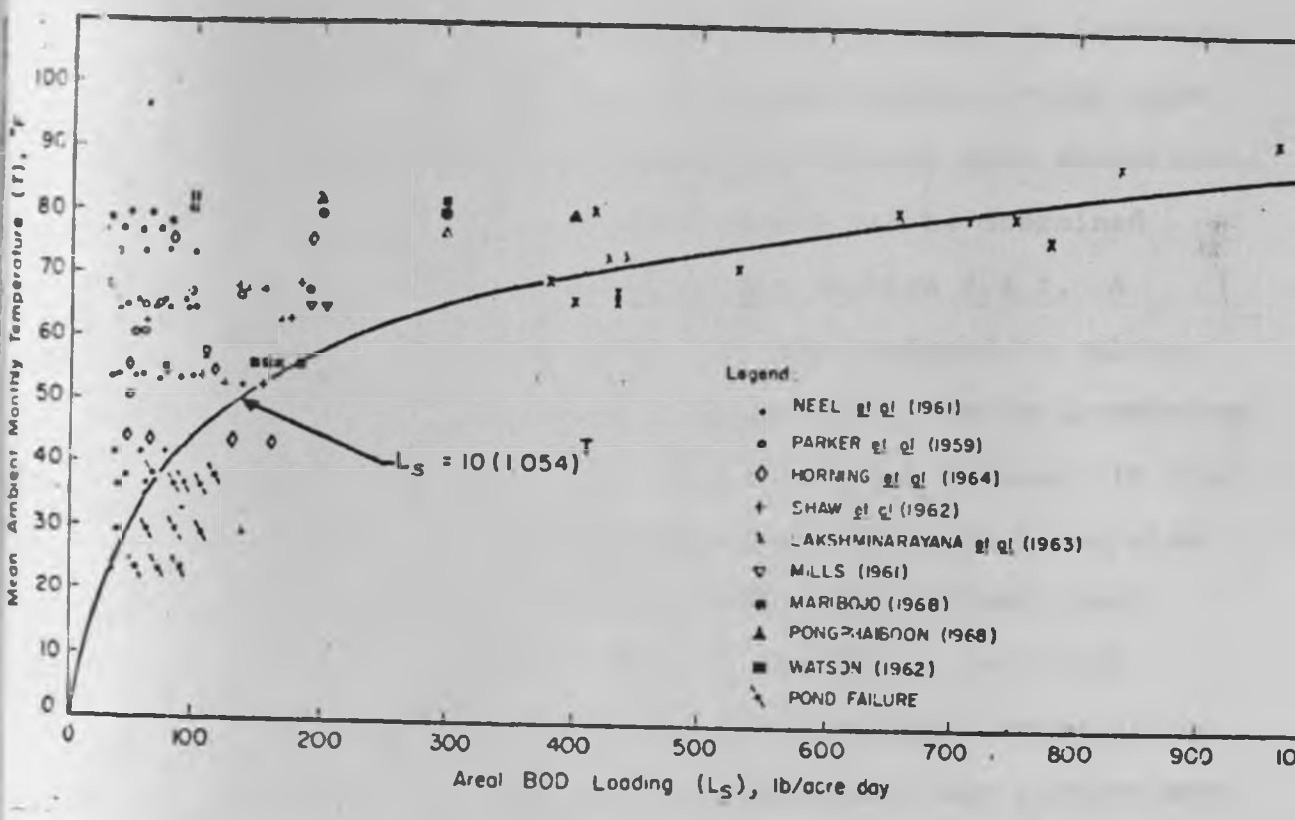


FIGURE 3.2. MAXIMUM BOD LOADING AS A FUNCTION OF AMBIENT TEMPERATURE.

(Mc Garry and Pescod, 1970)

3.5.5. Procedure Suggested by Thirumurthi

Thirumurthi (1969) advanced a design procedure for facultative ponds based on Wilhem and Wehner (1956) equation of chemical reactors. The equation 2.5.1. given in Section 2.5. assumes the following for stabilisation ponds:

(a) A non-ideal flow system

(b) Biochemical reactors whose BOD removal follows first order kinetics.

In his procedure, Thirumurthi introduces a dispersion index, d, a parameter which is affected by pond shape, short-circuiting, deadspaces, inlet and outlet arrangement. The author also suggested that in order to determine the value of d, scaled down laboratory models can be used. With the help of tracer studies, the actual mean detention time and variance of the tracer curve can be dtermined from which d can be calculated (See Section 2.4). A design chart shown in Figure 3.3 was produced by solving equation 2.5.1. using common values of d. Thus for a percentage removal of BOD required, a value of Kt can be read off from the chart. With known influent BOD a value of K can also be interpolated from another chart obtained from model studies. Alternatively K can be found from the field studies. The correction of K to the required temperature enables detention time to be solved, hence the surface area and depth of the pond.

Figure 3.3. <u>DESIGN FORMULA CHART</u>

(After Thirumurthi, 1969).

The main limitation of this procedure is that percolation, rainfall and effect of wind will affect the performance of ponds in the field. These factors are difficult to simulate in the laboratory studies. However, an introduction of carefully determined correction factors to cater for the above may present a solution.

CHAPTER FOUR

OBJECTIVE OF THE STUDY, MATERIALS, EQUIPMENT AND EXPERIMENTAL PROCEDURES

4.1. OBJECTIVE OF THE STUDY

The mixing characteristics of ponds have been discussed in Chapter Two under hydraulic properties of waste stabilisation ponds. These characteristics can be represented by the coefficient of diffusivity, d. The same characteristics will affect both molecular and turbulent diffusivity and detention time. They will also ultimately affect the BOD removal efficiency and hence the the first order BOD removal constant.

Therefore the objectives of the study were:

(i) To study the effect of length, breadth and depth on the value of d.

(ii) To study the effect of inlet arrangement on the value of d for a given hydraulic loading.

(iii) To study the effect of hydraulic loading on the value of d for a given pond geometry.

(iv) To evaluate the values of K from BOD reduction experimental results and compare the values with those reported in the literature.

4.2. MODEL PONDS

Glass rectangular tanks were used as laboratory model waste stabilisation ponds. Four ponds were made from 6 mm clear perspex sheeting by sealing the butt joints with chloroform. Each tank was made to achieve a different length to breadth ratio. As shown in Table 4.1. and Figure 4.1. the depth of each model pond also varied.

POND	SIZE (mm) l x b x h (Fig.4.1)	LENGTH TO BREADTH RATIO
1	900 x 540 x 150	1.7/1.0
2	1500 x 500 x 200	3.0/1.0
3	1720 x 430 x 275	4.0/1.0
4	1800 x 360 x 350	5.0/1.0

Table 4.1. <u>DIFFERENT SIZES OF MODELS USED</u>

The outlet was made of 6 mm diameter soft transparent poly-vinyl chloride (PVC) tubing fixed through a 6 mm diameter hole at one end of the model pond. Aldarite Cement was used for fixing the tubing into perspex glass. The effluent from the model ponds through the tubing was collected in buckets (Figure 4.3.) and was disposed of everyday.

To achieve a liquid depth of 130 mm for every pond, ponds 2, 3 and 4 were provided with an outlet tubing at a level of 130 mm from the bottom as shown in Figure 4.1. This outlet could be closed using a clip to obtain the full depth liquid.

Perspex glass struts, 50 mm in breadth, were fastened along the breadth of model ponds to provide extra strength so that the ponds' walls do not bulge which could result into failure.

An elevated circular metallic supply tank of approximately 157 litres capacity (Figure 4.4.) was used as a main

Note: Drawings not to scale, dimensions in mm.

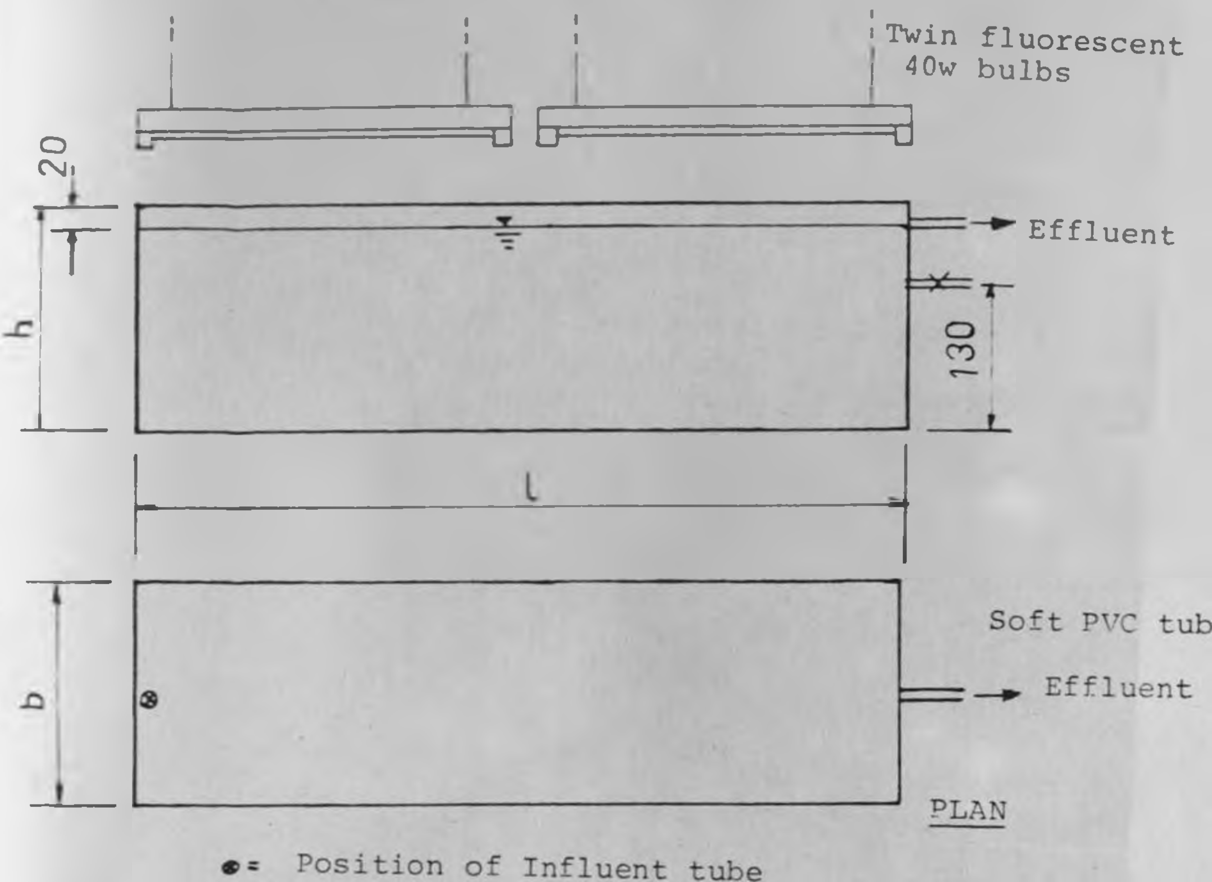


Figure 4.1. TYPICAL MODEL TANK

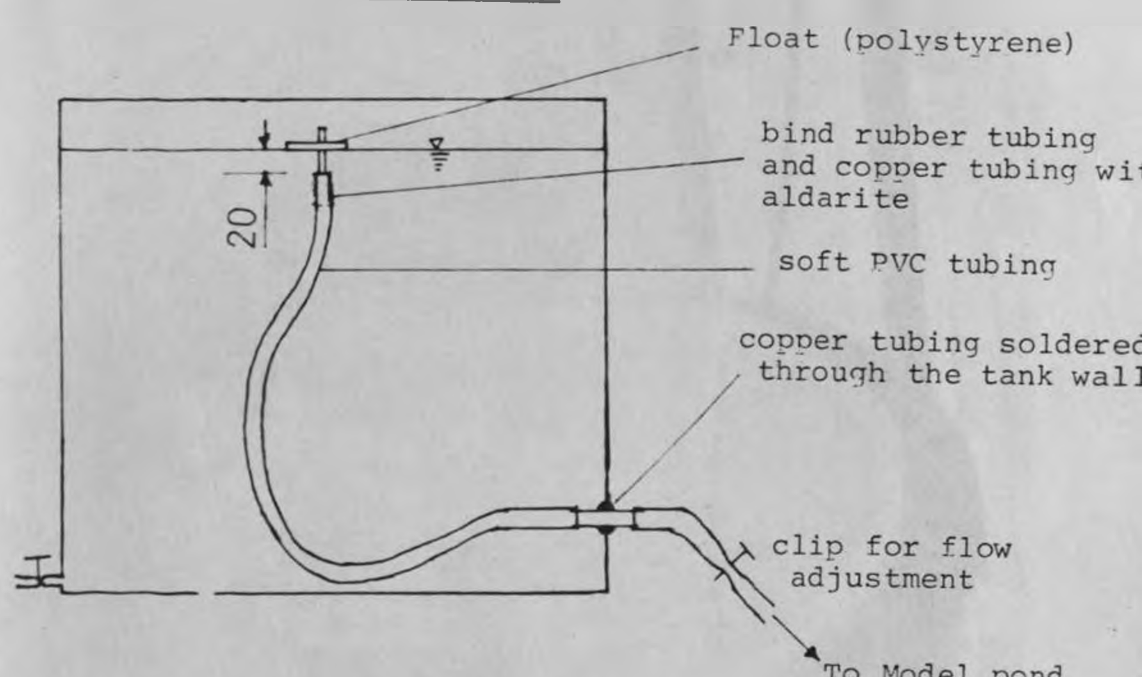


Figure 4.2. SECTION THROUGH THE ELEVATED SUPPLY TANK

Figure 4.3. COLLECTION OF EFFLUENT FROM MODEL PONDS

supply tank for water and synthetic sewage. As shown in Figure 4.2., a float was used to achieve a head of liquid of 20 mm above the 4 mm diameter tubing ensuring averagely a constant flow rate irrespective of the level of the liquid in the tank.

Each model pond was fed from the overhead tank through a separate 4 mm diameter PVC tubing as shown in Figure 4.4.

4.3. TRACER STUDY EXPERIMENTS

The inlet arrangement chosen was a simple inlet PVC tubing, leading from the overhead supply tank, dipping into the liquid in the pond at the middle half of the breadth (See Figure 4.1.). The influent tube was positioned at about 30 mm from wall and 30 mm deep below the liquid surface.

Analytical Sodium Chloride was chosen as a tracer.

4.3.1. Experimental Procedure

Test 1

A hydraulic loading of 2.66×10^{-2} m^3/m^2/day (28,000 gal/acre/day) was chosen; giving theoritical detention periods of 4.9, 6.8, 9.6 and 12.4 days for ponds 1, 2, 3, and 4 respectively running at full liquid depth. The influent flows were 539, 831, 820 and 719 mls per hour for ponds 1, 2, 3 and 4 respectively.

The cleaned model ponds and the supply tank were filled with tap water. The flows were adjusted for each model pond to achieve the hydraulic loading. The flow was left for overnight to stabilise.

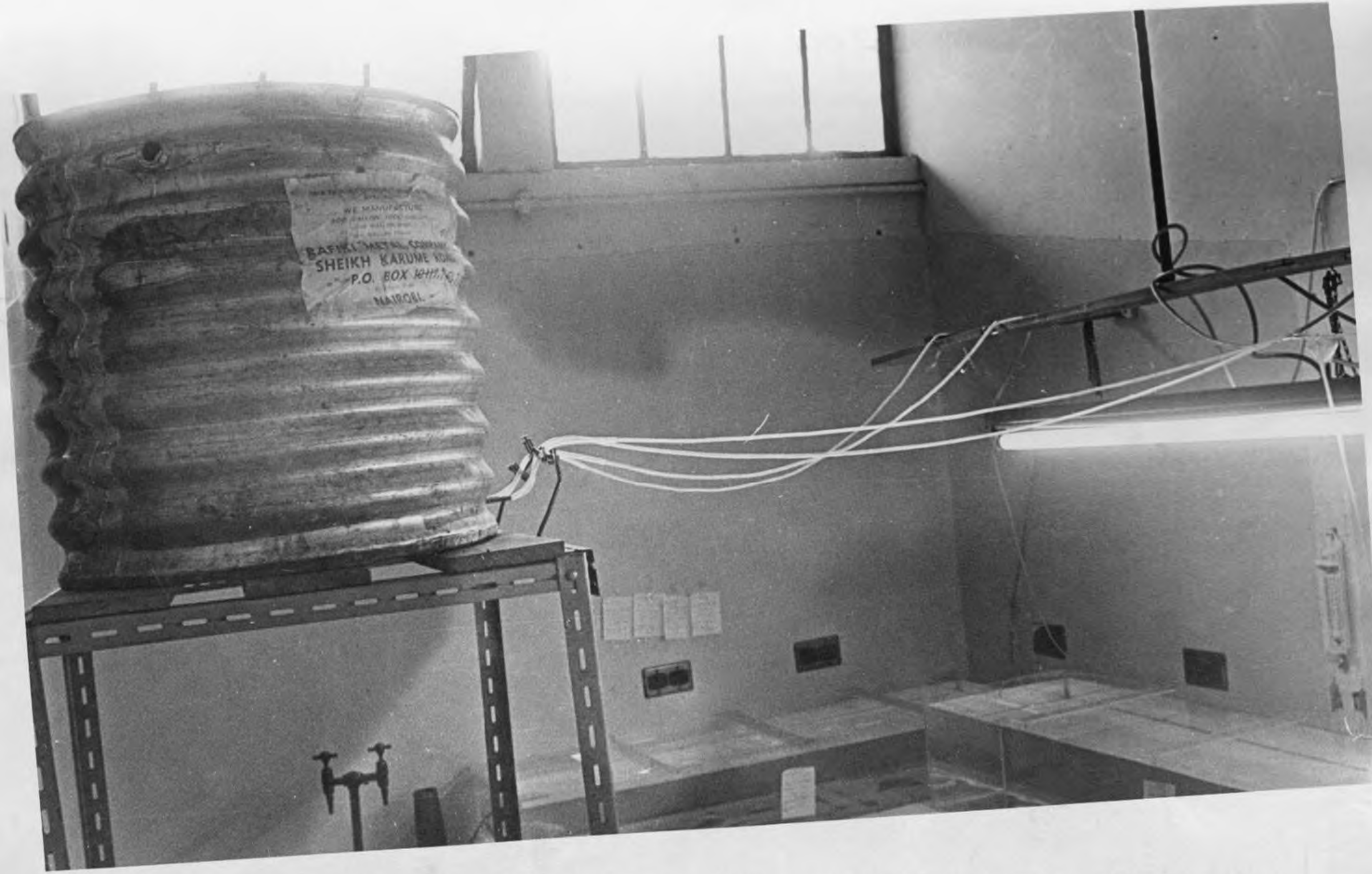


Figure 4.4. <u>THE SUPPLY TANK AND INFLUENT TUBES LEADING INTO MODEL PONDS</u>

A 30% concentration solution of the tracer was made by dissolving 150 gm of the sodium chloride into 500 mls of distilled water at room temperature. 50, 100, 150, and 200 mls of the solution were added into ponds 1, 2, 3 and 4 respectively. The tracer was added at the position of the influent tube. A filter funnel was used to introduce the tracer.

Daily effluent samples for each pond were collected in the morning between 900 hours and 1000 hours. Sampling was done by collecting effluent flow through the effluent tube. Analysis for concentration of chloride in the effluent was done using argentometric method as described in the Standard Methods (1971). At least two titration trials were done and an average taken as reported in Chapter Five. The concentration of the tracer is expressed as the volume (mls) of the titrant ($AgNo_3$) used.

The supply tank was filled with tap water daily. Influent and effluent flows for each pond were checked everyday. Where necessary, the influent flows were readjusted to achieve the hydraulic loading. The influent flow decreased as the supply tank emptied. In extreme cases, 29% reduction in flow was observed although the average was around 19%.

The test was continued as long as quantitatively the concentration of the tracer in the exit stream (effluent) was greater than the initial concentration. The initial concentration being the concentration of chlorides in the tap water.

Test 2

A liquid depth of 130 mm at a hydraulic loading of 2.66×10^{-2} m^3/m^2/day (28,000 gal/acre/day) for each model pond were chosen. The theoritical detention periods were 4.9 days for ponds 2 and 4, and 5.0 days for ponds 3. This test excluded Pond 1 since it would be repeating Test 1. The general procedure, tracing technique, sampling and analysis were the same as in test 1.

Test 3

Hydraulic loading of 3.42×10^{-2} m^3/m^2/day (36,000 gal/acre/day) at full depth liquid in each model pond was chosen. The theoritical detention time for ponds 1, 2, 3 and 4 were 3.8, 5.3, 7.5 and 9.7 days respectively. The influent flows were adjusted to 693, 1069, 1054 and 924 mls per hour in the same above order. The procedure outlined for test 1 was followed.

Test 4

The duration of a test was different for each pond as could be predicted from the theoritical detention periods. Whereas tests on Ponds 2, 3 and 4 could take a month or more (See Chapter Five), Pond 1 tests took much shorter. As a result, hydraulic loading 3.8×10^{-2} m^3/m^2/day (44,000 gal/gal/acre/day) and 4.56×10^{-2} m^3/m^2/day (48,000 gal/acre/day) were chosen and tracer studies carried out on Pond 1. The corresponding detention times and flows were 3.1 and 2.9 days and 846 and 923 mls/hr. The procedure remained the same.

For Pond 1, influent position was changed to a corner of the influent breadth end and a tracer study was carried out. The hydraulic loading of 3.42×10^{-2} m^3/m^2/day

(36,000 gal/acre/day) was chosen.

4.4. BOD REDUCTION EXPERIMENTS

The inlet arrangement as reported in Section 4.3. was used. Two tests were carried out for each model pond. A hydraulic loading of 2.66 x $10^{-2} m^3/m^2$/day (28,000 gal/acre/day) was chosen. Test 1 lasted for 19 days after ponds had turned anaerobic. Test 2 lasted for 45 days after obtaining a steady state. Synthetic sewage (Section 4.4.1.) was used.

4.4.1. Synthetic Sewage

Table 4.2. shows the composition of the synthetic chemical sewage used for influent feed for all ponds. The composition was designed basing on that used by Parpia (1973).

Substance	gm in 100 litres of tap water	
	Test 1	Test 2
D-Glucose (AnalAR)	36	12.7
NH_4HCO_3	30	10.6
Tryptone Water	25	8.9
Lab Lemco powder	12	4.30
$CaCl_2.2H_2O$	2.7	0.96
$ZnSO_4$	2.5	0.89
K_2HPO_4	1.8	0.64
KH_2PO_4	1.4	0.50
$MgSO_4.7H_2O$	1.0	0.35
$COCl_2.6H_2O$	0.08	0.028
$CuSO_4.5H_2O$	0.06	0.021
$NaMoO_4.2H_2O$	0.06	0.021
$MnSO_4.7H_2O$	0.06	0.021
$FeSO_4.7H_2O$	0.050	0.018
N/50 EDTA	50 mls	17.7 mls

Table 4.2. COMPOSITION OF SYNTHETIC SEWAGE

The composition of test 1 gave an initial average values of BOD_5 and COD of 320 and 620 mg/l respectively. The pH was 7.2. The test 2 composition gave the corresponding values of 110 and 220 mg/l respectively and a pH of 7.2.

4.4.2. Experimental Procedure

The model ponds were housed in a laboratory room 4½ m x 2½ m thus cutting away natural light. Artificial lighting was provided using twin-fluorescent lamps 1.2 m (48") long, 40 w day light as shown in Figure 4.1. and Figure 4.5. The average total radiant energy at the top surface of the ponds was 6.46×10^3 ergs/cm^2 sec (155 ft-c or 6.1 91 gram-cal/cm^2 per 12 hrs. The lights were switched on and off at 7.00 o'clock in the morning and 7.00 o'clock in the evening respectively using an automatic on/off switch. A maximum-minimum thermometer hanging on a wall within the room was used to measure the daily maximum and minimum air temperature of the room.

Test 1

The model ponds were washed clean and filled as follows: 55% tap water, 40% synthetic sewage of composition Test 1 in Table 4.2. and 5% seeding where the percentages are in volumes. The seeding was obtained from the effluent of an existing secondary pond serving St. Aquinas Seminary - Langata (Nairobi). The use of the seed was to provide initial biological life.

The overhead supply tank was filled with synthetic sewage of composition as described in Section 4.4.1. As in case of tracer studies, the sewage in the tank was the influent for all model ponds.

The flows were adjusted for each model pond to achieve the hydraulic loading. The initial BOD_5, COD for each model pond and influent were determined as described in the

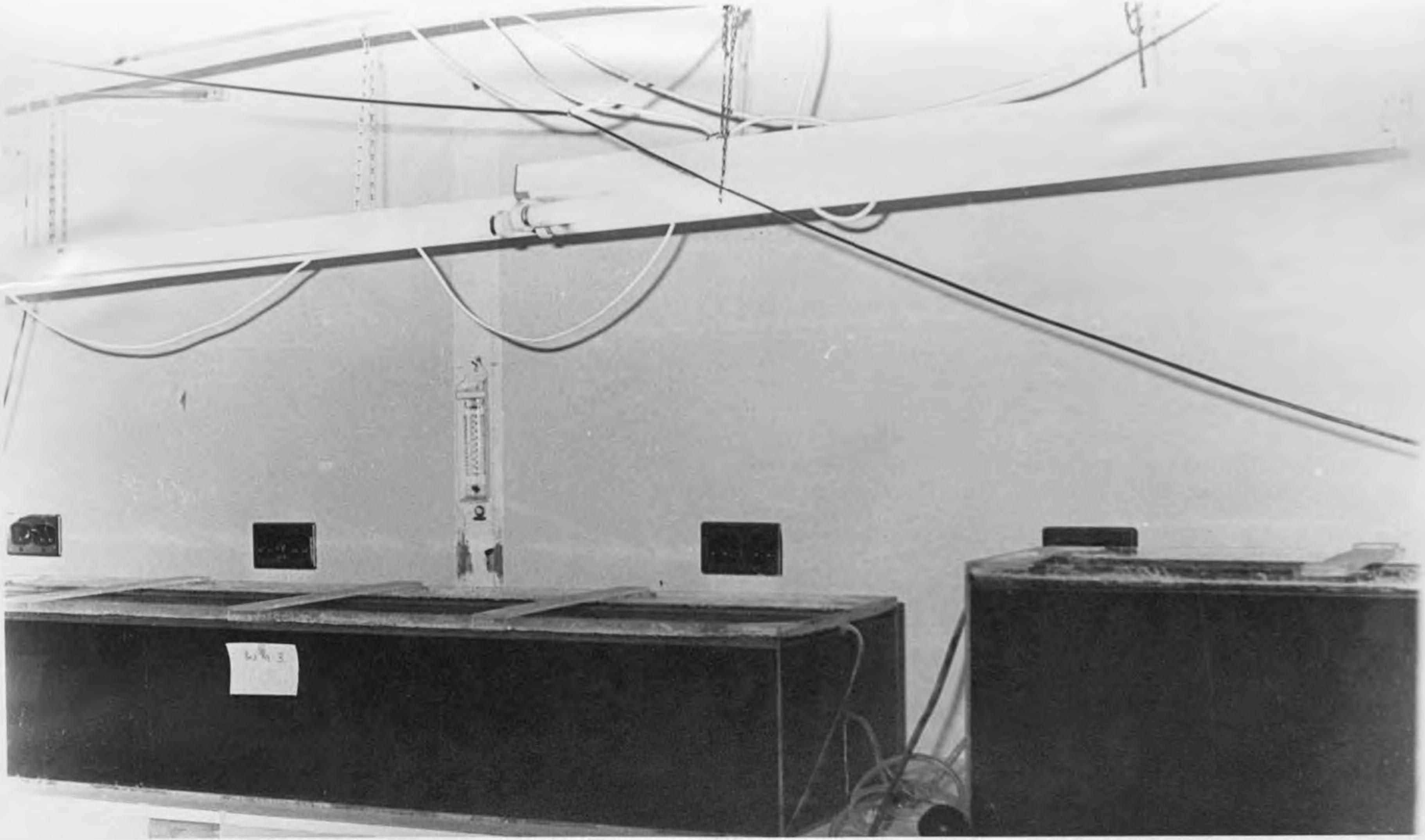


Figure 4.5. ARRANGEMENT OF MODEL PONDS AND LIGHTING SYSTEM

Standard Methods (1971). The titrimetric procedure for determining dissolved oxygen using azide modification (Winkler Method) was adopted for BOD_5. Influent and effluent BOD and COD were measured every other three days whereas pH and room temperature were recorded at a daily basis. Flows were checked daily, readjusted when necessary, the supply tank was filled every other two days.

The influent sewage could not be kept in refrigerator due to quantities involved. As a result there was substantial BOD reduction with increasing storage period in the tank (See Chapter Five). The pH also decreased with increasing time of storage in the supply tank.

The effluent was sampled as described in Section 4.3.1. The influent sample was a grab sample from the supply tank. Eleven days after the start of the experiment, the algal population looked to be decreasing. Each pond was therefore seeded again but this time with approximately 185, 395, 550 and 625 mls of the seeding into ponds 1, 2, 3 and 4 respectively. The above volumes constituted about 0.29% of each model ponds' liquid volume.

Test 2

The procedure as described for Test 1 above was followed except that the composition of the synthetic sewage was altered as shown in Table 4.2. This was done to lower the organic loading.

Four days from the start of the test, an algal specimen from a fresh water pond was added into each model pond in proportion of 0.29% by volume as in Test 1. The algae was identified by the Botany Department of the University of Nairobi as Spirogyra. Twelve days later, the seeding sample as in Test 1 was added to the model ponds because the green colouration had decreased. Twice as much volumes of the seed reported for Test 1 were used.

CHAPTER FIVE

OBSERVATIONS AND EXPERIMENTAL RESULTS

5.1. TRACER STUDIES

The temperature of the water in the model ponds during the tracer studies varied from 21°C to 23°C with 22°C most occuring.

The tables showing the variation of concentration of the tracer (Nacl) with time after start of the experiments for all tests performed as described in Section 4.3.1. are contained in the appendix. The initial concentration was in most cases 0.3 mls of the titrant used. This is an indication of the level of chloride content in tap water.

The tracer results for all tests performed in Test 1 are shown in Figure 5.1. P_1, P_2, P_3 and P_4 refer to curves for Ponds 1, 2, 3 and 4 respectively.

Figure 5.2. shows the results for Test 2. Pond 4 results have been excluded because as noted from the table of results in the appendix, the experiment was terminated before completion. The theoretical detention times for the ponds in Test 2 is the same (Table 5.1.) and the duration of the experiments is more or less the same as indicated in Figure 5.2.

The plot of tracer study results for Test 3 are shown in Figure 5.3. A plot of results obtained from further tests performed on Pond 1 are shown in Figure 5.4.

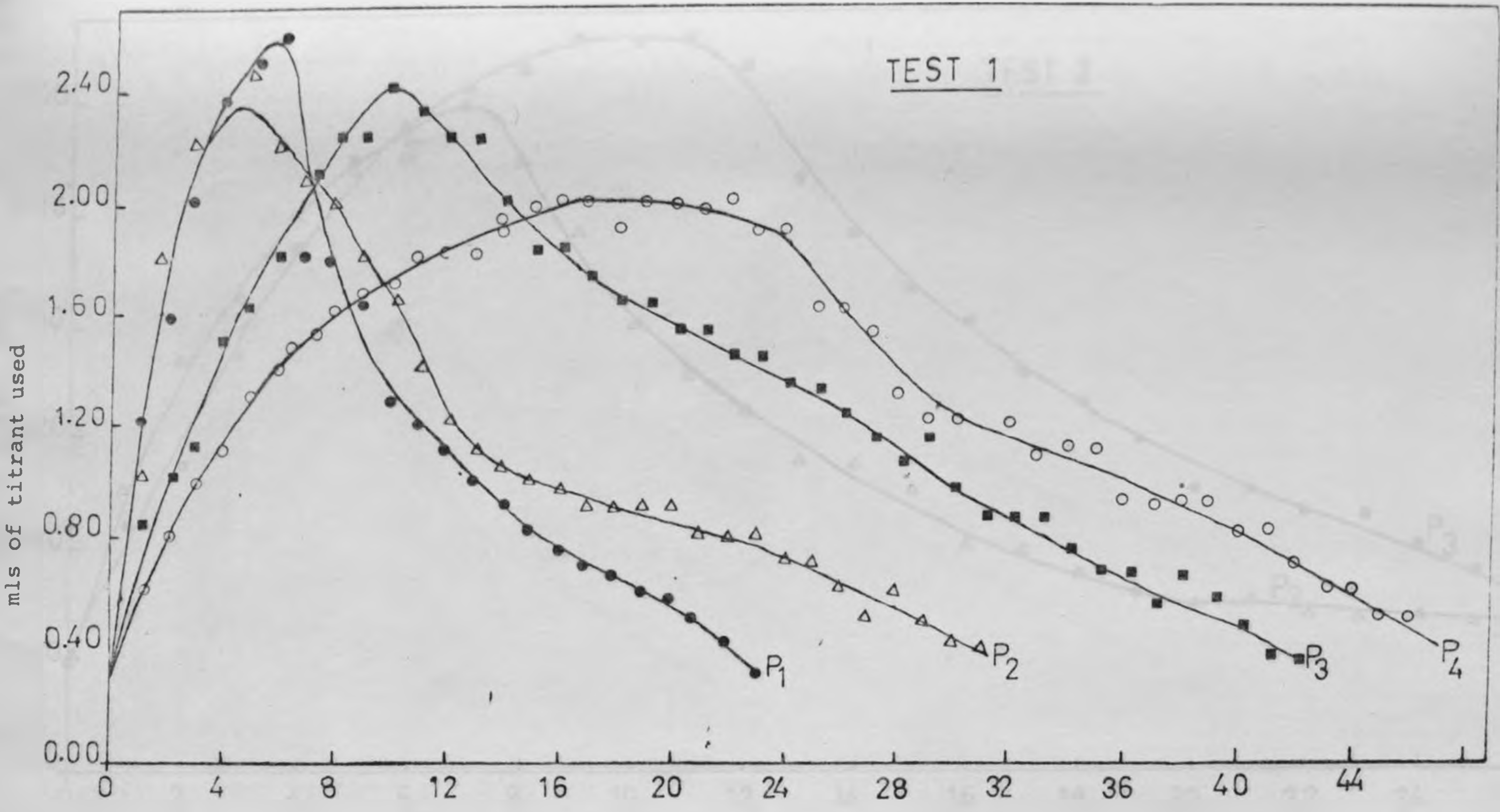


Figure 5.1 TRACER FOR TEST 1.

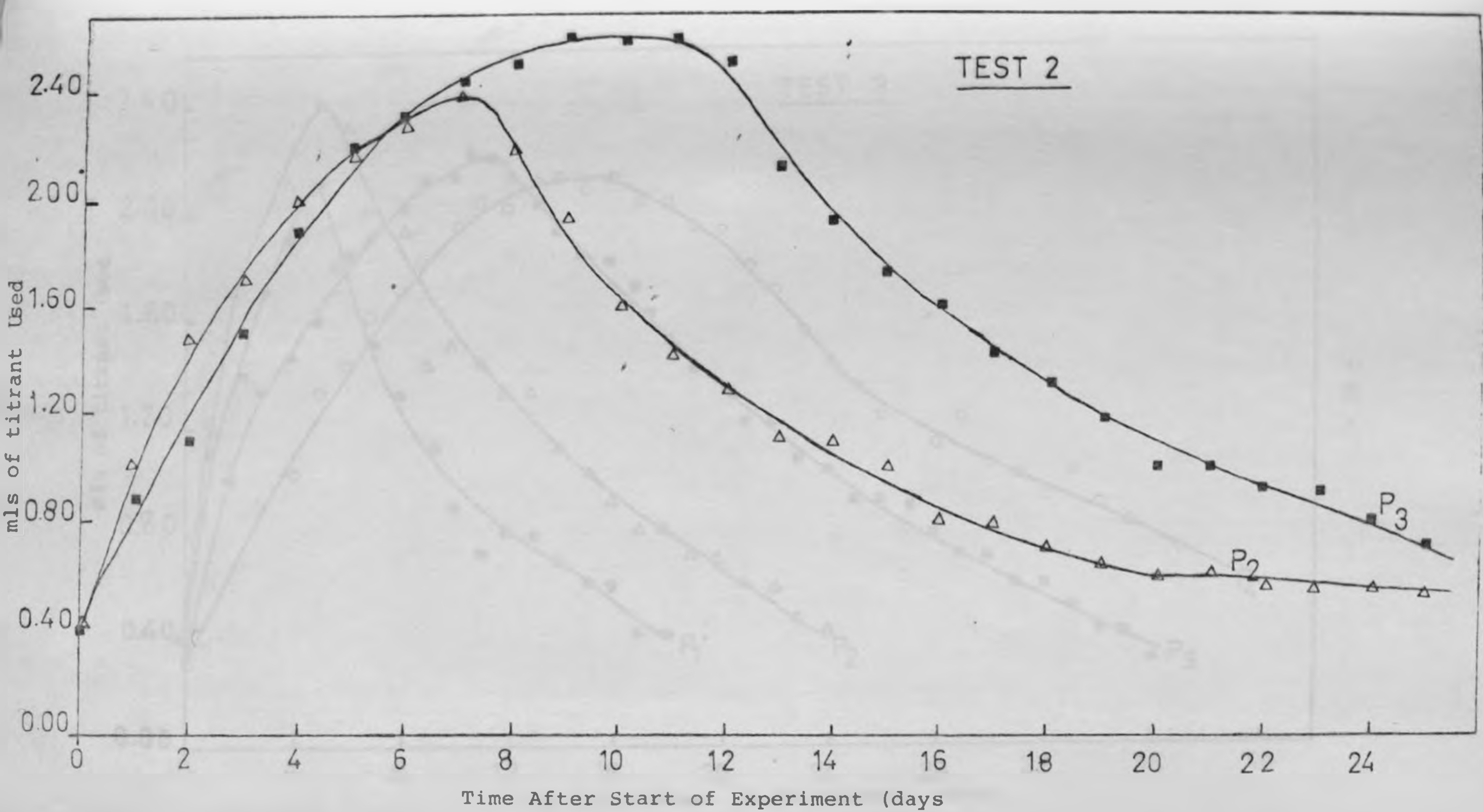


Figure 5.2 TRACEP CURVES FOR TEST 2

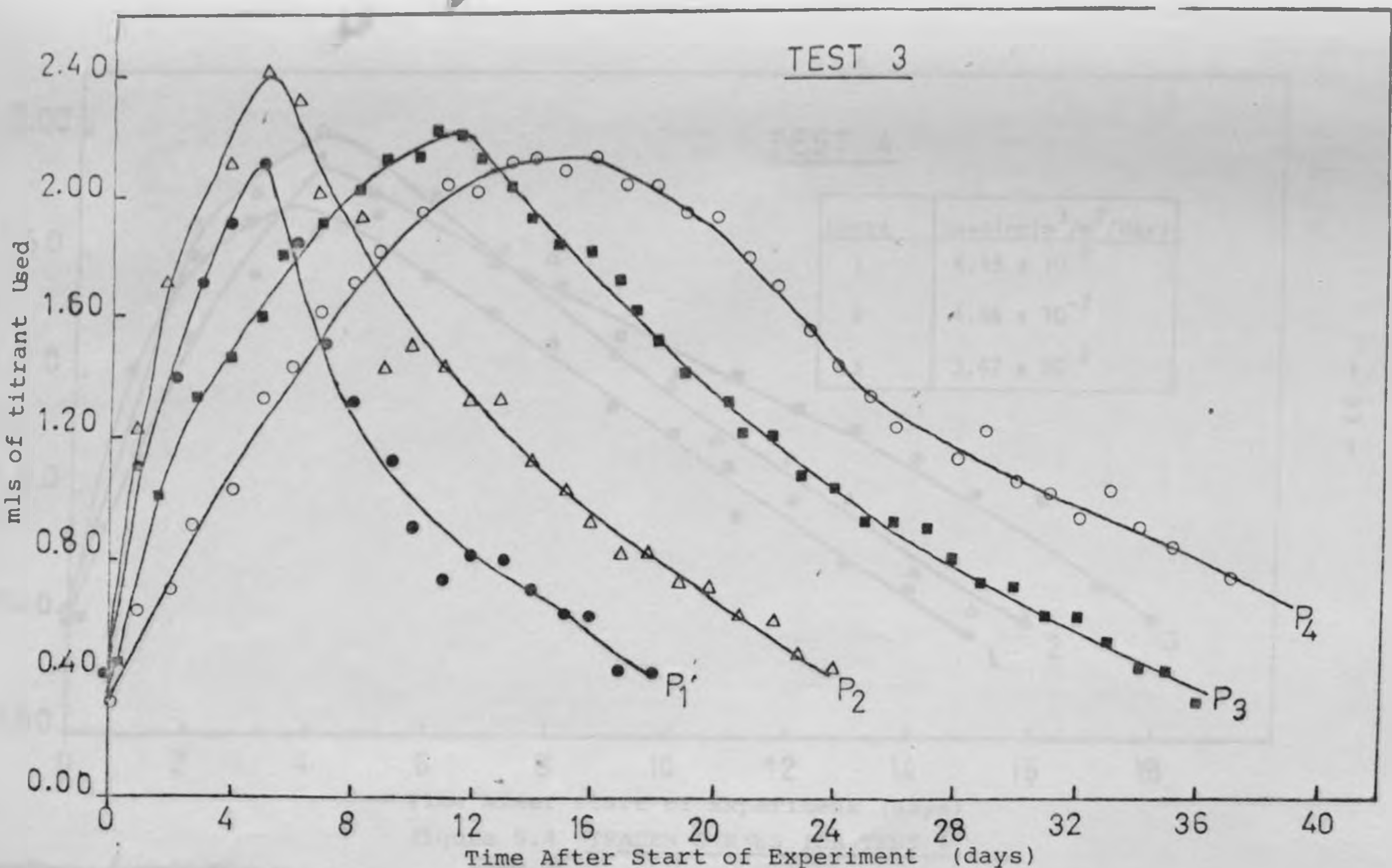


Figure 5.3 TRACER CURVES FOR TEST 3

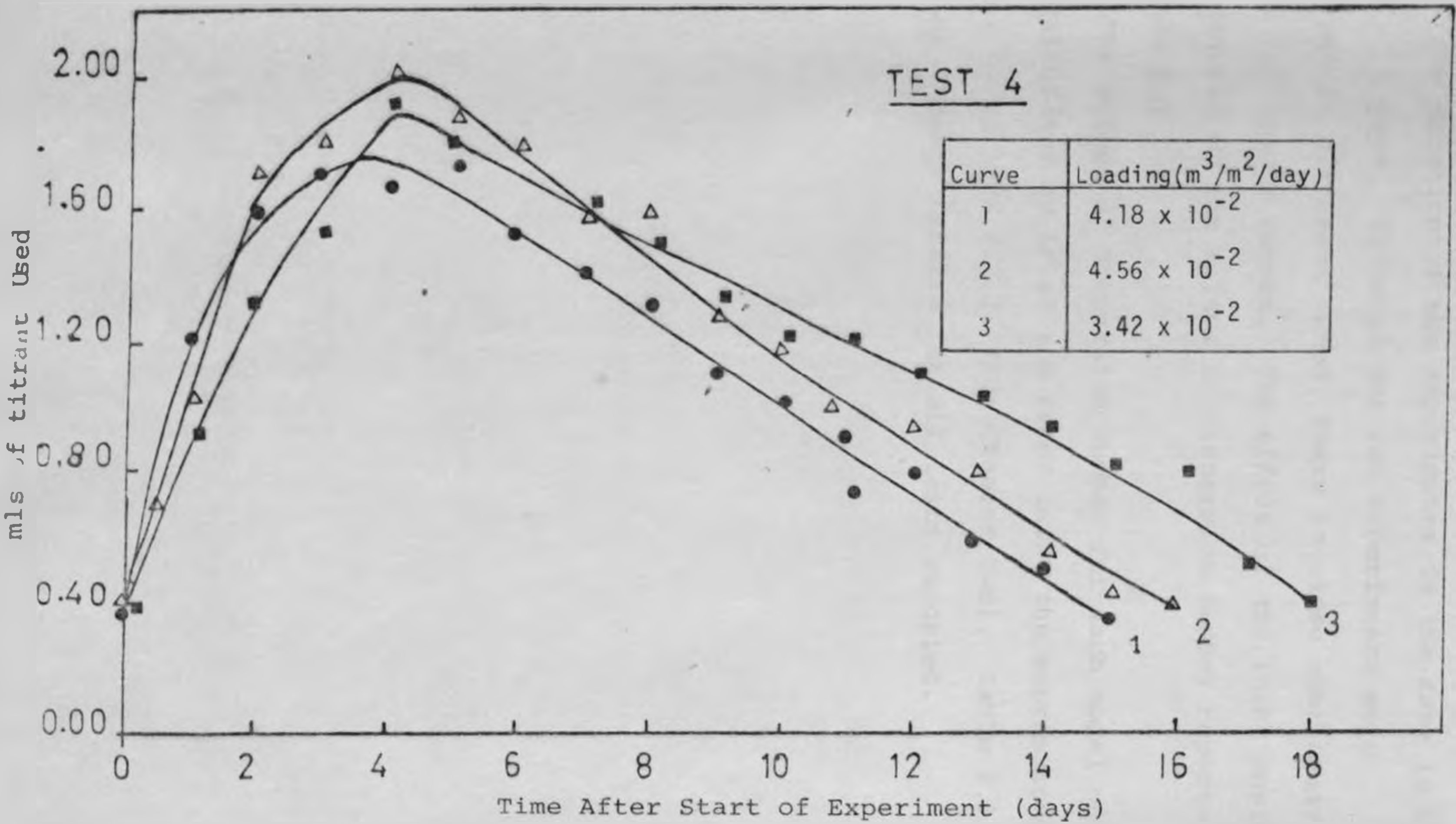


Figure 5.4 TRACER CURVES FOR TEST 4

Curve 3 in Figure 5.4. closely approximates to curve P_1 in Figure 5.3. The maximum concentration of the tracer occurs almost at a same time after the start of the experiment. The duration of the experiments is the same in both cases - 18 days. Although the two experiments were performed at different times, there is close similarity of the two tracer curves. The effect of the inlet position is indicated by the values of dispersion number reported in Table 5.1.

The values of dispersion number for each model pond were calculated by trial and error using the expressions 2.3.2., 2.3.3. and 2.4.2. (See Chapter Two). Table 5.1. contains summary results for all tests reported.

Pond	Liquid depth (mm)	d	$\bar{t}$ (days)	Theoritical detention time (days)	Remarks
1	130	0.299	8.81	4.89	Test 1
2	180	0.350	11.82	6.77	
3	255	0.227	16.94	9.59	
4	330	0.163	20.68	12.40	
2	130	0.294	10.11	4.89	Test 2
3	130	0.198	11.81	4.89	
4	130	0.152	11.11	4.89	
1	130	0.270	7.32	3.80	Test 3
2	180	0.214	9.97	5.26	
3	255	0.226	14.64	7.46	
4	330	0.175	18.08	9.65	
1	130	0.250	6.66	3.11	TEST 4 *
1	130	0.119	7.73	2.85	**
1	130	0.215	8.01	3.80	***

Table 5.1. SUMMARY RESULTS FOR TRACER STUDIES

Note: * Hydraulic loading = 4.18×10^{-2} m^3/m^2/day

** Hydraulic loading = 4.56×10^{-2} m^3/m^2/day

*** Hydraulic loading = 3.42×10^{-2} m^3/m^2/day and inlet in corner.

The variation of dispersion number with varying hydraulic loading for Pond 1 is shown in Figure 5.5. A straight line

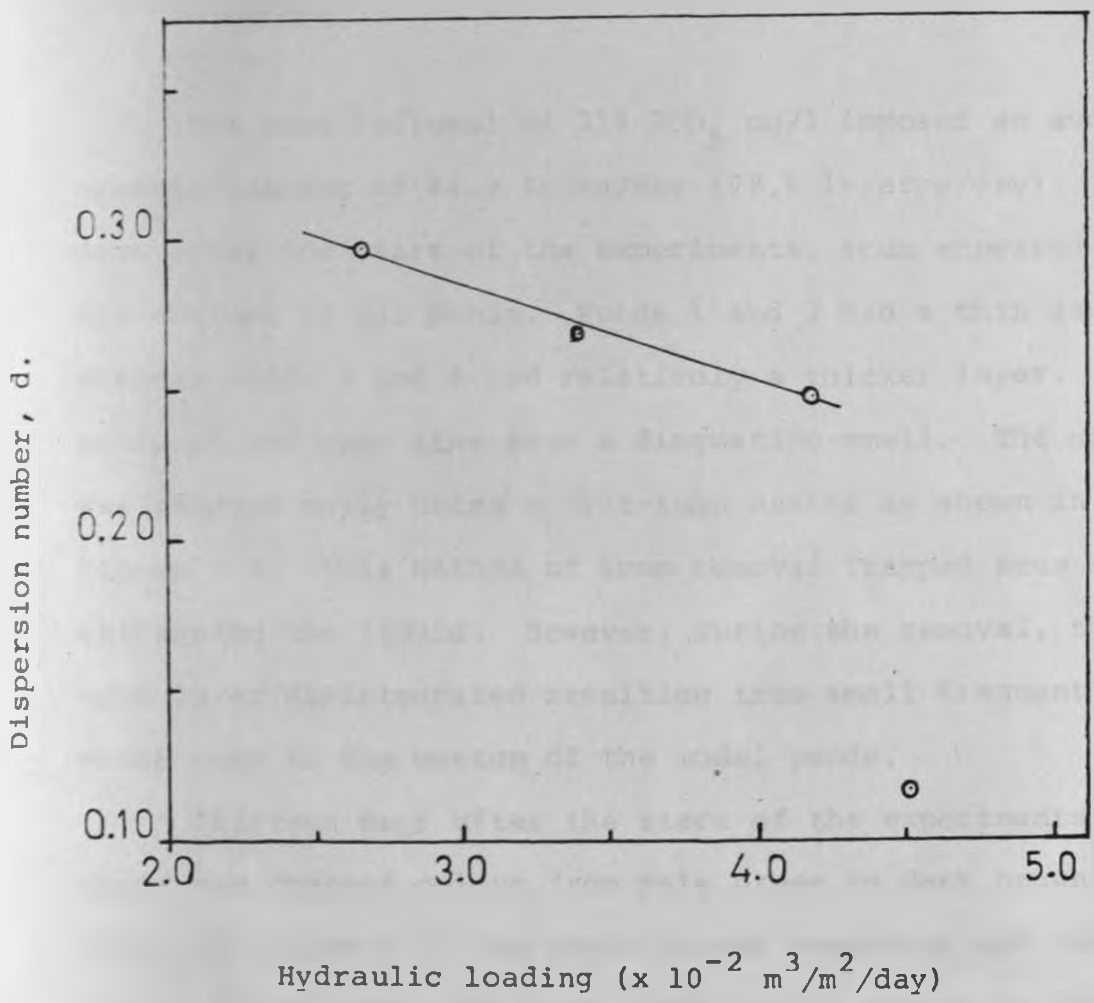


Figure 5.5. HYDRAULIC LOADING VERSUS DISPERSION NUMBER

(Pond 1)

has been assumed through the three points eliminating the fourth point. The divergence of this point could be due to experimental errors.

5.2. BOD REDUCTION EXPERIMENTS

5.2.1. Test 1

The mean influent of 319 BOD_5 mg/l imposed an average organic loading of 84.9 Kg/ha/day (78.8 lb/acre/day). Three days after the start of the experiments, scum appeared on the surface of all ponds. Ponds 1 and 2 had a thin layer whereas Ponds 3 and 4 had relatively a thicker layer. The ponds at the same time gave a disgusting smell. The scum was removed daily using a test-tube washer as shown in Figure 5.6. This method of scum removal trapped scum without entrapping the liquid. However, during the removal, the scum layer disintegrated resulting into small fragments which sunk to the bottom of the model ponds.

Thirteen days after the start of the experiments, the ponds had changed colour from pale green to dark brown. As shown in Figure 5.7, the ponds turned anaerobic and there was no visual trace of green algae colour except in Pond 4. The test was therefore terminated after 19 days of run.

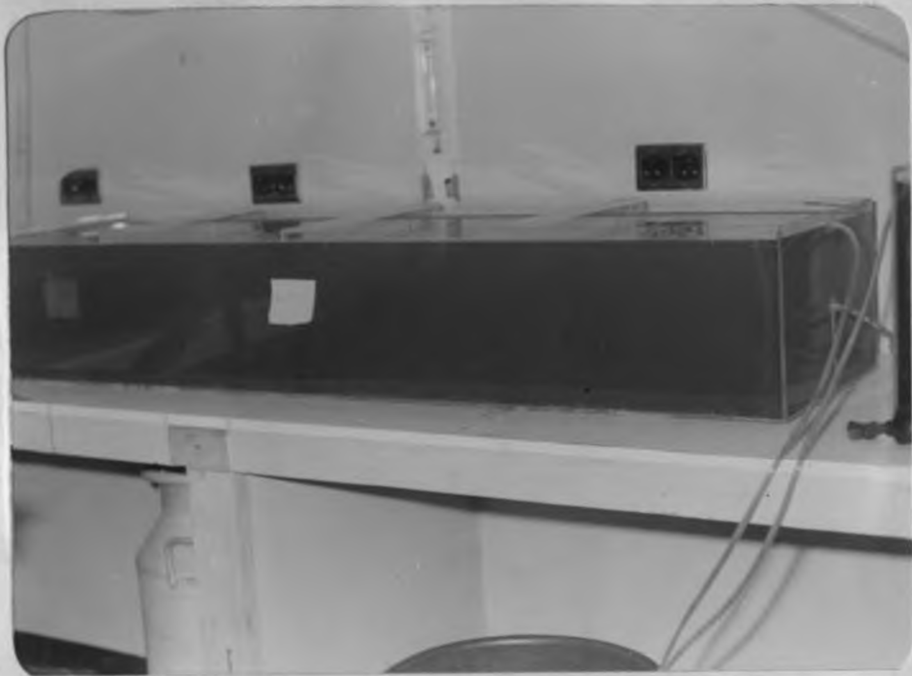
Table 5.2. shows the BOD results for all model ponds. Figure 5.8. is a plot of time after start of experiment versus percentage reduction for the results contained in Table 5.2. The influent BOD and pH are also shown in Figure 5.8. It appears from both Table 5.2 and Figure 5.8, that the test

Figure 5.6. REMOVAL OF SCUM

(i) Pond 1

(ii) Pond 2

Figure 5.7. APPEARANCE OF PONDS AT THE END OF TEST 1



(iii) Pond 3

(iv) Pond 4

Figure 5.7. Contd.

Date	Days	Influent	Pond 1		Pond 2		Pond 3		Pond 4	
1980		Mean BOD_5 =319 mg/l Mean PH =6.6	Mean PH = 7.3		Mean PH = 7.3		Mean PH = 7.2		Mean PH = 7.1	
		BOD_5 (mg/l)	Effluent BOD_5 (mg/l)	BOD Reduction (%)	Effluent BOD_5 (mg/l)	BOD Reduction (%)	Effluent BOD_5 (mg/l)	BOD Reduction (%)	Effluent BOD_5 (mg/l)	BOD Reduction (%)
8/3	0	320	210	-	213	-	218	-	230	-
12/3	4	325	210	35.3	173	46.8	163	49.8	145	55.4
15/3	7	310	113	63.5	100	67.7	80	74.2	85	72.6
18/3	10	230	95	58.7	85	63.0	128	44.3	125	45.7
21/3	13	340	143	57.9	115	66.2	115	66.2	153	55.0
24/3	16	370	130	64.9	133	64.1	163	55.9	143	61.4
27/3	19	335	65	80.5	75	77.6	105	68.7	120	64.2

Table 5.2 <u>BOD RESULTS FOR TEST 1</u>

(Mean temperature = 25°C)

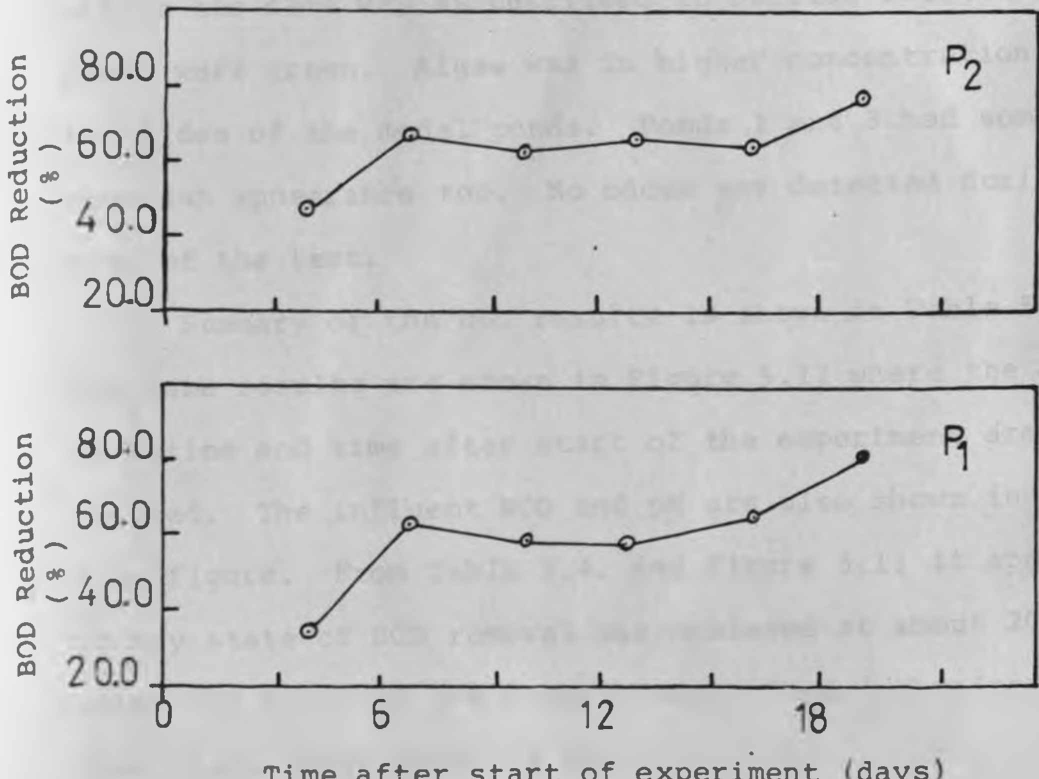
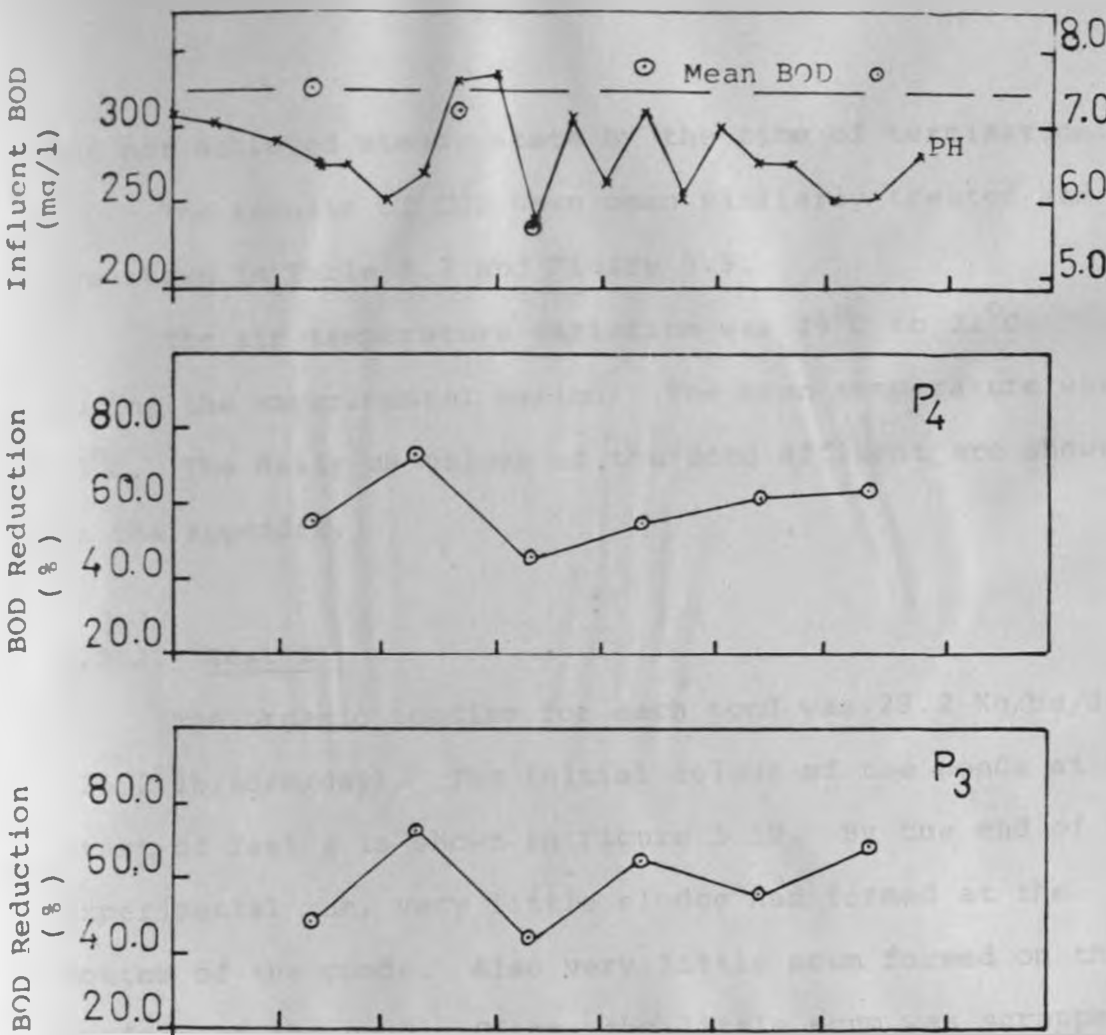


Figure 5.8. BOD RESULTS (TEST 1)


Influent BOD
(mg/1)
300
250
200
Mean BOD
PH
8.0
7.0
6.0
5.0
PH
BOD Reduction
(%)
80.0
60.0
40.0
20.0
P4
BOD Reduction
(%)
80.0
60.0
40.0
20.0
P3

had not achieved steady state by the time of termination.

The results of COD have been similarly treated and are shown in Table 5.3 and Figure 5.9.

The air temperature variation was 29°C to 21°C during the experimental period. The mean temperature was 25°C. The daily pH values of the pond effluent are shown in the appendix.

5.2.2. Test 2

The organic loading for each pond was 28.2 Kg/ha/day (25.2 lb/acre/day). The initial colour of the ponds at the start of Test 2 is shown in Figure 5.10. By the end of the experimental run, very little sludge had formed at the bottom of the ponds. Also very little scum formed on the surface of the pond. Often, the little scum was scrapped off in the same way as described in Section 5.2.1. The ponds were green. Algae was in higher concentration at the sides of the model ponds. Ponds 1 and 3 had some brownish appearance too. No odour was detected during the time of the test.

Summary of the BOD results is shown in Table 5.4. The same results are shown in Figure 5.11 where the BOD reduction and time after start of the experiment are plotted. The influent BOD and pH are also shown in the same figure. From Table 5.4. and Figure 5.11 it appears steady state of BOD removal was achieved at about 20 days after the start of the experiments. Pond 4 obtained the same state after about 14 days.

Date	Days	Influent	Pond 1		Pond 2		Pond 3		Pond 4	
1980		Mean COD =541 mg/l								
		COD (mg/l)	COD (mg/l)	COD Reduction (%)	COD (mg/l)	COD Reduction (%)	COD (mg/l)	COD Reduction (%)	COD (mg/l)	COD Reduction (%)
8/3	0	584	292	-	292	-	300	-	404	-
12/3	4	584	360	38.4	248	57.6	360	38.4	224	61.6
15/3	7	624	256	59.0	216	65.4	384	38.5	224	64.1
18/3	10	656	232	64.6	176	73.2	352	46.3	232	64.6
21/3	13	432	172	60.2	152	64.8	312	27.8	176	59.3
24/3	16	384	220	42.7	96	75.0	224	41.7	104	72.9
27/3	19	520	180	65.4	160	69.2	260	50.0	172	66.9

Table 5.3. COD RESULTS FOR TEST 1

(Mean temperature = 25°C)

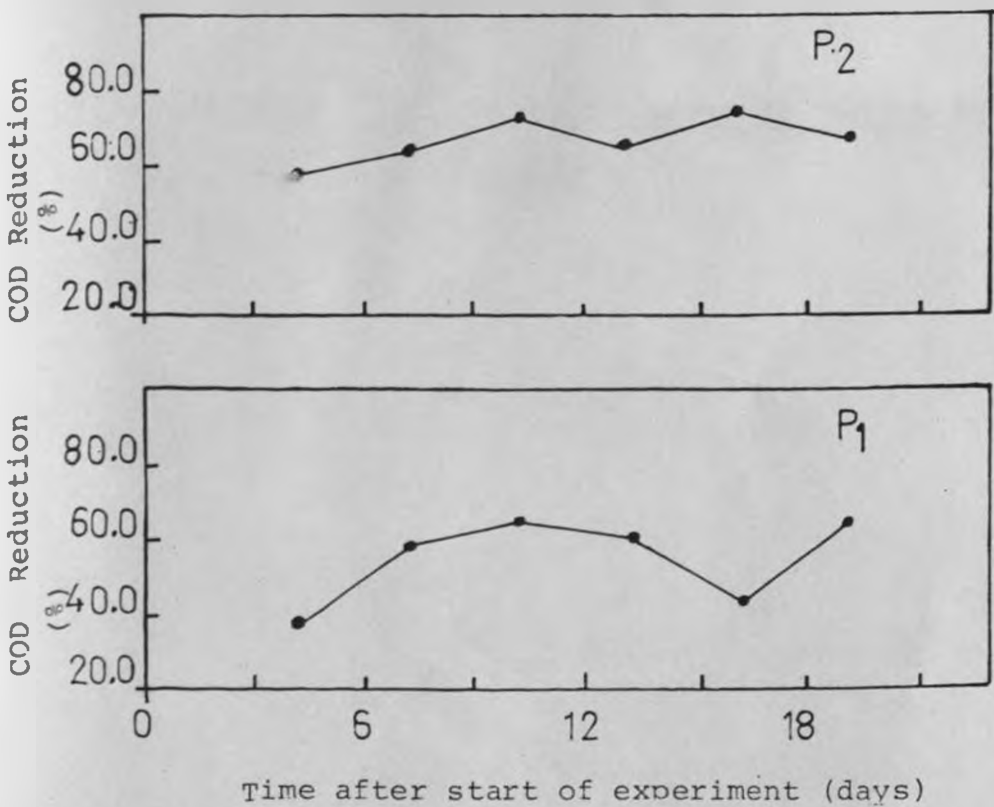
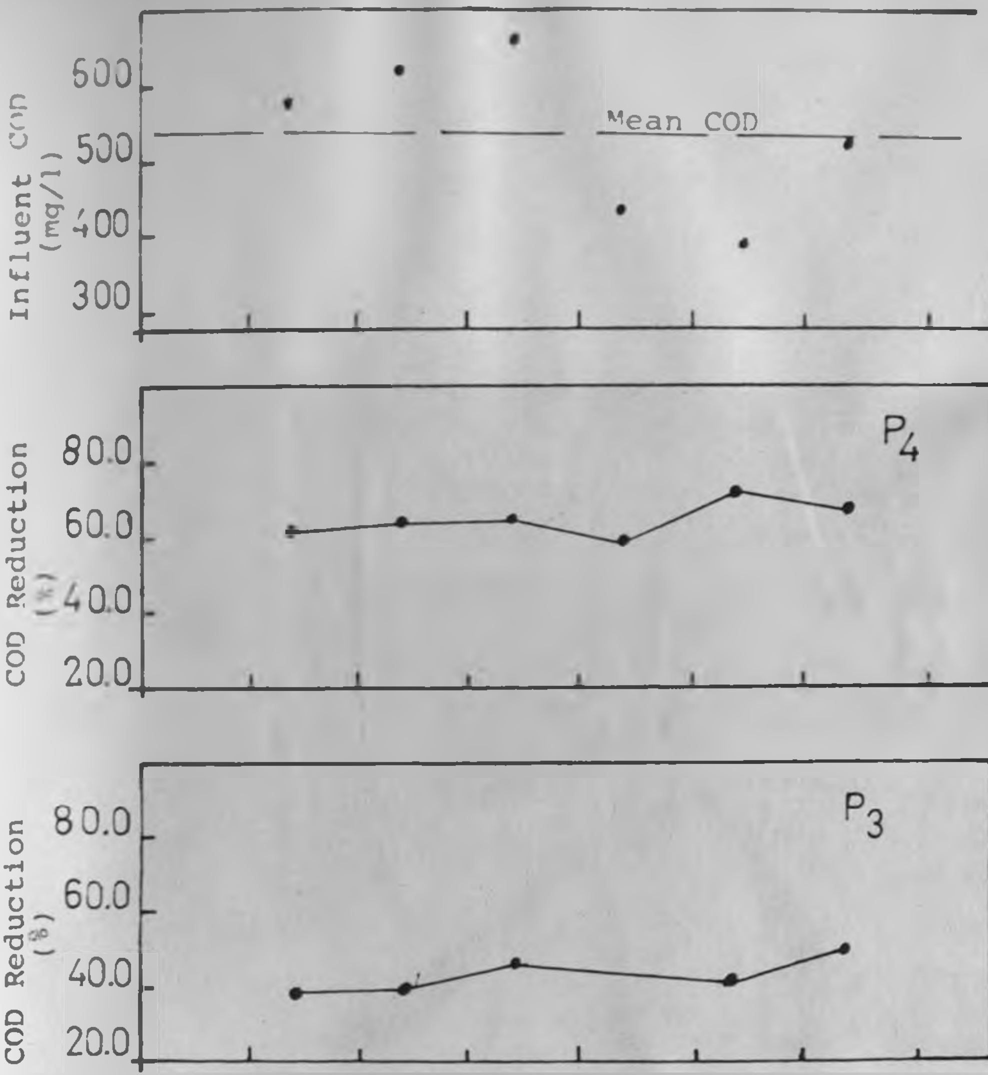


Figure 5.9. COD RESULTS (TEST 1)


Influent COD
(mg/l)
600
500
400
300
Mean COD
COD Reduction
(%)
80.0
60.0
40.0
20.0
P4
COD Reduction
(%)
80.0
60.0
40.0
20.0
P3

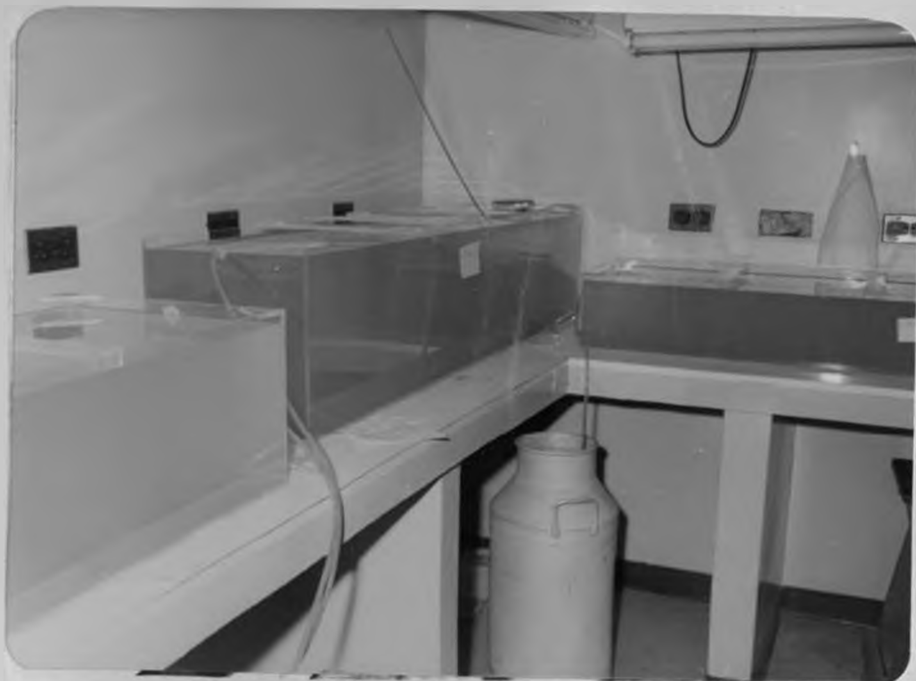


Figure 5.10. COLOUR OF PONDS AT THE START OF TEST 2

Date	Days	Influent	Pond 1		Pond 2		Pond 3		Pond 4	
1980		Mean BOD_5 = 106mg/l Mean pH=6.7	Mean PH = 7.2		Mean PH = 7.3		Mean PH = 7.2		Mean PH = 7.2	
		BOD_5 (mg/l)	Effluent BOD_5 (mg/l)	BOD Reduction (%)	Effluent BOD_5 (mg/l)	BOD Reduction (%)	Effluent BOD_5 (mg/l)	BOD Reduction (%)	Effluent BOD_5 (mg/l)	BOD Reduction (%)
29/3	0	173	118	-	125	-	105	-	90	-
1/4	3	75	40	46.7	55	26.7	65	13.3	53	29.3
4/4	6	110	28	74.5	30	72.7	43	60.9	63	42.7
7/4	9	-	-	-	-	-	-	-	-	-
10/4	12	105	25	76.2	30	71.4	35	66.7	53	49.5
13/4	15	125	43	65.6	30	76.0	33	73.6	40	68.0
16/4	18	135	40	70.0	43	68.1	50	63.0	40	70.4
19/4	21	100	25	75.0	20	80.0	20	80.0	30	70.0
22/4	24	70	13	81.4	15	78.6	15	78.6	20	71.4
25/4	27	70	15	78.6	20	71.4	15	78.6	20	71.4
28/4	30	100	20	80.0	30	80.0	20	80.0	30	70.0
1/5	33	-	-	-	-	-	-	-	-	-
4/5	36	90	15	83.3	28	69.8	20	77.8	20	72.2
8/5	40	110	15	86.0	30	72.7	20	81.8	30	72.7
13/5	45	110	35	68.2	35	68.2	25	77.3	25	77.3

Table 5.4 <u>BOD RESULTS FOR TEST 2</u> (Mean Temperature = 25.2°C)

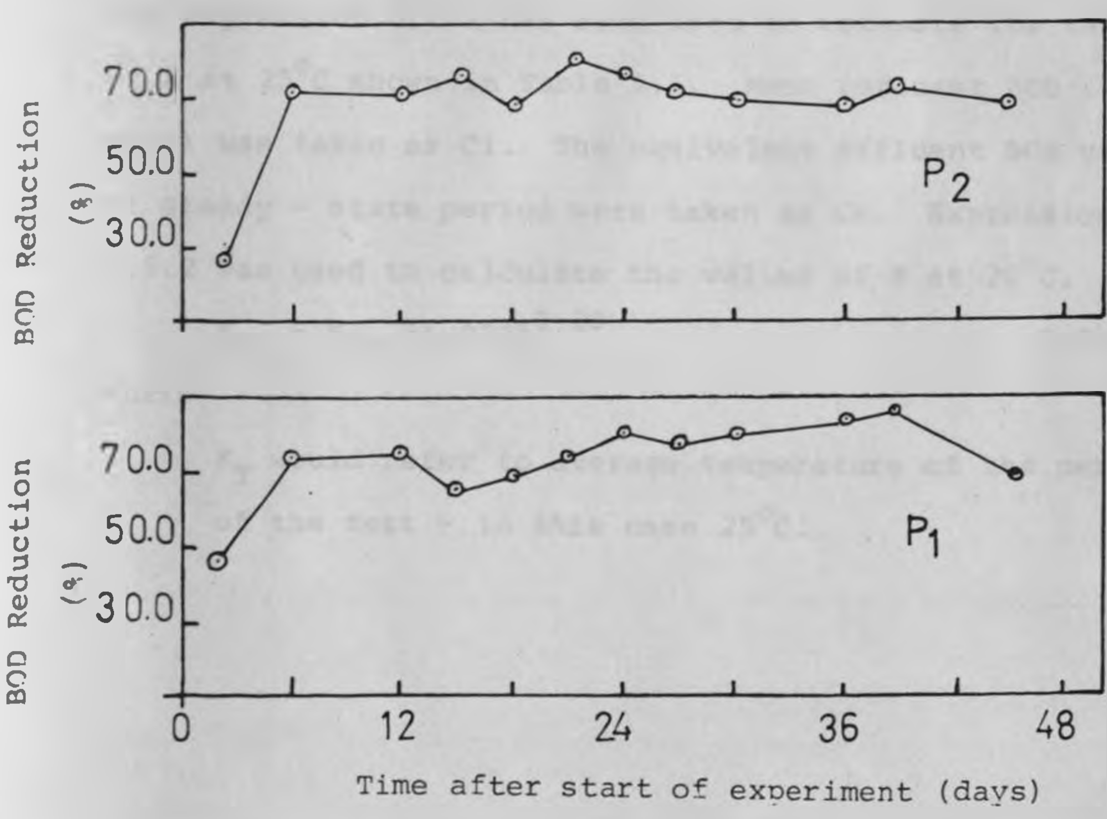
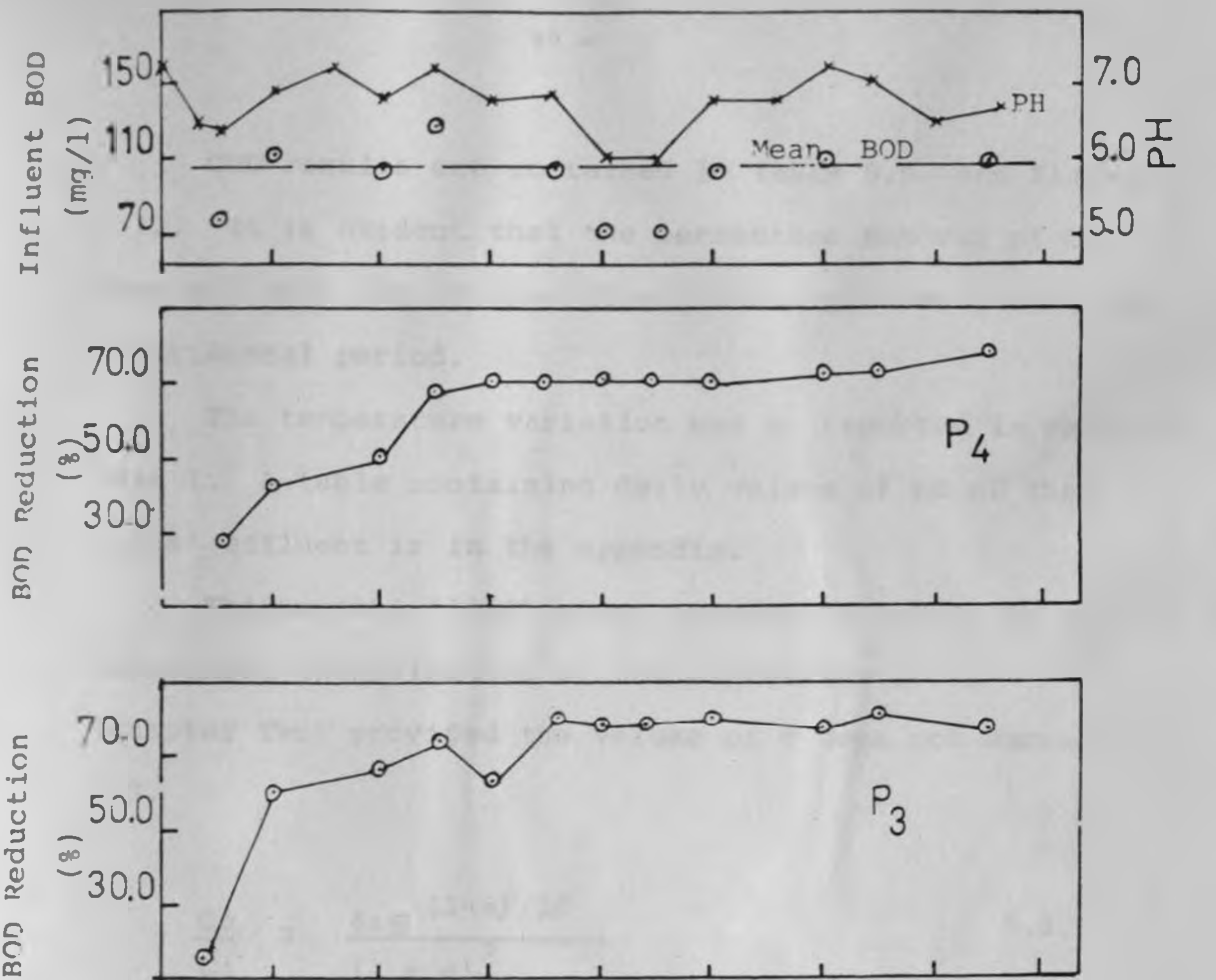


Figure 5.11. BOD RESULTS (TEST 2)


Influent BOD (mg/l)
150
110
70
7.0
6.0
5.0
PH
Mean BOD
BOD Reduction (%)
70.0
50.0
30.0
P_4
BOD Reduction (%)
70.0
50.0
30.0
P_3

COD results are contained in Table 5.5. and Figure 5.12. It is evident that the percentage removal of COD does not show any obvious steady state pattern during the experimental period.

The temperature variation was as reported in case of Test 1. A table containing daily values of pH of the ponds' effluent is in the appendix.

Thirumurthi (1969) gives expression 5.2.1. as a reasonable approximation of the expression 2.5.1 (Chapter Two) provided the values of d does not exceed 2.0.

$$\frac{Ce}{Ci} = \frac{4a\,e^{(1-a)/2d}}{(1+a)^2} \qquad 5.2.1.$$

The expression 5.2.1 has been used to estimate the values of K at 25°C shown in Table 5.6. Mean influent BOD (106 mg/l) was taken as Ci. The equivalent effluent BOD values at steady - state period were taken as Ce. Expression 5.2.2 was used to calculate the values of K at 20°C.

$$K_T = K_{20}\,(1.072)^{T-20} \qquad 5.2.2.$$

Where

K_T would refer to average temperature of the period of the test - in this case 25°C.

Date	Days	Influent	Pond 1		Pond 2		Pond 3		Pond 4	
1980		Mean COD =216 mg/l								
		COD (mg/l)	Effluent COD (mg/l)	COD Reduction (%)	Effluent COD (mg/l)	COD Reduction (%)	Effluent COD (mg/l)	COD Reduction (%)	Effluent COD (mg/l)	COD Reduction (%)
29/3	0	320	132	-	112	-	120	-	124	-
1/4	3	224	100	55.4	96	57.1	100	55.4	104	53.6
4/4	6	224	-	-	-	-	-	-	-	-
7/4	9	-	-	-	-	-	-	-	-	-
10/4	12	176	80	54.5	56	68.2	92	47.7	108	38.6
13/4	15	212	148	30.2	68	67.9	148	30.2	108	49.1
16/4	18	216	88	59.3	84	61.1	84	61.1	116	46.3
19/4	21	208	(148)	(28.8)	116	44.2	76	63.5	96	53.8
22/4	24	200	116	42.0	84	58.0	84	58.0	64	68.0
25/4	27	-	96	-	84	-	116	-	96	-
28/4	30	168	68	59.5	72	57.1	88	47.6	64	61.9
1/5	33	232	80	65.5	64	72.4	80	65.5	52	77.6
4/5	36	184	92	50.0	(160)	(13.0)	72	60.9	(112)	(39.1)
8/5	40	200	76	62.0	116	47	84	58.0	84	58.0
13/5	45	240	152	36.7	136	43.3	116	51.7	112	53.3

Table 5.5. COD RESULTS FOR TEST 2. (Mean Temperature = 25.2^{o}C)

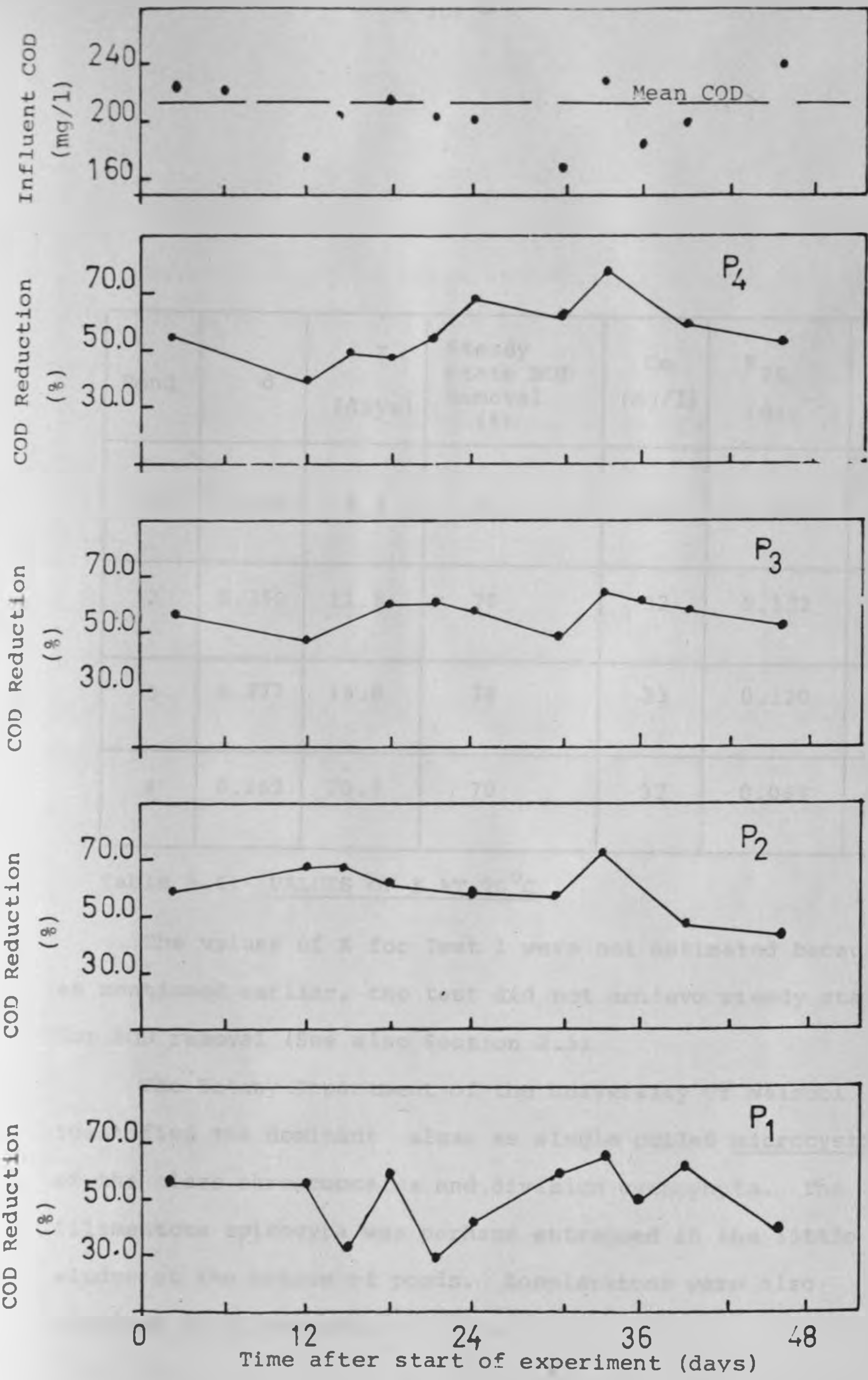


Figure 5.12. COD RESULTS (TEST 2)

Pond	d	$\bar{t}$ (days)	Steady State BOD Removal (%)	Ce (mg/l)	K_{25} (day^{-1})	K_{20} (day^{-1})
1	0.299	8.8	81	20	0.260	0.184
2	0.350	11.8	70	32	0.132	0.093
3	0.277	16.9	78	23	0.120	0.085
4	0.163	20.7	70	32	0.067	0.047

Table 5.6. VALUES OF K AT 20^{o}C

The values of K for Test 1 were not estimated because as mentioned earlier, the test did not achieve steady state for BOD removal (See also Section 2.5)

The Botany Department of the University of Nairobi identified the dominant algae as single celled microcystis of the class chroococcales and division cyanophyta. The filamentous spirogyra was perhaps entrapped in the little sludge at the bottom of ponds. Zooplanktons were also reported to be present.

CHAPTER SIX

DISCUSSION

Mara (1975) notes that the hydraulic studies and field experience have shown that the best shape for lagoon is rectangular. Arceivala et al (1970) however, writes that it is not necessary that the pond shape should be of any particular type, and that the pond can be oval, square, rectangular or polygonal depending on the site contours. Following the studies made in this report, it appears shape and geometry of a pond are important parameters in the design of ponds because they affect the hydraulic properties. Mara (1975) and Arceivala et al (1970) recommend a length to breadth ratio of 2:1 to 3:1 and 3:1 respectively.

Increase in depth of a given pond geometry increases the detention time. Choice of depth is also important in pond design because shallow ponds encourage emergent growth of vegetation and consequent mosquito breeding. Very deep ponds turn anaerobic. For the above reasons, choice of depth in pond design is of paramount importance.

The choice of inlet and outlet is an important facility in design of ponds. The inlet of a facultative and anaerobic ponds should always discharge below the water surface level. Accumulation of quantity of floating matter and scum will increase with inlets which discharge at the surface. This increases the chances of odour nuisances. Gloyna (1971) writes that the inlet of a rectangular pond should be located at a point one third of the distance along a line drawn from the upstream edge of the pond to the outlet. Most ponds have weirs as effluend ends.

The hydraulic loading rate in the pond is important parameter too. While the area of the pond should be sufficient for adequate production of oxygen by algae and solar energy utilisation, the hydraulic loading (detention time) should be sufficient for this oxygen to be utilised by biochemical reactions.

6.1. TRACER STUDIES

Although waste water flow rate may be stated as a certain value, there is usually considerable variation depending on the population or industries served. Tebbutt (1971) writes that the normal minimum flow at all except very small sewage works is usually taken as about 40% dry weather flow. In view of the above, the variation of flow of maximum 29% during the experimental period can be justified.

The tracer curves drawn for all the tests are of normal shape for intermediate type of flow. A look at the actual mean detention time and the theoritical detention time indicate that short circuiting was absent. Dead spaces could have, however, been present. The ratio of actual mean detention time to the theoritical detention time range between 1.7 and 2.7. Mixing was not very pronounced. In order to obtain reliable results, the exact manner of adding a tracer should be carefully standardised. Because of the volumes of the model ponds, appreciable quantities of tracer were necessary so that the concentration of the tracer, after dilution in the pond, could be detected by the method employed. The quantities used (See Section 4.3.1) could not be introduced by injection because with the available equipment this would have resulted into an extended period of tracer introduction. It is usually desired that the tracing time be instanteneous, although this was not very practical with the quantities of tracer used.

The larger the number of points on the tracer curve included, the smaller the relative error there will be in the calculated dispersion number. To achieve this, the tracer tests were conducted for a long duration of time as shown in the appendix. In existing ponds, where environmental conditions may need to be maintained during the tracer tests, the method may have some practical hazards.

Table 5.1. indicate that at the same hydraulic loading there is a decrease of dispersion number with increasing length to breadth ratio. The discrepancy occurred in Pond 2, Test 1 and Pond 3, Test 3 perhaps due to unknown experimental errors. For the same hydraulic loading of a given pond, there appear to be a decrease of dispersion number with decrease in depth. There is also a decrease of dispersion number with increasing hydraulic loading for a given pond geometry. Figure 5.5. shows that 3 out of 4 points approximate to a linear relationship. Thirumurthi (1967) results also indicate a decrease of d with increase of hydraulic loading, although the actual mean detention times were smaller than the theoritical detention time.

Under the same hydraulic loading of Pond 1, a change of inlet from the position as shown in Figure 4.1. to a corner in the influent end reduced the dispersion number by 20.4%. With consideration that the dispersion number is quite reproducible (Thirumurthi, 1969) it appears that the change of inlet position has obvious effect on the dispersion number. Change of influent to other positions could not be performed because of time factor. However, further investigation to establish the best position of the influent for best performance of the stabilisation ponds will be of great help to design engineers. This may require simultaneous tracer and BOD reduction experiments on model ponds or/and prototype ponds.

6.2. BOD REDUCTION EXPERIMENTS

Ponds which have light organic loading will tend to perform uniformly over varying climatic and other conditions. Ponds with greater organic loading tend to become more sensitive to changes in sewage quality and temperature. They also tend to give odour problems. The allowable organic loading on a pond depends on the rate at which biological processes can satisfactorily decompose the organic matter without creating nuisance conditions.

From the observations of the two tests, scum formation increases with increasing organic loading. In Test 1, the sinking disintegrated scum could have resulted in entrapping the algae. This would result into the algal deposits together with scum at the bottom of the pond where anaerobic conditions prevail.

Olson et al (1968) reported that the purple colouration of lagoons was caused by purple - sulphur bacterium - chromatium which grow well in overloaded lagoons. The lagoons operated at a loading of 336 Kg/ha/day (330 lb/acre/day). The problem with chromatium population in ponds is the pigmentation and anaerobic nature which prevents the growth of algae.

Raman et al (1970) reported that the pink colouration of facultative lagoons was caused by merismopedia tenuissima, an algae that has a metabolism similar to that of sulphur bacteria. The pink phenomena were favoured by reduced sunlight, high organic loading, increased depth, high temperature and excessive sludge deposits.

It therefore appears that the anaerobic conditions which developed in Test 1 were due to high organic loading (84.9 Kg/ha/day) which resulted to either or both of the above organisms.

Removal of BOD at first increases rapidly and then less rapidly with time, until for a steady influent a steady removal rate is achieved. This fact is evident in Figures 5.9 and 5.11. The steady state BOD removals for each pond in Test 2 are shown in Table 5.6. The removals range between 70% to 80%. High removals in Ponds 1 and 3 compared to 2 and 4 may be explained by the anaerobic conditions that had started to develop. From Table 5.2, it is also evident that anaerobic conditions result into higher BOD removals because by the end of the test, Pond 4 (Figure 5.7) which was least anaerobic, had the least BOD removal. BOD test samples were not filtered. Algae could have therefore exerted some BOD resulting into relatively higher effluent BOD values. Zooplanktons were present and could have been feeding on algae, reducing the quantity of algae hence affecting BOD removal.

The results also indicate that in most cases, COD removals were lower than BOD removals. The COD results are more flactuating for reasons unknown.

PH of the pond effluent of each pond was quite steady during the experimental time (See Appendix).

The organic loading, illumination and type of influent used in Test 2 gave the estimated values of K as contained in Table 5.6. Thirumurthi (1967, 1969) experimenting under different laboratory conditions reported K values at 20°C in the range 0.02 to 0.204 day^{-1}. The values reported in Table 5.6. therefore lie within the range of the earlier reported values. It would therefore appear that before a selection of K and d for design is made, prototype experiments under local field conditions may be necessary. The values of K decreased with decreasing values of dispersion number.

6.3. APPLICATION OF THE STUDY

Proper application of model studies to field conditions will usually require properly determined scaling up factors. Otherwise the predicted behaviour should be tested on prototype plants to confirm that the predicted behaviour is true for field conditions. With proper scaling up factors, most economical design parameters can be cheaply predicted through laboratory model studies then the data of various investigators in different localities of the world can be compiled. The charts and tables of values of K, $\bar{t}$, d as functions of hydraulic loading and other specified controlled conditions such as illumination, temperature, organic loading could also be compiled. These charts and tables would obviously help in design and maintainance of waste stabilisation ponds.

The knowledge of effect of inlet, outlet arrangement and hydraulic loading on the dispersion number of a pond, could be used to improve the performance of existing ponds as explained below. Rearrangement of the above parameters coupled with field tracer studies could lead to alteration of the dispersion number. This could be done to minimise short circuiting and achieve better BOD removal. In this case overloaded ponds can be improved.

CHAPTER SEVEN

CONCLUSIONS

Model tests were performed on pond models of varying sizes as described in Chapter Four. Laboratory simulated conditions have been described in the same Chapter. Synthetic chemical sewage of the compositions specified in Chapter Four and the effluent of an existing pond as a seed were used in BOD reduction tests. Therefore based on the conditions of the study, the following conclusions can be drawn:

(1) Length, breadth, depth and inlet position of a pond will affect its hydraulic properties as indicated by the values of dispersion number. The flow pattern was the arbitrary type as verified by the shape of tracer curves and the values of dispersion numbers.

(2) At the same hydraulic loading, increase in length to breadth ratio decreased the dispersion number.

(3) Increase in detention time by increasing depth of a given pond geometry at a given hydraulic loading resulted into increase of dispersion number.

(4) For a given pond geometry, increase in hydraulic loading decreased the dispersion number.

(5) Organic loading of 84.9 Kg/ha/day (78.8 lb/acre/day)

under the prevailing conditions resulted into ponds' failure to behave facultative. The formation of scum increased with increase in organic loading.

(6) BOD removal efficiencies for all the pond models did not follow any pattern related to dispersion number.

(7) The values of first order BOD removal constant for the experimental results were calculated using the expression given by Thirumurthi (1969). The values obtained satisfactorily fall within the range of those reported by Thirumurthi (1967, 1969).

(8) At a given hydraulic loading, ponds with lower values of dispersion number had lower values of first order BOD removal constant.

(9) Proper field application of the values of d and K reported in this study requires further confirmation using prototype experiments operating under field conditions.

A P P E N D I X

(A) RESULTS OF TRACER STUDIES

Hydraulic loading: 2.66 x 10^{-2} m^3/m^2/day

Date experiment started : 10/10/79

Date experiment ended : 9/11/79

Time after start (days)	Tracer concentration (mls of titrant used	Time after start(days)	Tracer concentration (mls of titrant used)
0	0.30	13	1.00
1	1.20	14	0.90
2	1.60	15	0.80
3	2.00	16	0.70
4	2.30	17	0.70
5	2.50	18	0.65
6	2.60	19	0.60
7	1.80	20	0.60
8	1.80	21	0.50
9	1.70	22	0.40
10	1.30	23	0.30
11	1.20		
12	1.10		

POND 1, Test 1.

Hydraulic loading: 2.66 x 10^{-2} m^3/m^2/day

Date experiment started : 10/10/79

Date experiment ended : 10/11/79

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start(days)	Tracer Concentration (mls of titrant used)
0	0.30	16	0.95
1	1.00	17	0.90
2	1.80	18	0.90
3	2.20	19	0.90
4	2.30	20	0.90
5	2.50	21	0.80
6	2.20	22	0.80
7	2.10	23	0.80
8	2.00	24	0.70
9	1.80	25	0.70
10	1.70	26	0.60
11	1.40	27	0.50
12	1.20	28	0.60
13	1.10	29	0.50
14	1.05	30	0.40
15	1.00	31	0.40

POND 2, <u>Test 1</u>

Hydraulic loading : 2.66×10^{-2} m^3/m^2/day

Date experiment started : 10/10/79

Date experiment ended : 21/11/79

Time after start(days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.30	22	1.40
1	0.80	23	1.40
2	1.00	24	1.30
3	1.10	25	1.30
4	1.50	26	1.20
5	1.60	27	1.10
6	1.80	28	1.00
7	2.10	29	1.10
8	2.20	30	0.90
9	2.20	31	0.80
10	2.40	32	-
11	2.40	33	0.80
12	2.20	34	0.70
13	2.20	35	0.60
14	2.00	36	0.60
15	1.80	37	0.50
16	1.80	38	0.60
17	1.70	39	0.50
18	1.60	40	0.40
19	1.60	41	0.30
20	1.50	42	0.30
21	1.50		

POND 3, Test 1

Hydraulic loading: $2.66 \times 10^{-2}\ m^3/m^2/day$
Date experiment started : 10/10/79
Date experiment ended : 26/11/79

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer Concentration (mls of titrant used)
0	0.30	24	1.90
1	0.60	25	-
2	0.80	26	1.60
3	1.00	27	1.60
4	1.10	28	1.50
5	1.30	29	1.30
6	1.40	30	1.20
7	1.60	31	1.20
8	1.70	32	-
9	1.80	33	0.90
10	1.70	34	0.90
11	1.80	35	1.10
12	1.80	36	1.10
13	1.80	37	0.90
14	1.90	38	0.90
15	2.00	39	0.80
16	2.00	40	0.80
17	2.00	41	0.70
18	1.90	42	0.60
19	2.00	43	0.60
20	2.00	44	0.50
21	2.10	45	0.50
22	2.00	46	0.40
23	1.90	47	0.40

POND 4, <u>Test 1</u>

Hydraulic loading : 2.66 x 10^{-2} m^3/m^2/day Date experiment started : 15/11/79 Date experiment ended : 12/12/79			
Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.40	14	1.10
1	1.00	15	1.00
2	1.50	16	0.80
3	1.70	17	0.80
4	2.10	18	0.70
5	2.20	19	0.60
6	2.30	20	0.60
7	2.40	21	0.60
8	2.20	22	0.55
9	2.00	23	0.60
10	1.60	24	0.60
11	1.40	25	0.50
12	1.30	26	0.40
13	1.10		

POND 2, Test 2

Hydraulic loading : 2.66 x 10^{-2} m^3/m^2/day

Date experiment started : 22/11/79

Date experiment ended : 20/11/79

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.40	15	1.70
1	0.90	16	1.60
2	1.10	17	1.40
3	1.50	18	1.30
4	2.00	19	1.10
5	2.20	20	1.00
6	2.30	21	0.90
7	2.40	22	0.90
8	2.50	23	0.80
9	2.60	24	0.70
10	2.60	25	0.70
11	2.60	26	0.60
12	2.50	27	0.50
13	2.10	28	0.40
14	1.90		

POND 3, Test 2

Hydraulic loading : 2.66 x 10^{-2} $m^3/m^2/day$

Date experiment started : 28/11/79

Date experiment ended : 18/12/79

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.30	11	1.90
1	1.00	12	2.20
2	1.60	13	2.10
3	1.50	14	2.20
4	1.80	15	2.10
5	2.00	16	2.40
6	2.20	17	1.90
7	2.70	18	2.30
8	2.10	19	2.50
9	1.90	20	2.40

POND 4, Test 2

Note: This experiment was terminated before completion.
P_4 is not shown in Fig. 5.2.

Hydraulic loading : 3.42×10^{-2} $m^3/m^2/day$

Date experiment started : 15/11/79

Date experiment ended : 3/12/79

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.40	10	0.90
1	1.10	11	0.70
2	1.40	12	0.80
3	1.70	13	0.80
4	1.90	14	0.70
5	2.10	15	0.60
6	1.80	16	0.60
7	1.59	17	0.40
8	1.30	18	0.40
9	1.10		

POND 1, <u>Test 3</u>

Hydraulic loading : 3.42×10^{-2} $m^3/m^2/day$

Date experiment started : 9/1/80

Date experiment ended : 2/2/80

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.40	13	1.30
1	1.20	14	1.10
2	1.70	15	1.00
3	-	16	0.90
4	2.10	17	0.80
5	2.40	18	0.80
6	2.30	19	0.70
7	2.00	20	0.70
8	1.90	21	0.60
9	1.40	22	0.60
10	1.50	23	0.40
11	1.40	24	0.40
12	1.30		

POND 2, <u>Test 3</u>

Hydraulic loading : 3.42 x 10^{-2} m^3/m^2/day
Date experiment started : 9/1/80
Date experiment ended :14/2/80

Time after start (days)	Tracer concentration (mls of titrant used	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.30	18	1.50
1	1.00	19	1.40
2	1.20	20	1.30
3	1.30	21	1.20
4	1.40	22	1.20
5	1.60	23	1.00
6	1.90	24	1.00
7	2.00	25	0.90
8	2.00	26	0.90
9	2.10	27	-
10	2.00	28	0.80
11	2.20	29	0.70
12	2.20	30	0.70
13	2.00	31	0.60
14	1.90	32	0.60
15	1.80	33	0.50
16	1.80	34	0.40
17	1.70	35	0.40
		36	0.30

POND 3, <u>Test 3</u>

Hydraulic loading : $3.42 \times 10^{-2}\ m^3/m^2/day$

Date experiment started : 9/1/80

Date experiment ended :18/2/80

Time after start(days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.30	21	1.80
1	0.60	22	1.70
2	0.70	23	1.50
3	0.90	24	1.40
4	1.00	25	1.30
5	1.30	26	1.20
6	1.40	27	-
7	1.60	28	1.10
8	1.70	29	1.20
9	1.80	30	1.00
10	1.70	31	1.00
11	2.00	32	0.90
12	2.00	33	1.00
13	2.10	34	0.90
14	2.10	35	0.80
15	2.00	36	0.90
16	2.10	37	0.70
17	2.00	38	0.50
18	2.00	39	0.50
19	1.90	40	0.30
20	1.90		

POND 4, <u>Test 3</u>

Hydraulic loading : 4.18×10^{-2} $m^3/m^2/day$ Date experiment started : 4/12/79 Date experiment ended : 20/12/80			
Time after start(days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.40	9	1.10
1	1.20	10	1.00
2	1.60	11	0.85
3	1.70	12	0.70
4	1.65	13	0.60
5	1.80	14	0.60
6	1.90	15	0.50
7	1.40	16	0.40
8	1.30		

POND 1, Test 4

(for hydraulic loading 4.18×10^{-2} $m^3/m^2/day$).

Hydraulic loading : 4.56 x 10^{-2} m^3/m^2/day

Date experiment started : 9/1/80

Date experiment ended : 25/1/80

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.50	9	1.10
1	1.00	10	1.20
2	1.60	11	0.90
3	-	12	0.90
4	2.00	13	0.80
5	1.80	14	0.50
6	1.80	15	0.40
7	1.80	16	0.40
8	1.60		

POND 1, Test 4

(for hydraulic loading 4.56 x 10^{-2} m^3/m^2/day)

Hydraulic loading : 4.18×10^{-2} m^3/m^2/day

(INLET IN CORNER)

Date experiment started : 26/1/80

Date experiment ended : 13/2/80

Time after start (days)	Tracer concentration (mls of titrant used)	Time after start (days)	Tracer concentration (mls of titrant used)
0	0.40	10	1.20
1	0.90	11	1.20
2	1.30	12	1.10
3	1.50	13	1.00
4	1.90	14	0.90
5	1.80	15	0.80
6	1.50	16	0.80
7	1.60	17	0.50
8	1.60	18	0.40
9	1.30		

POND 1, Test 4 (INLET IN CORNER)

(for hydraulic loading 3.42×10^{-2} m^3/m^2/day)

PH RESULTS

Days	Date	Influent	Pond 1	Pond 2	Pond 3	Pond 4
0	8/3/80	7.2	7.8	7.6	7.9	7.5
1	9/3/80	7.0	7.6	7.3	7.7	7.3
2	10/3/80	-	-	-	-	-
3	11/3/80	6.8	7.3	7.1	7.2	7.2
4	12/3/80	6.5	7.0	6.9	6.7	6.8
5	13/3/80	6.5	7.0	7.6	6.8	6.9
6	14/3/80	6.0	7.2	7.1	7.0	6.9
7	15/3/80	6.4	7.2	7.2	6.8	6.8
8	16/3/80	7.6	7.1	7.2	6.9	6.8
9	17/3/80	7.7	7.5	7.2	7.4	7.0
10	18/3/80	5.6	7.1	7.1	6.9	6.7
11	19/3/80	7.1	7.6	7.5	7.1	7.0
12	20/3/80	6.2	7.2	7.4	7.1	7.0
13	21/3/80	7.2	7.2	7.3	7.1	7.0
14	22/3/80	6.0	7.2	7.2	7.5	7.2
15	23/3/80	7.0	7.2	7.2	7.1	7.2
16	24/3/80	6.5	7.3	7.3	7.2	7.2
17	25/3/80	6.5	7.5	7.7	7.3	7.3
18	26/3/80	6.0	7.3	7.7	7.2	7.2
19	27/3/80	6.0	7.3	7.6	7.2	7.2
	Average	6.6	7.3	7.3	7.2	7.1

PH VALUES FOR TEST 1 (Mean temperature = 25^{o}C)

Time (days)	Date	Influent	Pond 1	Pond 2	Pond 3	Pond 4
0	29/3/80	7.2	7.4	7.3	7.3	7.4
1	30/3/80	6.3	7.4	7.5	6.8	7.2
2	31/3/80	6.3	7.2	7.0	6.7	6.9
3	1/4/80	6.3	7.1	7.1	6.8	7.0
4	2/4/80	-	-	-	-	-
5	3/4/80	7.0	7.2	7.3	7.3	7.3
6	4/4/80	6.8	7.3	7.4	7.5	7.3
7	5/4/80	6.4	7.4	7.4	7.2	7.0
8	6/4/80	6.7	7.5	7.6	7.2	7.2
9	7/4/80	7.2	7.3	7.4	7.4	7.2
10	8/4/80	7.2	7.3	7.4	7.4	7.5
11	9/4/80	6.7	7.5	7.5	7.4	7.3
12	10/4/80	6.8	7.0	7.1	7.4	7.2
13	11/4/80	6.5	7.3	7.4	7.3	7.3
14	12/4/80	-	-	-	-	-
15	13/4/80	7.2	7.2	7.4	7.2	7.2
16	14/4/80	6.4	7.1	7.4	7.3	7.2
17	15/4/80	7.1	7.1	7.3	7.2	7.1
18	16/4/80	6.8	7.1	7.2	7.2	7.2
19	17/4/80	6.8	7.1	7.3	7.2	7.3
20	18/4/80	6.9	7.0	7.3	7.2	7.2
21	19/4/80	6.9	7.0	7.2	7.1	7.2
22	20/4/80	6.6	7.2	7.4	7.4	7.4
23	21/4/80	-	-	-	-	-

Time (days)	Date	Influent	Pond 1	Pond 2	Pond 3	Pond 4
24	22/4/80	6.5	7.1	7.1	7.2	7.2
25	23/4/80	6.8	6.9	7.3	7.1	7.2
26	24/4/80	6.8	6.9	7.2	7.1	7.2
27	25/4/80	6.9	6.9	7.1	7.0	7.2
28	26/4/80	-	-	-	-	-
29	27/4/80	6.8	7.0	7.2	7.1	7.2
30	28/4/80	6.9	7.0	7.3	7.2	7.2
31	29/4/80	7.0	7.4	7.5	7.2	7.3
32	30/4/80	6.8	7.4	7.5	7.3	7.4
33	1/5/80	7.1	7.1	6.9	7.4	7.5
34	2/5/80	-	-	-	-	-
35	3/5/80	6.2	7.2	7.3	7.3	7.0
36	4/5/80	7.2	7.2	7.3	7.2	7.1
37	5/5/80	6.9	7.2	7.3	7.1	7.1
38	6/5/80	6.5	7.2	7.3	7.1	7.1
39	7/5/80	7.1	7.1	7.4	7.2	7.2
40	8/5/80	6.5	7.2	7.4	7.1	7.2
41	9/5/80	-	-	-	-	-
42	10/5/80	6.3	7.2	7.3	7.1	7.3
43	11/5/80	-	-	-	-	-
44	12/5/80	6.3	7.2	7.3	7.1	7.3
45	13/5/80	6.9	7.2	7.1	7.2	7.6
	Average	6.7	7.2	7.3	7.2	7.2

PH VALUES FOR TEST 2

(Mean temperature = 25^{o}C).

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