MIXING CHARACTERISTICS AND PERFORMANCE

OF STABILIZATION PONDS

BY

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DECLARATION

I, MOSES OLADAPO OLANREWAJU, hereby declare that the studies reported in this Thesis was carried out by me. It has not been previously published or accepted elsewhere for a higher degree. All works, opinions and ideas other than mine have been fully acknowledged by means of references.



Moses Oladapo OLANREWAJU
(1986)

D E D I C A T I O N

This Thesis is dedicated to my brother,

Chief R. Daodu Olanrewaju;

my children:

Jide, Bola, Wale and Bisi;

and my beloved wife,

Felicia.

CERTIFICATION

This dissertation entitled "Mixing Characteristics and Performance of Stabilization Ponds", by MOSES OLADAPO OLANREWAJU, meets the regulations governing the award of the degree of Doctor of Philosophy of Ahmadu Bello University, Zaria, and is approved for its contribution to knowledge and Literary presentation.

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A C K N O W L E D G E M E N T

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ABSTRACT

Influence of hydraulic properties on baffled stabilization ponds performance was studied. The hydraulic model was studied by tracer test. From this test, the Dispersion Number Index, d was evaluated for each of the five ponds at four different hydraulic loadings. The results of this first part of the study formed basis for the second part which is the performance of facultative ponds.

The performance of laboratory baffled facultative ponds was investigated by the measurement of Biochemical Oxygen Demand (BOD); Chemical Oxygen Demand (COD) and Suspended Solids (SS) of synthetic milk waste. Also the changes in Alkalinity, the Dissolved Oxygen (DO) and pH of the ponds were measured.

The results show that for good efficiency, the actual (mean) detention time must be greater or at least equal to the nominal detention time. Also the ratio of the effective volumes of two different ponds of the same dimension and shape is equal to the ratio of their actual (mean) detention times.

Generally, excessive short-circuiting are reduced by some baffle arrangements in the ponds. The results indicate that baffled ponds have low values of Dispersion Index, d and exhibit flow patterns nearer to a plug flow. The overall performance is improved by increasing the number of baffles. Baffled ponds are analogous to ponds in series or a stream.

A significant oxygen demand and SS are imposed on the ponds contents by

algae. In terms of organic removal and bacterial die-off, the results indicate that baffled pond performs better than a plain pond of the same size operating at approximately the same hydraulic and organic loadings.

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NOTATIONS AND ABBREVIATIONS

NOTATION		UNITS (Unless Otherwise stated)
A	Pond surface Area	Hectare
A_c	Cross Sectional Area available for flow	m^2
a	The number of squares occupied by n number of calls	
α	Coeficient of toxicity	
α_d	Index of dead space	
α_s	Index of short circuiting	
β	Coefficient of sulphide contents in ponds	
C	Influent reactant	(mg/L)
C_e	Effluent reactant	(mg/L)
C_i	Influent BOD_5	(mg/L)
C_e	Effluent BOD_5	(mg/L)
C'	Concentration gradient in a river	
C'_i	Influent ultimate BOD	mg/L
D	Coefficient of dispersion	(m^2/t)
d =	D/uL Dispersion Index Number	
d'	depth	m
d*	dilution factor	
	fraction of dead space	
	Dirac delta function	
E_q	quantum energy	
$\bar{F}$	Conversion efficiency	
F'	Molar concentration	
F_i	Molar flow rate	
f	Fraction of light energy utilized for phtosynthesis	
H	Total Energy required for agal growth	KCal.

NOTATION		UNITS (Unless otherwise stated)
h	Extinction Coefficient	
I	Light intensity after passage	lm /m^2
I_o	Light intensity subsequent to passage	lm /m^2
K	Reaction or Removal Rate Constant	per day
K_1	First Order BOD Removal Rate Constant	per day
K_s	A standard BOD Removal Rate constant at an arbititary Environment	per day
K_T	Reaction or Removal Rate at a Reference Temperature T	per day
K	Light Absorption coefficient	
L	Mean Path Length of a Typical Particle in a Reactor	m
L_a	Ultimate Biochemical Oxygen Demand	mg/L
L_e	Load factor	
L_i	Influent BOD	mg/L
L_e	Effluent BOD	mg/L
L_b	Length of travel in baffled pond	m
L_p	Length of travel in plain pond	m
L_t	Length of travel in a pond	m
λ_i	Areal BOD loading	Kg/ha-day
λ_r	Areal BOD removal	Kg/ha-day
λ_{ru}	Areal BOD removal (Unfiltered)	Kg/ha-day
λ_{rf}	Areal BOD removal (Filtered)	Kg/ha-day
λ'_i	Areal COD loading	Kg/ha-day
λ'_r	Areal COD removal	Kg/ha-day

NOTATION		UNITS (Unless otherwise stated)
λ'_{ru}	Areal COD removal (Unfiltered)	Kg/ha-day
λ'_{rf}	Areal COD removal (Filtered)	Kg/ha-day
λ_i	Maximum Areal Loading	Kg/ha-day
N	Number of baffles or cells/100mL	
N_i	Influent Coliforms per 100mL	
N_e	Effluent Coliforms per 100 mL	
n	Number of cells	
N_e	Molar flow rate for out reactant	day^{-1}
n_i	Molar flow rate for input reactant	day^{-1}
O_p	Oxygen production rate	Ka/ha-day
Q	Mean flow rate	L/day
Q_1	Fraction of Q entering active zone	
Q_2	Fraction of Q short circuiting	
Q_p	Pollution flow rate	L/day
Q_r	River flow rate	L/day
q	per capita wastewater flow rate	L/C day
r	Correlation cefficient	
r^2	Coefficient of determination	
S_E	Average Visible solar radiation at elevation, E	Langley
S_o	Average Visible solar radiation at Sea level	Langley
σ	Variance of C-Curve dimensionless	
σ_t^2	Variance of C-curve in time squared	
T	Temperature	^oC
T_m	Mean monthly ambient temperature	^oC
τ	space Time	day

NOTATION		UNITS (unless otherwise stated)
τ_{obs}	Mean Hydraulic retention time observed in actively mixed zone	day
t	Detention time	day
t*	Detention time	day
$\bar{t}$	Mean residence time	day
t_a	Median time or time at which 50% of the tracer is observed at the effluent	
t_e	= t_s Effective detention time, t_e	day
	Standard detention time, t_s	day
t_g	Mean (Actual) detention time	day
t_i	Time interval for the initial indication of the tracer in the effluent	
t_n	Theoretical (Nominal) detention time	day
t_p	Time to reach peak concentration	day
t_n	= t_s/t_n = Standard detention time efficiency	
u	Mean fluid velocity	m/day
V	Theoretical (Nominal) Reactor Volume	m^3
V_a	Active volume	m^3
ΔV	Infinitestinal volume	m^3
$\forall$	Effective Detention Volume	m^3
W_m	Milk weight	mg
ø	Retardation cefficient	
θ	Temperature coefficient	
ψ	Percentage Difference	
η	Efficiency	

ABBREVIATIONS:

BOD_5	Five day BOD	mg/L
BOD_u	Unfiltered BOD_5	mg/L
BOD_f	Filtered BOD_5	mg/L
COD_u	Unfiltered COD	mg/L
COD_f	Filtered COD	mg/L
DO	Dissolved Oxygen	mg/L
erf	Error function	
MI	Morril Index	
OD	Optical density	
SS	Suspended Solids	mg/L
TC	Total coliform	
FC	Faecal coliform	
FS	Faecal Streptococci	

CHAPTER 1

1.0 INTRODUCTION

1.1.0 Waste Management Problems in Developing Countries

A small minority of the population of developing countries lives in houses which are sewered. Information collected by the World Health Organisations (W.H.O.) in 1962 showed that not over ten percent of the people had piped water in their houses and only a fraction of those houses are sewers. Today after a period of over twenty years the improvements have not been all that encouraging as a result of social, economical and political instability in most of the developing nations. Where sewers do exist the main reason especially in rural areas may have been to recover waste water for irrigation. Those people who have neither piped water nor sewerage system available make full use of nearby water courses for drinking, cooking, animal keeping, washing and waste disposal.

The greatest percentage of the source of contamination of water supplies in these developing nations is from domestic wastewater (sewage) that contains waste from human origin. This problem is even worse in the rural areas where majority of the people live. It had been projected that with tremendous investment by 1980s only 50 percent of the people in the developing nations will have access to safe drinking water (Diamant, 1979).

With the present rate of development in the developing nations, the objective of the U.N. World Water Conference Declaration of 1981-1990 to provide sufficient potable water and good sanitation to every country by the year 1990 may not be achieved. A major concern to the international health community is population growth and the resultant inability of man to keep pace with this growth in the provision of an adequate or extension facilities for removing from his immediate environment the wastes he generates.

In most parts of the towns in the developing countries there has been little planning. In Nigeria, for example in cities like Ibadan, Lagos, Zaria, Benin City and Ife the planning is so bad that expenses of installation of sewerage systems apart, the construction of the network of sewers will be accompanied by dislocation of traffic and general disorganization of business and property in the community. These factors might have led to the Lagos Government rescinding its order on mass demolition in early August, 1985. Other states governments are facing the same problem but they are yet to follow that step taken by Lagos Government. The only effective way of tackling the problem in these areas is to combine sewerage with slum clearance and rehousing since it is not economically viable to invest so much on such disorganised planning that may just be temporary. Such slum areas are the hubs of dense population and a lot of politics, finance and problems may be associated

to rehousing problems that must follow slum clearance (Olanrewaju, 1977).

If the existing towns have followed accepted town planning principles, the disorganisation effects and high cost of sewerage would have been less serious. Unfortunately in addition to high degree of urbanisation in the country, the pattern of urbanism is urban sprawl in which the expansion is in lateral direction. The problem associated with this sprawl are longer sewers, more operational cost such as in pumping and acquisition of plant site for the waste treatment.

Other problems of waste management in the developing countries are: education, manpower and technology. Most of the now available waste treatment methods require fairly high technology beyond the present reach of most of developing countries. Where the technology is transferred there comes the problem of manpower. Unfortunately it is very expensive to keep the expatriates just as training the indigenes is exhorbitant. Most of the people in the developing countries lack the education on the need for good sanitation. Education may bring awareness of the importance of good sanitation. So on the part of the people one hopes that that phase of War Against Indiscipline (WAI) campaign launched at the end of July 1985 by the Nigerian Government would go a long way to improve the sanitary situation in the country.

A few consideration need to be given to waste management in developing countries:

(i) As a result of varying customs of personal and community hygiene and because of the greater prevalence of communicable diseases, primary attention must be focused on the way people dispose waste at the source;

(ii) For one reason or the other, majority of the people in the developing countries are neither having sufficient nor good quality water supply in their households. This inadequate supply may be responsible for the higher organic concentrations in the sewage in the developing nations relative to those in the affluent nations;

(iii) Surface waters in most developing countries are not classified as to appropriate usage and little is known of their quality; and

(iv) Most communities in developing country's rural areas and many large cities have at present neither facilities for collecting and disposing of wastewater nor the financial, educational and organisational means to build and operate them.

In Nigeria there is yet not a single town with a central sewerage system which is still the most satisfactory method of sewage disposal. The only one being planned in the country is that of the new Federal Capital, Abuja. In August 1985, at the commissioning of a ₦1.3 million sewage treatment plant having about 12km sewer for a group of flats with a population of 2,000 in Lagos, Nigeria, the Lagos State Government indicated

the government's plan for central sewerage system for Lagos in due course of time. This is a huge amount of money when one considers what the cost would be for a population of over 2 million.

1.2.0 Waste Treatment Methods

There are many waste treatment methods, some requiring very high and sophisticated technology and a lot of financial backings while some are relatively simple processes.
The treatment processes include:

(a) the trickling filter process
(b) the activated sludge process
(c) the oxidation ditch
(d) the aerated lagoon
(e) the septic tanks
(f) the pit latrines
(g) the bucket latrine, and
(h) the waste stabilization pond system.

(a) Trickling Filter is a fixed-film reactor in the shape of a column. Until the mid 1960s trickling filters made of stone were been used.. More recently plastic media like egg crates are been use to increase the height to about 6m (Middlebrooks, et. al. 1980). .The waste-water runs in a thin sheet over the packing material upon which a microbial film(heterotrophic microbes) grows. These organisms extract the soluble organic matter such as CO_2 as a

source of carbon and energy. As a result of the detachment of microorganisms from the film surface, the concentration of the suspended solids (SS) in the effluent from the reactor is in excess of the concentration in the influent. This is because, a trickling filter converts soluble organic matter into an insoluble form which can be removed by sedimentation. Trickling filters are simple, easy to operate, and tolerant to changes in the influent; however, there is no effective way of controlling the quality of the effluent from them.

(b) Activated sludge reactors are generally open tanks about 4 - 5m deep constructed of concrete with vertical side walls, although steel and earthen basins are also sometimes used. Oxygen is supplied by either bubble diffusers or mechanical surface aerators. Major activated sludge types are conventional (CAS), Tapered aeration (TAAS), Step aeration (SAAS), Contact stabilization (CSAS), Completely Mixed (CMAS), Extended Aeration (EAAS), and Pure Oxygen (POAS). All have mean cell residence time, MCRT of between 3-15 days except the extended aeration.

The main role of activated sludge process is the removal of soluble organic matter, however, colloidal and suspended organics will be entrapped in the floc and removed as well.

Although it produces an effluent of high quality at reasonable cost, its main defect is that it is a complicated process requiring relatively sophisticated operation.

Regardless of the reactor configuration, all activated sludge systems share four common characteristics: (i) they employ aerobic flocculent slurries of microbes, (ii) by sedimentation, they separate the microbes from the effluent, (iii) they recycle the microbes from the settler to the reactor and (iv) their performance is dependent on the mean cell residence time (MCRT).

(c) Oxidation Ditch: Oxidation ditch (extended aeration) was developed in Holland by Pasveer (Middlebrooks et al, 1980). The oxidation ditch is an earthen tank of special shape similar to that of a large drainage ditch. Raw wastewater is brought into the tank and aerated for an extended period of time. The organic matter is removed and converted into cell protoplasm by the activated sludge inside the oxidation ditch. These protoplasm would be degraded with sufficient aeration period such as 30 - 60 days mean cell residence time (MCRT).

This method of treatment achieves a high degree of purification. A cage rotor aerator (Kessener brush) is installed inside the ditch to aerate the mixed liquor and circulate the contents. The rotor may operate continuously or intermittently. In case of continuous operation, final settling tanks should be provided. In intermittent operation the ditch functions as both aeration tank and settling tank.

(d) Aerated Lagoons: The term aerated lagoons has been applied to a variety of processes which can be defined further as:

(i) completely mixed aerated lagoon

(ii) facultative aerated lagoon, and

(iii) aerated oxidation pond.

(i) Completely mixed Aerated Lagoon is an activated sludge aeration basin without sludge return where all solids are maintained in suspension. All organic stabilization is dependent upon the suspended bacterial solids. Oxygen is machine derived and algae are not apparent. The types of mixing equipment available develop different process mixing levels for the same power. The large slow moving impellers are more efficient mixing devices than either the fast moving, small impellers or gas diffusion (Kalinske, 1960).

(ii) The facultative aerated lagoon has a degree of mixing which permits heavier solids to settle in the lagoon while biologically treating suspended and soluble organics in the liquid portion. The oxygen transfer is primarily associated with the machine. Algae may be present in small numbers but play only a small or insignificant role in the oxygen supply. Less power is required than for a completely mixed aerated lagoon and higher treatment efficiencies are attainable for wastes having a large portion of organic settleable solids. (Benjes, 1969).

(iii) Aerated Oxidation Pond is essentially a high rate oxidation pond. The oxygen supply is primarily from algae during the warm months. Loadings over 1000 kg/ha per day could be treated. (Benjes, 1970).

(e) Septic tanks are an on-site wastewater disposal means. In their simplest form, they are single storied settling basins in which settling solids are held sufficiently long to undergo or possibly complete digestion. Separation of the decomposing sludge from the flowing sewage in a two-storied structure was conceived by Travis and perfected by Imhoff (Okun et al, 1971). However, the great depth and cost of these two storied structures have discouraged many people. Another set back of this new concept is the impossibility of heating the digestion compartments effectively to keep down their size.

At present septic tank is still very popular in developing countries (Oluwande, 1978). Its construction, operation and maintenance is carried out solely within the confines of the private property apart from the ocassional desludging of the tank. Although septic tank appears simple and apparently cheap on the final analyses it is equally expensive like other treatment processes if not even more. It costs more on a per household basis. Its use is confined to wealthier suburbs. It requires suitable soil structures. The soil must be permeable otherwise the effluent still laden with pathogens may pollute the area.

In the slum areas or in densely populated areas, the open spaces may be difficult to acquire. If not properly vented, there is a danger of being a breeding place for insects. There is also possibility of ground water pollution if not well designed.

(f) The Pit Latrine is relatively the simplest and cheapest method of sewage disposal. Once constructed, it requires neither additional input of water or any energy. Despite the fact that neither sewer nor any other equipment is required for its operation, it can be made to satisfy most of the criteria except the sub-soil ecosystem (Oluwande, 1978).

In order to meet certain demands, the common pit latrine has been modified and some of such modifications are:

(i) Pour flush latrine also known as water seal or Chiengmai latrine;

(ii) the borehole latrine constructed by boring a narrow hole into the ground with a hand auger, and

(iii) the trench latrine which is meant for temporary sites like military and holiday camps. The trench is usually about 60cm deep with the excavated soil forming a ridge along the sides of the trench.

(iv) Ventilated improved pit latrine (VIP) and

(v) Reed Odourless Earth System

Although the system is simple and very cheap, it is still unpopular because of the odour, sanitary habit of defacating on other people's excereta, superstitious belief, flies and

cockroaches breeding, danger of accident, lack of space for the pit construction and danger to underground water in some situations.

(g) In some rural areas bucket latrines are used. This system is very unpopular. The disposal is often offensive and never meets the standard. It is a nuisance to the public, especially during the daily round of the workers usually late in the evening. The open bucket gives rise to odour, and serve as a breeding place for vermin and vector diseases. The bucket is often washed in the immediate vicinity of the residence with the result that excreta is splashed over the payment and road. The psychological and social stigma suffered by the workers is highly humiliating.

(h) Waste Stabilization Pond can be defined as a shallow man-made basin utilizing natural processes under partially controlled conditions for the reduction of organic matter and destruction of pathogenic organisms in wastewaters. It is widely recognized as one of the most economic and effective methods of biological wastewater treatments. It is particularly suitable for developing tropical countries where land is normally available at relatively low cost and where the prevailing climatic conditions are favourable to its operation.

Data on waste stabilization ponds are generally limited in

developing countries. Their design is usually based on the data obtained from elsewhere, especially in the temperate region of the United States. It is apparent that the behaviour of ponds in tropical and sub-tropical climatic regions differs significantly from that in the temperate climate. It is therefore, desirable to accumulate data thoroughly for design purposes in the tropical regions.

1.3.0 Objective of the Study

A number of environmental factors are known to influence the performance of stabilization ponds. Some of the factors are visible solar energy, temperature, organic load and toxicity of the influent materials. Factors that require more information for proper design are the relationship of shapes of the pond and mixing in the pond to its performance. Specifically, few investigations have been carried out on the effects of baffles on the performance of ponds. Hitherto, limited investigations were carried out on baffled anaerobic and facultative ponds (Hove 1964 Nemerow, 1969, Renolds, 1976 and Kilani and Ogunrombi, 1984)

Baffles are physical barriers that direct the flow of wastewater through a pond and therefore alter its hydraulic characteristics and mixing properties. It is the objective of this study to investigate the performance of baffled

facultative pond in the treatment of a synthetic milk waste and to compare it with that of a control plain pond of the same size. Also the hydraulic characteristics of the ponds that may affect the performance are estimated by the use of a tracer technique. Specifically, the study investigates the hydraulic properties of ponds as a function of number of baffles and also the performance of the ponds under varying organic loadings.

CHAPTER 2

2.0 TYPES, ADVANTAGES AND DISADVANTAGES OF WASTE STABILIZATION PONDS

2.1.0 Types

Waste stabilization pond is a complex biological waste-water treatment system which calls for the metabolic activities of bacteria and algae. The two groups of micro-organisms are previously known not to compete with each other for food within their environment. Rather, they depend on each other for their activities and survival (symbiosis).

The major types of waste stabilization ponds include:

(i) the anaerobic pond

(ii) the facultative pond,

(iii) the maturation pond, and

(iv) the mechanically aerated pond.

2.1.1 The Anaerobic Pond

The anaerobic pond is a deep earthen pond designed for the treatment of high-strength wastewater in the absence of dissolved oxygen. It is most advantageously employed as a pretreatment unit of a very strong wastewater with high solids content.

Anaerobic ponds have been found to tolerate fairly high loadings and give high BOD removals (McGarry & Pescod, 1970). Removal of BOD is brought about both by sedimen-

tation and anaerobic digestion. Heavily loaded ponds tend to have low pH which may result in the release of H_2S if the ponds contain sulfide and hence odour occurs. Many methods have been adopted to handle odour problem such as the use of chemicals (e.g Sodium nitrate in canning industry) and recirculation (Abott, 1962 and Canham, 1968).

The level of BOD loading is very important in the transformation occuring in anaerobic ponds. Neither nutrient deficiency nor the presence of toxic substances limit the development of satisfactory microbial population. In the tropics, the detention time and pH are the most important (Parker , 1950; Amalorpavam, 1963 and Gloyna, 1965). These are further substantiated by using synthetic sewage in Bangkok (Alivio, 1968) and with anaerobic pond in treating of tapioca starch waste (Uddin, 1970). The higher temperature in the tropics allows anaerobic fermentation to proceed at a rapid rate at pH levels down to 6.0 which is lower than the minimum practical limit suggested by Oswald (Oswald, 1968).

2.1.2 Facultative Ponds

Facultative ponds are the most widely used type. The ponds behaviour is determined by a complex interaction of physical and biochemical processes. Fig. 2-1-1 shows the main biochemical process features of a facultative stabilization

pond system.

When some waste is discharged into facultative pond a portion of the organic load enters into the liquid body of the pond and the settleable part settles on the bottom of the pond to form sludge layer. In the sludge layer the degradation action is anaerobic; in the pond liquid it is mostly aerobic and or facultative.

Six main biological transformations occuring in the stabilization ponds are (Oswald, 1968):

$$6\ (CH_2O)_x + 5O_2 \rightarrow (CH_2O) + 5CO_2 + 5HO_2 + \text{Energy} \qquad 2\text{-}1\text{-}1$$

$$5\ (CH_2O)_x \rightarrow (CH_2O) + 2CH_3\ COOH + \text{Energy} \qquad 2\text{-}1\text{-}2$$

$$21/2\ CH_3COOH \rightarrow (CH_2O)_x + 2CH_4 + 2CO_2 + \text{Energy} \qquad 2\text{-}1\text{-}3$$

$$(CH_2O)_x + CO_2 \rightarrow 2(CH_2O)_x + O_2 + HO_2 \qquad 2\text{-}1\text{-}4$$

$$(CH_2O)_x + CO_2 + 2H_2S \rightarrow 2(CH_2O)_x + H_2O + 2S \qquad 2\text{-}1\text{-}5$$

$$3/2\ CH_3COOH + SO_4 \rightarrow (CH_2O)_x + 2CO_2 + H_2S + 2OH + \text{Energy} \qquad 2\text{-}1\text{-}6$$

$(CH_2O)_x$ is used to indicate carbohydrate or microbial cells. Reaction 2-1-1 occurs in the presence of oxygen and reaction 2-1-4 produces oxygen by algal photosynthesis. Reactions 2-1-2 and 2-1-3 occur under anaerobic conditions at various depths of the lagoon. Purple sulfur organisms are able to utilize a variety of amino and fatty acids for growth and energy (May and Stahh, 1971; Holme & Vennes, 1970).

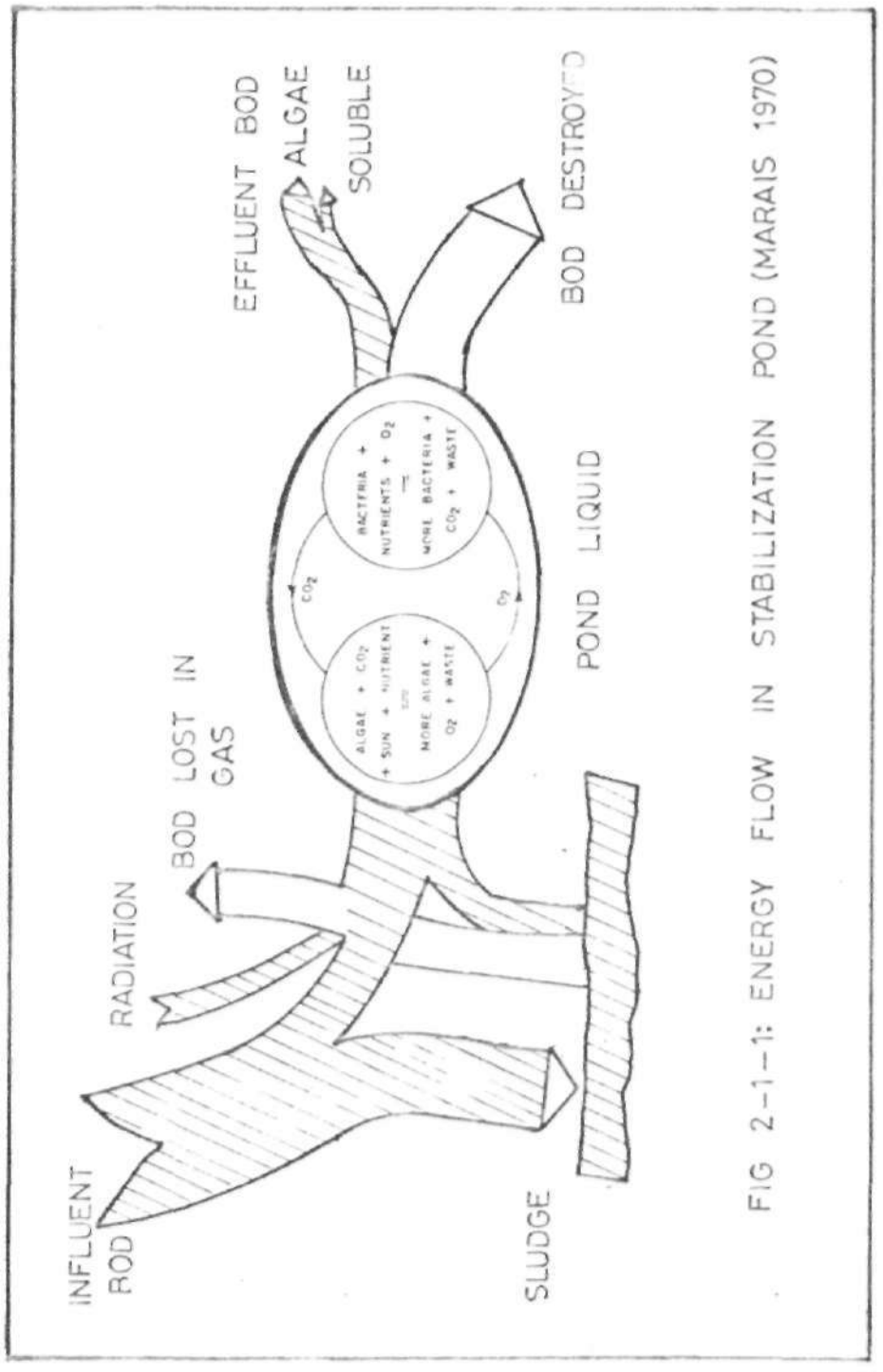


FIG 2-1-1: ENERGY FLOW IN STABILIZATION POND (MARAIS 1970)

In addition the organisms utilize hydrogen sulfide and CO_2 in the photosynthetic process - reaction (2-1-5). Sulphate reducing bacteria are capable of utilizing fatty acids and sulphate with the production of CO_2 and hydrogen sulfide - reaction (2-1-6) (Postgate, 1965).

2.1.3 Maturation Pond

The maturation pond is a low rate organic loading stabilization pond designed to provide for secondary effluent polishing and seasonal nitrification. Operationally, the residual biological solids are endogeneously respired, and ammonia is converted to nitrate using the oxygen supplied from surface reaeration and from algae. A detention time of 18 to 20 days has been suggested as the minimum period required to provide for complete endogeneous respiration of the residual solids.

Its other function is to reduce the number of pathogens through extended detention time in an environment hostile to the microorganisms. It also achieves nitrification of the treated effluent especially as it is wholly aerobic. However, BOD removal in a maturation pond is small (Onuoha, 1980). A maturation pond usually has the same depth as the preceeding facultative pond. Due to its shallow depth, the destruction of faecal bacteria and viruses is quicker. The manner in which pathogens are

destroyed in a maturation pond include the following:

(i) the cysts and ova of intestinal parasites, due to the long detention time, settle to the bottom of the maturation pond where they eventually die (relative density = 1.1);

(ii) solar radiation probable germicidal effect on both bacteria and viruses;

(iii) viral adsorption to static surfaces and thus removed from the effluent, and

(iv) if the detention period in a maturation pond exceeds the normal survival time of an infectious virus, the virus dies in the maturation pond (Mara, 1975; Gloyna, 1976).

2.1.4 Aerated Ponds

The term aerated pond conventionally describes ponds which use mechanical or diffused aerator to avoid the problems associated with the use of algae. The introduction of aeration to a waste stabilization pond make the stabilization pond a more reliable waste treatment facility. The aeration is introduced to the pond either through a mechanical aerator installed at the surface of the pond or by an air compressor that injects air into the pond through tubing installed at the bottom of the pond.

Aerated ponds are typically about one-fifth the size of a conventional stabilization pond but often require very high power input . Aerated pond are always followed by a quiescent, second-stage pond to remove the suspended solids (SS) from the aerated-pond effluent.

The flow characteristics, amount of turbulence and mixing and efficiency of the oxygen transfer equipment determine the length of the retention time. The percentage of the total organic loading stabilized aerobically is determined by the amount of turbulence available to maintain the solids in suspension. An aerated pond can thus be designed to handle high organic loading such that the size of pond can be decreased considerably. This reduction in size or land requirement permits more flexibility in site selection. The aerated pond maintains aerobic conditions at all times and thus eliminates the probable odour problem experienced in conventional stabilization pond. (Gloyna and Espino; 1969; Pohl, 1970). The degree of oxygen saturation in a well aerated pond will vary from 90 to 98 percent. (Gloyna, 1976).

The amount of oxygen supplied by mechanical aeration is given by equation 2-1-7 (Gloyna, 1976).

$$N = N_o \frac{C_{sw} - C}{C_s} (\alpha) \theta^{T-20} F_A \quad \ldots\ldots\ldots 2\text{-}1\text{-}7$$

where:

N = oxygen supplied by mechanical aeration (oxygen per hour)

N_o = characteristics of aeration, 4 to 5lb oxygen per horse power per hour. (This value decreases with pond size).

C_{sw} = oxygen saturation level in the pond at some temperature, T (^oC).

C = level of Dissolved Oxygen in pond (mg/L).

C_s = saturation of distilled water at 20^oC (mg/L)

α = oxygen transfer characteristics of the wastewater to water

θ = temperature reaction coefficient

T = Pond operating temperature (^oC)

F_A = altitude correction factor for altitude above 1.2×10^3m.

The number of aeration units selected will depend on the size of the unit, and this in turn depends on the basin geometry. The maximum spacing of aerators is in the neighbourhood of 60 to 75m.

2.2.0 Advantages of Stabilization Ponds

There are several advantages of stabilization ponds; the most striking advantages are:

(i) They afford an opportunity for small communities to

share in the comfort and satisfaction of a sewerage system which provides an excellent treatment. Properly designed and operated ponds have provided a certain degree of purification to many communities at a cost comparable to individual household units with greater benefit to the home owners.

(ii) The goals of waste treatment are concerned with two processes, (a) removal or greatly reduction of pathogens and (b) transformation of utilizable inorganic and organic substrates into stable end products. This transformation to acceptable end products (in stabilization ponds) requires the combined metabolic activities of many microbial agents and these are provided by stabilization ponds. Properly laid out for size and embodying a few simple details, stabilization ponds absorb and destroy the odours of sewage leaving nothing worse than the smell of a swamp. The effluents are permanently stable with BOD values as good or better than those of Trickling Filter effluents. As a destroyer of sewage bacteria, they possess almost unbelievable efficiency. Final overflow from ponds designed for 15 days detention time showed 5-50 E. Coli per mL compared to something like 100,000 per mL in the original sewage (Caldwell, 1946).

(iii) Among the areas where stabilization ponds can find useful application is in domestic wastes treatments. Canadian ponds receiving organic loadings from 11 - 50kg BOD/ha per day gave removal efficiencies of from 65 to 90 percent in the summer and from 0 to 55 percent in the winter (Fisher, et al, 1968).
Coliform reductions of over 99 percent were observed Other data on BOD reduction in domestic wastes stabilization ponds have been reported (Giesickle and Zeller, 1936; Caldwell, 1946; Gotaas and Oswald, 1954; Allen, 1955; Malchow-Moeller, 1955; Neel and Hopkins, 1956; Mertz, 1957; Oswald, et al., 1957; Lackie, 1958; Kabler, 1959; Parker 1959; Marais, 1961; Watson, 1962; Geldreich, et al, 1964; Hodgson, 1964; Middlebrooks, 1965; Thirumurthi, 1967, 1969; Kothandaraman, 1968). More recent useful application of ponds for domestic wastewater are also reported (Gloyna, 1971; Parhad, 1976; Bradly, 1982; Songonuga, 1982; Mara, 1979).

(iv) Another major advantage of stabilization ponds is in the treatment of industrial and agricultural wastes or most often combined wastes. In the US back to 1962 about 827 stabilization ponds were being used by

industries (Porges, 1963). Canning industry utilized the greatest number of about 29 percent with meat and poultry industry ranking second with about 20 percent. Porges concluded that experimental data were not sufficiently definitive to indicate pond efficiency but operational data suggest many successful operations.

In the Canning industry the use of stabilization pond has been confined to mainly holding the processing seasons waste with the ultimate addition of sodium nitrate as an electron acceptor (Canham, 1968). Most of these ponds are not loaded at rates acceptable to facultative ponds. Ponds are used in meat processing industry (Steffen, 1968). In pulp and paper industry they are often used as secondary treatment device (Gellman and Berger, 1968).

Organic industrial waste treatment by lagoon in California had been investigated (Cooper, et al, 1965). Rendering plant wastes were discharged into anaerobic lagoon with an average load of 255kg BOD/ha/day and detention times of about 160 days. About 88 percent reduction was observed. Another pond (facultative) was loaded at about 17kg BOD/ha/day with reductions in the BOD in the range of 78 percent. Overall BOD reduction in the two ponds in series was about 98 percent. The anaerobic pond had between 10^7 and 10^8 purple sulfur (Thiopedia - rosea) organisms per mL.

Algal counts ranged from 10^5 to near 10^7 of Chlorella Vulgaris between May and July in the facultative pond. A variety of other heterotrophic bacteria were identified and were in the range of 10^5 total viable organisms per mL.

Over 90 percent BOD removal had been reported in poultry processing wastes with loadings of 75kg BOD/ha/day (Gilde, 1967). A 92 percent efficiency was reported in a 2-stage pond (Nemerow, 1967). The first pond loaded at 1000kg BOD/ha/day with 7.4 days detention time gave 72.5 percent while the second (about 0.62m deep) showed a 70 percent reduction.

(v) Simplicity of operation and economy have been some of outstanding virtues of stabilization ponds (Giesickle, 1936; Gillespi, 1944; Caldwell, 1946; Malcho-Moeller, 1955; Lackie, 1957 and Mertz, 1957). Compared with conventional treatment methods ponds are much cheaper to build and since they do not require the attention of highly skilled staff, they are also cheaper and simple to operate. Tables 2-2-1, 2-2-2, 2-2-3and Figures $_{2\text{-}2\text{-}1}$ and 2-2-2 compare the cost of different conventional waste treatment works. In all these, stabilization ponds have been established to be the less expensive sewage treatment plants.

Table 2-2-2 shows a comparison of the actual contract costs including construction, land and outfall sewer costs, while Table 2-2-3 shows the land costs for treatment works (Gloyna, 1971).

TABLE 2-2-1: COMPARISON OF CONSTRUCTION COST FOR DIFFERENT TYPES OF WASTE TREATMENT IN US (DOLLARS) (GLOYNA, 1971)

POPULATION	CAPITA COST US ($)		ANNUAL RUNNING COSTS US ($)	
	CONVENTION PURIFICA-TION WORKS	STABILIZATION PONDS (NO BOTTOM SEALING	CONVENTIONAL WORKS	STABILIZATION PONDS
240	86.80	5.60	-	0.84
900	44.80	5.60	-	0.56
1,000	50.40	5.60	-	0.42
3,000	36.40	5.60	4.20	0.42
5,000	33.60	5.60	-	0.42
10,000	21.00	5.60	-	0.42
30,000	22.40	5.60	2.80	0.42
50,000	16.80	5.60	-	0.42
200,000	22.40	5.60	-	0.42
1000,000	9.80	5.60	0.84	0.42

TABLE 2-2-2: COMPARISON OF CONSTRUCTION COSTS OF 262 WASTE TREATMENT PLANTS IN THE MIDWESTERN STATES OF THE USA

Population' (10^3)	*Cost for Design Population Equivalent (US Dollars) Primary Settling & Digestion Plants	Secondary Biological Treatment Plants	Secondary Waste Stabilization Ponds	Raw Sewage Ponds
0.10				
0.10	77.91	127.30	103.20	29.36
1.00	40.05	64.83	53.34	18.52
10.00	20.58	333.02	27.04	11.69
100.00	10.58	16.82	13.71	7.37

*Cost per capita

TABLE 2-2-3: LAND COSTS FOR WASTE TREATMENT PLANTS IN THE MIDWESTERN STATES OF THE USA (1957-1959)

TYPE OF PLANT	COST IN US DOLLARS FOR POPULATION EQUIVALENT
Primary Treatment	1.25
Secondary Treatment	1.20
Raw Sewage Waste Stabilization Ponds	3.10
Secondary Waste Stabilization Ponds	2.30

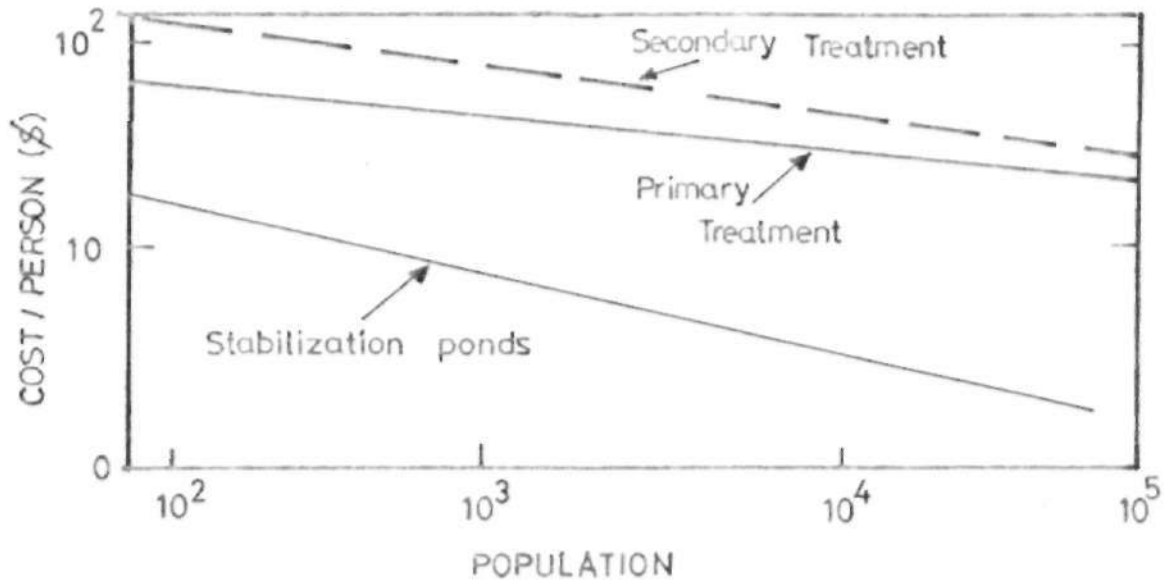


FIG 2-2-1: POPULATION SERVED (DESIGN CAPACITY)

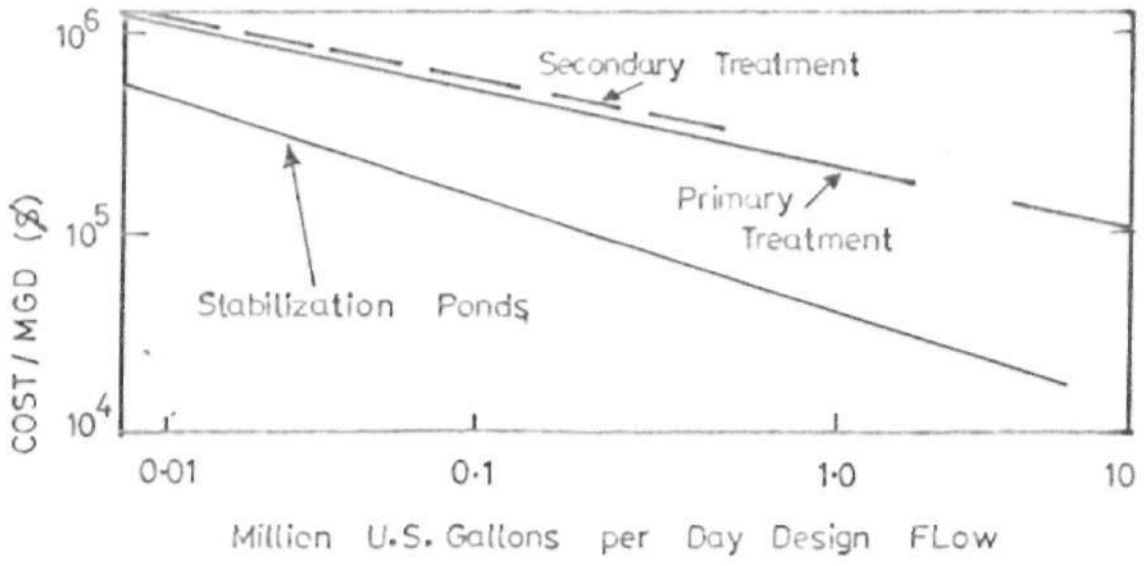


FIG 2-2-2: COMPARISON OF CONSTRUCTION COSTS FOR DIFFERENT TYPES OF WASTE TREATMENT IN U.S. DOLLARS (GLOYNA 1971)

There appears to be no upper limit to the size of community that can be served by ponds, except as economically limited by land costs. The size of any one pond has finite limits presumable; but with split flows and parallel operation the largest of cities could use ponds. However, at present land cost should not be a problem in many places in developing countries. The flexibility in design and operation of stabilization ponds allows for reclamation of the land area for some other purposes in future.

(vi) For many centuries ago in Asia, the use of stabilization pond had been known not for waste treatment but rather for fish culture little realising their importance as sewage treatment facility (Falck, 1936; Mortimer, 1954; Gloyna and Hermann, 1957 and McKinney, 1982). The most promising specie of such fish to be cultivated on both aesthetic and hygienic point of view would be those that can be used for purposes other than direct consumption (Fitz-gerald, 1958 and Steele, 1955). Fish like several species of top minnow have been cultivated in ponds in the US for mosquitoes and other aquatic insects control (Gielsickle, 1936 and Caldwell, 1946).

(vii) Other possible economic crops of stabilization ponds are the algae. They are considered a valuable source of protein. However, unlike crops of fish, harvesting of algae is very cumbersome and expensive (Oswald, 1955;

Steele and Gloyna, 1955; Gloyna, 1957 and O'Brien, 1973).

(viii) The use of stabilization pond effluents for irrigation especially in arid regions has been of great advantage in agriculture. In Israel, properly managed effluent has been used exclusively by farmers for irrigating industrial crops. Sewage water containing chlorides in excess of 300ppm is usually considered unsuitable for irrigating some crops (Watson, 1960). A satisfactory sodium absorption ratio (SAR) of not more than 3.5 is an important factor in determining the suitability of an effluent for irrigation.

(ix) It has been found that a system comprising several cells of ponds in series is very efficient for the removal of pathogenic organisms (Gloyna and Davis, 1972; Dissanayake, 1981 and Arthur, 1981).

(x) Algae harvested from the waste stabilization ponds can be a source of protein especially for poultry and animal feeds. However, the development of an economic method of harvesting algae is still a problem (Berry, 1961; Pipes, 1961; Golueke and Oswald, 1965; 1967; Van Vuuren and Van Duuren, 1965; Foess and Borchardt, 1969; McGarry, 1970; Bare et. al., 1975 and Reid, 1976).

2.3.0 Disadvantages of Stabilization Ponds

As numerous as the advantages of ponds are, they have their own disadvantages. These include:

(i) Like any treatment unit when overloaded, the effectiveness of stabilization ponds can be expected to suffer. Also there tends to be a gradual deterioration in the effluent quality as ponds age.

(ii) The required land areas and siting for ponds preclude the economic use of such facilities in many localities.

(iii) Odour problem is difficult to control especially spring odour and odour due to industrial discharge.

(iv) Improper construction and operation of ponds may result in groundwater pollution.

(v) Algae in stabilization ponds may be a problem. Excessive algae may upset the ecosystem in the receiving stream and this may result in eutrophication apart from impairing the biological and recreational uses. Fish may die in large numbers especially at night because the excess of algae cells introduced from the pond re-absorb too much oxygen at night. Dissolved Oxygen (DO) of about 5ppm is desirable at all times in fish ponds while at least 3ppm is said to be essential to maintain fish life (Watson, 1961). Other problems of algae are associated with odour and taste resulting from the oil they produce upon death. In addition algae may contribute considerable BOD to the receiving stream.

(vi) The mosquitoes, insects, flies and a few other aquatic organisms are potential danger to man. In the tropics or for improperly designed ponds, these are favourable

environment for breeding these creatures. Mosquitoes are always prevalent especially in summer months. Mosquito production is related to overhanging vegetation, the type of wastes, DO content, available food and presence of parasites and predators. Any lagoon change that causes floating mats of algae allows for oviposition sites for mosquitoes (Smith and Enns, 1967). Culex mosquitoes were found in large number in lagoons in Washington State (Mykeebush and Harmston, 1962).

Control of mosquitoes was achieved by keeping the lagoons free of emergent aquatic or flood terrestial vegetation. Control of Culex pipiens were obtained by applying simazine for weed control and malathion for mosquito control (Mack, 1962). Organophosphates were found to be more effective than chlorinated hydrocarbons for larvae control. The malathion was not found to have any effect on lagoon bacteria or BOD reduction. Several insecticides such as dusban, naled, abate and fentum at concentration of about 1mg/L were effective in controlling Culex mosquitoes and were neither bacteriocidal nor phycocidal (Steelman, 1967).

CHAPTER 3

3.0 LITERATURE REVIEW

3.1.0 Historical Background of Stabilization Ponds

Quite a number of articles have described the history of the application of stabilization ponds in sewage treatment. However, in the past people were not interested in stabilization ponds because of low incentive from such engineering scheme. Actually, the use of stabilization ponds a few decades ago seemed to be more by accident than design (Giesickle and Zeller, 1936 and Pearse 1948). Little engineering research went into the construction of early ponds resulting in the failure of some of them. The geometry and loading of the ponds varied. Neither legislation nor effluent standards were set up.

By early 1960s there were about 2000 stabilization ponds in the United States for treating domestic wastes and possibly an equal number for the treatment of industrial or agricultural wastes (Porges, 1963). A few design recommendations have been made especially by various health departments in the United States. These have been in terms of parametric standards; as a result pond design formulations are now available (Hermann and Gloyna, 1957; Oswald, 1963 and Marais, 1966).

In 1901 the city of San Antonio, Texas, constructed an impoundment (Mitchell Lake) of about 275 hectares with an average depth of about 1.40m. As a result of the successful operation of this impoundment other cities in Texas, California, North Dakota and a few other places in the USA started to apply ponds as a means of sewage treatment (Caldwell, 1946 and Gloyna, 1971).

In 1924, in an attempt by Santa Rosa in California to find a cheaper means of handling their sewage, uncovered gravel beds to act as filter prior to discharging waste water into the then highly polluted Santa Rosa Creek (Gillespie, 1944). Unfortunately the bed became clogged and an impoundment of sewage to about 0.90m resulted. Fortunately on the other hand, the overflow effluent was found to resemble that from Trickling Filter (Gloyna, 1971).

Later that same year the town of Vacaville, California, applied to the State Bureau of Sanitary Engineering for permission to impound septic tank effluent (Caldwell, 1946). Probably with the motivation from that of Santa Rosa, the application was granted and ponds were also built at Vacaville. Five years later stabilization ponds were designed and built for the town of Sonoma, California to treat septic tank effluent. However, these were eventually abandoned because of over-loading from individual wastes.

The first pond in North Dakota came into operation in 1928 and operated for over thirty years (Svore, 1960). The first engineered stabilization pond started operation in North Dakota in 1948 and by 1960 there were over 100 ponds in North Dakota and roughly 300 in Texas.

Although stabilization ponds have been known in Asia for centuries, data on their performances are available only for the last 50 years (Caldwell, 1946).

The earliest available data being from various ponds in Germany. After a period of field studies, the development of rational design criteria was initiated. From 1950 there have been significant research and field data (Gotaas, et al, 1954; Hermann and Gloyna, 1958 and Parker, 1959).

There were successful experiences in Australia (Parker, 1962); New Zealand (Collom, 1965); Israel (Wachs, 1961 and Watson, 1962); Brazil (Azevedo-Netto, 1967); in South Africa, (Marais, 1961; Shaw, 1962 and Stander et al 1968) India (Jayangouda et al., 1970) and in Canada (Fisher et al., 1968).

3.2.0 Factors Affecting The Performance of Stabilization Ponds

Factors affecting stabilization ponds fall into four broad classes, namely:

(i) climatic or natural factors which are almost beyond the designers control and include radiation, temperature,

rainfall, humidity and wind; (ii) design or physical factors which unlike the natural factors are controllable and the degree of control is what makes the design quality or standard, these include hydraulic properties of the reactor (mixing), configuration (shape and size) and depth; (iii) chemical factors such as pH, Dissolved Oxygen (DO) and strength of the waste; and (iv) microbiological factors - the algae-bacteria symbiosis.

By the nature of stabilization ponds, most of these factors are strongly inter-related. For example the chemical and microbiological factors are features of the natural and physical factors. Also when one considers the influence of wind on the pond, then mixing comes into play just as radiation and temperature can not be separated from pH and Dissolved Oxygen (DO). Some of these factors are discussed below.

3.2.1 Aerial Organic Loading

Proper loading of the pond is very essential for satisfactory performance. The effect of overloading can be illustrated by Table 3-2-1 (Gloyna, 1971). As the organic loading increases, the percentage BOD removal reduces even under the same climatic condition; in India similar trend was observed by Jayangouder (Jayangouder et al., 1970).

A study of loading on performance of stabilization ponds was made in Australia. Loading up to 75kg BOD/ha/day gave 90 to 100 percent reduction, loading higher than this from 118 to 134kg BOD/ha/day, gave only 30 to 50 percent removal. An average of

TABLE 3-2-1: ORGANIC LOADING APPLIED AND REMOVED
(JAYANGOUDAR, ET AL., 1970)

Organic Load Applied (kg/ha-d)	Percentage of BOD Removal			
	Monsoon	Post-Monsoon	Cold Monsoon	Hot Weather
112	87.0	-	78.0	87.8
168	81.0	95.0	73.5	82.0
224	77.0	82.5	69.5	78.0
280	74.0	74.0	66.5	74.0
336	73.0	71.0	62.5	70.0
392	72.0	68.0	59.0	66.0
448	70.5	65.0	56.0	64.0
450	70.0	64.0	53.5	61.0
560	69.0	63.0	51.0	60.0
617	68.0	62.0	50.0	58.5
673	67.0	61.0	48.5	57.0
730	66.5	60.5	47.0	56.0
785	65.5	60.0	46.0	55.0
897	64.0	59.0	45.0	54.0
Average				
480	71.8	68.1	57.6	65.3

78kg BOD/ha-day, was then chosen for design. Whereas in Ocean side, California a load of 50kg BOD/ha-day was found to be a safe limit for a pond (Samson, 1955). Sometimes it is difficult to reconcile theory with experimental finding as illustrated by Table 3.2.1 where the average organic loading is 480kg BOD/ha-day with annual average removal efficiency of 66 percent while rational design criteria results by Jayangoudar show a value of 190kg BOD/ha-day from actual data and 280kg BOD/ha-day from theoretical value. The latter is nearly half and the former almost one third of the experimental value (Jayangoudar et al., 1970).

Another index of maximum loading limit is the surface tension. As the pond loading approaches the limit, the surface tension apparently decreases the effect of wind waves on the pond surfaces. Hence waves on windy days is indicative of load limit and this is termed "Wave Index" (W.I.) (McKinney, 1982).

Other places where pond loadings were investigated are South Dakota with 56kg BOD/ha-day loading and Delaware poultry processing with loading of 240kg BOD/ha-day. BOD removal was directly related to loadings; this relationship only allows definitive information on loadings and it does not appear applicable to temperate region (McCarry and Pescod, 1970). In the tropics, seasonal variations have little effect on BOD reduction and high BOD loadings are possible year round.

Lagoons in Colorado functioned without odour under loading condition of 11kg BOD/ha-day with 1.22 to 2.19m operating depths (Svore, 1968). Increasing the loadings to about 22kg BOD/ha-day caused persistent odour problems. At the same time hydraulic problems were experienced with loadings of less than 22kg BOD/ha-day. Operational problems were associated with adequate liquid depth not being maintained and primary solids not evenly distributed.

Loadings of 56kg BOD/ha-day in Southern Kraft Mills have shown BOD removal efficiency of about 85 percent. Loadings above 56kg BOD/ha-day have resulted in reduction from 23 to 55 percent. Multiple ponds covering 271 hectare reduced a total BOD load of 22000kg/day by 95 percent (Fitzgerald et al., 1958).

3.2.2 pH and Dissolved Oxygen (DO)

As algal metabolism increases, the pH increases because CO_2 is removed causing a shift in the carbonate equilibrium and then pushing the pH upward. Nevertheless an alkaline environment is necessary for successful anaerobic and facultative stabilization ponds operations (Kilani, 1985).

Some chemical equations were presented by Middlebrooks et al. to illustrate the carbonate buffering process that control the pH in ponds (Middlebrooks, et al., 1980).

$$CO_2 + H_2O \rightleftarrows H_2CO_3 \rightleftarrows HCO_3^- + H^+ \qquad 3\text{-}2\text{-}1$$

$$H(HCO_3)_2 \rightleftarrows H^{++} + 2HCO_3 \qquad 3\text{-}2\text{-}2$$

$$HCO_3^- \rightleftarrows H^+ + CO_3 \qquad 3\text{-}2\text{-}3$$

$$CO_3^= + H_2O \rightleftarrows HCO_3^- + OH^- \qquad 3\text{-}2\text{-}4$$

$$OH^- + H^+ \rightleftarrows H_2O \qquad 3\text{-}2\text{-}5$$

Algal photosynthesis has great impact on the equilibrium of carbonate buffering process. The close relationship between pH and photosynthesis results in diurnal variation in pH of pond contents (Gloyna, 1971 and Mara, 1978). The autotrophic activity (A - activity) is generally always higher than the heterotrophic activity (H-activity) hence, the bicarbonate ions present in equations 3-2-1 and 3-2-3 dissociate into CO_2 to meet the algal demand and the alkaline hydroxyl ion which increases the pH value as illustrated by 3-2-6.

$$pH = pK_2 + \log \frac{CO_3}{HCO_3} \qquad 3\text{-}2\text{-}6$$

As photosynthesis decreases, the rate of CO_2 extraction by algae decreases and dissociation of bicarbonate ions is reduced or reversed and thus falls the pH (Oswald and Gotaas, 1957; Pipes, 1962 and Mara, 1978).

An increase in Dissolved Oxygen usually accompanies high pH and like the pH, always follows a diurnal variation (Marais, 1970 and Mara, 1978).

3.2.3 Toxicities

Adequate concentrations of certain elements are vital and essential

for microbial growth but these may become toxic to the organisms if present in higher concentrations. Toxicity may interfere greatly with the enzymatic system of ponds. It may reduce metabolic rate and impend the optimum microbial growth. Stabilization ponds failure to treat industrial waste effectively may sometimes be due to the presence of toxic substances. Although the waste to be treated in ponds should contain nutrient for microorganisms, it should also be free of any material that is toxic to them (Gotaas, 1954 and 1955 and Kilani, 1985.). Sulfides concentration of 6-7 mg/L had been found to be toxic to algae (Tschorter, 1967 and Gloyna, 1968). A variety of organic acids, alcohols and pesticides were studied in some highly loaded lagoons to determine their toxicities on chlorophyl production (Gloyna, 1968). Gloyna suggested the inclusion of these toxic agents in any estimation of algal activity in lagoons.

Several studies have concluded that in biological waste treatment, bacteria have viral inactivating activity (Kelly, 1961; Erlich, 1966; Pienta, 1966; Christie, 1967 and Shuval, 1970). However, the chemical nature of viruses makes them difficult to control with chemical agents. The only effective method of combating viruses has been through the production of anti-bodies which are protein materials that react with the viruses and prevent their reproduction (McKinney, 1962).

3.2.4 Temperature

In the past most of the reports had focused on the biological aspect of the ponds in terms of the reaction rate (Lamanna & Mallette, 1959; Hok, 1963; Cody and Tischer, 1965; Wentz, 1967; Slanetz , 1970 and Shuval 1970). . And recently the influence of temperature has been studied extensively (Marais, 1974 ; Parhad, et al., 1976; Arthur, 1981 and Kilani, 1985). It has been shown that reaction rate, K is temperature dependent and is given as

$$K_T = K_{To} \quad \theta^{T - 20} \qquad 3\text{-}2\text{-}7$$

where K_T is the value of K at temperature, T, K_{To} is the value of K at a reference temperature (usually 20°C) and θ is the temperature reaction coefficient. According to Marais (Marais, 1974)

$$K_T = 2.6\ (1.19)^{T - 20} \qquad 3\text{-}2\text{-}8$$

This is for facultative pond and is valid between approximately 5°C and 21°C. Reduction of coliform Salmonella and enteric viruses in ponds in series was studied (Slanetz, 1970). Most of the culture of Salmonella were negative while many of the viruses cultures were positive particularly during the colder months in comformity with theory that viability of viruses increases inversely with temperature.

In another investigation Salmonella and Shingella were studied in stabilization pond (Cody and Tischer, 1965). They indicated that Shingella were never detected in the ponds studied but

Salmonella were present in a considerable amount. While in another report 99.9999 percent of Salmonella abortis equivalent were removed in 41 days at 10°C and between 20 and 30°C an equivalent removal was observed in 6 days (Hok, 1963). Another study reported that salmonella survive within 3 to 5 days in summer and 2 to 4 weeks in winter (Hsu and Kruse, 1967).

In a study of oxidation of a variety of substrates by lagoon population as a function of change in temperature it was discovered that oxidative capability of microorganisms was greatest during the summer (Halverson et al., 1968). According to the Vant Hofff-Arrehenius relationship, there is an approximate doubling of biological activity for every 10°C increment (Lamanna and Mallette, 1959).

Probably due to temperature shock stabilization ponds have proven to be effective means of bacterial and viral inactivation or removal. At a relatively low temperature, a 99.99 percent of coliform reduction had been found in five experimental ponds (Okun, 1962). Maturation ponds gave 99.6 percent Faecal Coli reduction in summer and 96.8 percent in winter (Drew, 1966). 99.98 percent reduction of E. Coli in stabilization pond was also reported in summer by Coetzee and Fourie (Coetzee and Fourie, 1965). As a result of ease with which this Faecal Coliform is eliminated, due to the temperature gradient, especially between the warm human intestinal tract and cool liquid environment, they considered this organism to be an infallible

indicator of pathogenic organism in such pond effluents just as a few others who considered streptococcus faecalis and clostridium welchii more reliable indicator of pathogens than Faecal coliform (Malchow-Moller 1955).

At a state prison dairy farm in the USA, a reduction of 85.9 percent in coliform bacteria in winter and 94.4 percent in autumn were observed (Geldrech et al., 1964). Faecal Coliform reductions was found to be greater than 87.9 percent while Faecal Streptococcus reductions were over 97.4 percent. The effect of temperature on feasible aerial organic loading rates has not been thoroughly investigated and applied to design standards on a world wide scale. The most recent reports are those given by Arthur and Kilani (Arthur, 1981 and Kilani, 1985). Both are of the opinion that temperature as well as detention time have influence on the pond removal rate. For maximum efficiency and avoidance of anaerobism primary facultative pond should be designed using the relationship (Arthur, 1981).

$$\lambda_i = 22T + 65 \qquad 3\text{-}2\text{-}9$$

λ_i is the aerial organic loading (kg/ha-day) and T is the ambient temperature (^{o}C). The value of T in equation 3-2-9 corresponds to an average temperature of 24.6 ^{o}C. Considerable higher loading rates are possible than are currently being used in the tropical region as a result of favourable climatic conditions especially temperature. From equation 3-2-10

$$K_{Tt} = K_{To}\ \theta^{(T-To)}/(1 + \phi t) \qquad 3\text{-}2\text{-}10$$

Arthur presented the following relationships for K_{Tt} and effluent

BOD_5, L_e as a function of temperature using 20°C as a reference temperature (Arthur, 1981).

$$K_{Tt} = \frac{0.92(1.077)^{T-20}}{1 + 0.23t^*/_3} \qquad 3.2.11$$

$$L_e = Lo/(1 + K_{Tt}t^*) \qquad 3.2.12$$

where: Lo = the influent BOD (mg/L)

Le = the effluent BOD (mg/L)

K_{To} and K_{Tt} = the first order removal rates at temperature T and time zero and at temperature T and time t respectively.

$\emptyset$ = retardation coefficient.

t^* = the detention time (day)

T = temperature (°C)

Both McGarry and Pescod, and Mara presented mathematical models to illustrate the influence of temperature on facultative ponds. McGarry and Pescod showed that

$$\lambda_r = 0.725\ \lambda_i + 10.34 \qquad 3-2-13$$

This equation describes the relationship between possible areal BOD loading and ambient monthly mean temperature in the range of about 0 to 30°C and λ_i is given as (McGarry & Pescod, 1970):

$$\lambda_i = 11.2\ (1.054)^T \qquad 3-2-14$$

where: λ_r = areal BOD removal (kg/ha-day)

λ_i = areal BOD loading (kg/ha-day)

T = ambient mean monthly temperature (°F)

Mara suggested the use of a safety factor in equation (3-2-14)

for design as

$$\lambda_i = 20T - 120 \quad \text{(Mara, 1978)} \qquad 3\text{-}2\text{-}15$$

The McGarry and Pescod method and Mara's design equation are most favoured for warm climates except for the set-back of involving just only two parameters.

The most recent study on the influence of temperature is that of Kilani (Kilani, 1985). The following linear regression equations were presented:

$$\lambda_i = 40.65 + 1.4\lambda_r - 1.59T \qquad 3\text{-}2\text{-}16$$

and $$\lambda_i = 31.01 + 1.06\lambda_r - 1.05T \qquad 3\text{-}2\text{-}17$$

for the unfiltered and filtered effluent COD respectively. Both indicate a gradual increase in organic removal rate with increasing organic loading at a given operating temperature. They both also show that the organic loading rate resulting in a given removal rate essentially remained the same for the temperature range of 25°C to 35°C. Kilani's conclusions that a combination of high temperature and high detention time results in high efficiency and that temperature influences organic removal more significantly than detention time probably need further investigation especially in terms of pH and suspended solids (SS) removal.

3.2.5 Radiation

Strictly, this is one of the factors that is beyond the control of man. Unfortunately most of the available data are from the laboratory where the factor (light) is simulated and thus con-

trollable (Hermann and Gloyna, 1958 and Thirumurthi, 1969 and 1974).

With sufficient nutrient such as nitrogen, phosphorus, CO_2 and a few other compounds and in the presence of light there would be algal growth with a net result of photosynthesis. All these requirements are no problem in most environment; even algae grow to some extent under ice cover but the most conducive environment is warm climate with sufficient light (Caldwell, 1946; Hermann and Gloyna, 1958). The light available at geographic locations in which there is no significant winter ice is more than adequate to support the required photosynthetic activity. The impact of radiation on ponds is on the production of the microorganisms especially algae that help in setting up the reaction necessary in the ponds.

A few designers have relied on radiation and tried to make maximum use of the concept by basing their design model on optimum algal production. This is always by maximizing the surface area or through the use of high rate stabilization pond (Marais, 1963 and Gloyna, 1976).

In North Dakota where ponds freeze over in winter, 4 ha per 1000 population is suggested or the pond capacity should be about 200 times the daily flow (Van Heuvele, and Svore, 1954). The BOD load per unit area theory which postulates that the BOD loading rate is governed by the average daily radiation originates from observation of the marked influence of solar radiation in

association with algae on re-oxygenation of the pond liquid. However, the BOD load per unit area method may be misleading as a criterion for the preservation of aerobic condition (Marais, 1966). In a homogeneous culture that is deep and dense enough to absorb practically all the light, the cells may cause refraction wiich may result in gradation of light below the surface. Furthermore, the light intensity, I in a homogeneous chlorella suspension obeys Beer-Lambert Law.

$$I = I_o \; e^{-kcd'} \qquad 3\text{-}2\text{-}18$$

where I_o is the light intensity subsequent to passage through a medium with absorption coefficient k, concentration c and depth d, I is the intensity after the passage (Hermann & Gloyna, 1958).

There is a critical point (compensation point) at which the light intensity just equals the saturation intensity, I_s. Also all the light that penetrates to greater depth is utilized with maximum efficiency. At lesser depth only a fraction of the original light intensity is utilized. The fraction, f of the light energy utilized in photosynthesis by a homogeneous suspension is given by equation (3-2-19).

$$f = \frac{I_s}{I} \left(\ln \frac{I_o}{I} + 1\right) \qquad 3\text{-}2\text{-}19$$

However, light-energy utilization in waste stabilization is complicated by several additional factors. For instance the algal and bacterial cultures in stabilization ponds are not homogeneous since numerous light scattering and absorbing

substances other than algae are present in the pond. Moreover, occlusion of light in varying degrees is caused by scum formation.

With the use of optical density (OD) crude estimates of the effective depth to which light of various intensities will penetrate a waste stabilization pond can be obtained, using Beer-Lambert Law as shown in equation 3-2-18.

Since

$$\text{Optical density (OD)} = \log \frac{I_o}{I} \qquad 3\text{-}2\text{-}20$$

then

$$OD = kcd.$$

With a photometer and light having a wave length of 410 mu, average values for OD were found to be about 0.10 (Hermann and Gloyna, 1958). From this, a value for kc can be obtained. Using $d' = 1.8cm$ from the sample tube of Hermann and Gloyna

$$kc = \frac{OD}{d'} = \frac{0.10}{1.8} = 0.056cm^{-1} \qquad 3\text{-}2\text{-}21$$

The depth of light in homogeneous culture media can now be obtained by

$$d = \frac{\log I_o/I}{kc} \qquad 3\text{-}2\text{-}22$$

When I is zero, the expression has no physical meaning. However, I_c is the light intensity of the compensation point at which photosynthesis is balanced by respiration. Unless one assumes that the system is homogeneous, then expression given by 3-2-22 needs to be modified for stabilization ponds.

3.2.6 Biological Factors

There are some controversies over the microorganisms that predominate in ponds. However, the predominant bacteria as assessed by morphological and biochemical study in stabilization ponds always consist of Gram positive, rod-shaped bacteria belonging to the general Achromobacter, Pseudomonas and Flavobacterum. (Caldwell, 1946; Oswald, 1953; McKinney, 1962 and Uhlmann, 1979). It had been pointed out by Fitzgerald that there is little correlation between microbial species which dominate in a pond and the geographical location of the pond, the degree of BOD loading, and the strength and type of sewage pretreatment (Fitzgerald, et al. 1958). However, Neel found that phytoplanktons in ponds in Missouri were usually green algae and euglenophytes. Euglena exhibited the greatest adaptability to loading variations. Chlamydomonas and various Chlorococcales were the other abundant types noted (Neele et. al, 1961). Here algae production increased with loading up to 67kg BOD/ha-day and then levelled off with greater loadings.

Changes in organic loading may influence the physical and microbial content of the stabilization pond. For example, a petroleum waste pond alternated from a pink colour (Chromatium present) to a green colour (Carteria present) when the loading of the pond changed from 67 kg BOD/ha-day to about 54kg BOD/ha-day (Cooper et al, 1965).

Loadings of about 644kg BOD/ha-day were responsible for the observed purple sulfur organism in a poultry manure waste (Cooper et al., 1965). Ponds in series were studied in California (Oswald, et al 1957). The primary cells 0.74ha and 2.44m deep were loaded at 70kg BOD/ha-day; the secondary cells of 0.63ha and 2.44m deep were loaded at 38kg BOD/ha-day and the third or tertiary cells 0.76ha and 1.22m deep were loaded at 24kg BOD/ha-day. The detention times were 48, 54 and 90 days respectively. Effluent from the tertiary pond had a BOD from 3 to 11mg/L, Ammonia Nitrogen of less than 1 mg/L and 0.2 to 2.3 x 10^3 coli organisms/100ml. Euglena were the predominant algal species in primary and secondary cells in concentrations of from 3 x 10^5/100mL in the secondary cell to 1.3 x 10^6/100ml/ in the tertiary cell. Micractinium was the predominant plankton in the primary cell with concentration of about 3 x 10^6/100mL.

With similar studies in India, blooms of blue green algae mainly Microcystis were found to dominate the pond flora (Jayangoudar, et al, 1965). Euglena had been detected first in heavily loaded ponds during summer operation (Parker, 1962). Euglena, Crustacea and Colosterium were the dominant population in the more lightly loaded lagoons.

16 species of algae were identified in the primary cell of a two cell pond in Nebraska (Donoyelles, 1967). In the secondary cell 28 species were identified. Ankistrodesmus falcatuus versus

acecularis was the most prevalent species of algae. The rotifier, Brachiorious was the prime grazer of algae. In another development it was found that filtrates of Chlorella inhibited coliform organisms while filtrates of Chlamydomonas were stimulatory (Moawad, 1968). In North Dakota Clostridium Botulinum type F was identified in a pond (Mentz, 1967). In another finding stabilization pond had been shown to completely reduce Mycobacterium tuberculosis populations in a waste from a sanitarium (Viraraghavan, and Raman 1967).

Where pond loadings exceed design limits, purple sulfur bacterial have been found; the occurence of which may be due to the presence of hydrogen sulfide, CO_2 and sometimes volatile acids (Holm and Vennes, 1970). Cooper et al identified Thiopedia rosea in a Californian pond treating rendering plant wastes and Chromatium sp in another treating petroleum wastes. In both cases lowering the BOD to a significant degree, was associated with these organisms (Cooper et al., 1965).

Some concentrations of nutrients were studied for algal growth in citrus wastes (Kott, 1968). 50mg/L nitrate and 20mg/L PO_4 were found to be optimum concentration for algal growth. Higher concentrations of these two inorganic inhibited algal growth. Chlamydomonas mundant were found in the heavily loaded aerobic lagoon and utilized acetate produced by anaerobic bacteria so the algae function only as a heterotroph since little oxygen was produced. In latter reports, ponds receiving liquid bovine manure show a change in flora from the predominantly

fermentative populations. Coliforms and Enterococci die-off was very rapid in these ponds (McCoy, 1967). Information on the survival or fate of enteric bacteria and viruses in stabilization pond as well as their concentration is limited. Most reports are on coliform reduction and die-off (Little, 1970, and Parhad & Rao, 1976). Many have concluded that enteric viruses removal in the ponds was by adsorption just as in activated sludge (Mack , 1962; Sanderson & Kelly, 1964; Bloom, 1969; Lund, 1970 and Sobsey et al., 1973).

Coliform reduction was found to be associated with BOD removal probably due to the coliform inability to compete for nutrients. Their activity is restricted mainly to the vicinity of the influent where 88 to 92 percent were found (Gann et al., 1968). Leman and Daphnia which could be successfully grown in lagoons were found to be capable of removing Chlorella or total coliform and Faecal Coli (Ehrlich, 1966). Unfortunately high pH is not always attributed to bacteria reductions (Caldwell, 1946). Actually it has been found that in the production of acid from carbohydrates by strains of Pseudomonas any acid produced was completely masked by alkali production from the protein (Levine and Bender, 1942). About 85 percent of stabilization ponds is often made of Pseudomonas-Achromobacter-Flavobacterium group, so the masking is probably due to the alkaline reactions. Organisms of this class are highly resistant to alkaline but very susceptible to acidic conditions. Their optimum pH for growth lies between 7.2 and 7.5 and below pH values of 5.5, they are killed. So this high pH effect due to algae may be an additional problem

as far as removal of this group of microorganisms is concerned.

Major variations in algae concentration also depend on period of the year (Gotaas, 1951 and Neel et al., 1956). Neel et al found that there seems to be a seasonal variations in algal dominance plus an unfailing appearance of Chlamydomonas after each anaerobic period. When most of the organic matter has been removed and high pH tends to precipitate mineral nutrients a mixed flora including Chlamydomonas succeed Chlorella because the organic compounds excreted by Chlamydomonas make the minerals available for algal growth.

Sometimes several factors influence each other during daylight hours for instance mixing equalizes temperature and concentration of oxygen and nutrients throughout the pond. These factors have great influence on the microorganisms inside the pond. The non-motile planktonic algae are transported cyclically into the photic zone which in heavily loaded pond rarely extends further than about 0.6m below the pond surface. The nonmotile ones below the themocline have no means of entering the photic zone and hence die off as a result of lack of photosynthesis. The diurnal rise in temperature causes a drop in the density and viscosity of the water (water viscosity at 30°C is about 30 percent less than at 15°C). This may promote nonmotile organism eventually to sink through the thermocline and die (Parhad and Rao, 1976). Both motile and nonmotile algae above the top layers of the stratification may be affected by excessive high radiation of light (Oswald, 1953). As light intensity becomes excessive, the rate

of chlorophill metabolism approaches its rate of synthesis, resulting not only in bleaching and yellowing but in reduced hydrogen supply and hence reduced photosynthesis such that ageing of the cells ensues. Carbohydrate and fat tend to increase with increased light intensity and nitrogen, protein and ash tend to decrease. Euglena grown with optimum light intensities and short retention period may have up to 50 percent crude protein (Oswald, 1953 and Meron 1963).

Variation in algae specie may also affect the amount of energy associated with oxygen liberated by the algae (Oswald, 1953 and Jayangoudar et al., 1970). The efficiency of light energy conversion by algae grown in waste water seldom exceed 10 or 12 percent (Gotaas et al., 1955 and Oswald, 1957). Gotaas et al. and Oswald were able to generate a relation for the quantity of oxygen that will be produced for a given efficiency. The oxygen donating capacity of algae may vary from genus to genus and from species to species because their chemical composition and their unit heat of combustion may differ. Culture containing more than one species are difficult to maintain in the laboratory. However, it must be realised that an heterogeneous system will be more efficient (Gloyna, 1971). While two or more algal species seem to coexist in equilibrium in natural water, the same cultures, when brought to the laboratory will invariably develop into a single culture system (Fogg, 1965 and Ju-Chang and Gloyna, 1968).

Among some species of green algae the overall photosynthetic or light energy conversion efficiencies may vary (Oswald, 1963). Oswald found that Chlamydomonas agloiformis possessed the highest overall photosynthetic efficiency, while Chlorella pyrenoidosa, Scenedesmus obliquus and Euglena gracilis exhibited lesser efficiencies. This photosynthetic efficiency of algae affects the loading considerably. One may be tempted to use other types of algae such as Oscillatoria due to their relationship to light conversion and growth efficiencies because it has been thought that small unicellular algae that are free floating relatively are more effective for photosynthetic oxygenation. However, Oscillatoria a fillamentous algae grows as well as produces approximately as high yields in organic wastes as do Chlorella or Scenedesmus.

Adequate population of some microorganisms especially algae is necessary. An oxygenation factor of 1.2 to 1.8 is considered essential for continuous maintenance of aerobic conditions because oxygen is lost to the atmosphere during periods of good light and saturation of oxygen. A factor above 1.8 as in shallow ponds due to algal bloom leads to high pH in the pond that will inhibit the oxidation of influent sewage by bacteria (Oswald, 1957).

The weight of algae theoretically estimated is normally less than that of algae actually grown because some CO_2 from the atmosphere dissolved in the liquid providing an additional source of carbon (Oswald, 1955 and Jayangoudar et al., 1970).

It has been estimated that an average of 634g/person/day of protein can be produced (Renn, 1954). Chlorella yields of 55g/L in the laboratory have been obtained using a small annulus as growth chamber and full sunlight (May and Stahh 1971). Yields of 16g/m^2/day and 14 to 21.2g/m^2/day have been reported (Fitzgerald, and Rohlich 1958).

Analysis of sewage and algae shows that carbon is the most apt limiting nutrient (Gotaas, 1954 and Allen, 1955). It was shown that Chlorella in sterile sewage required 24mg of carbon per liter per day to maintain constant growth rate, but the sewage has only 13.2mg of carbon. The deficit of 10.80mg of carbon/L/day was supplied from bubbling air. Other studies are on the yield of Euglena in sewage giving 0.18g/L/day of algae when air was the source of carbon and with added CO_2 the yield increased to 0.26g/L/day. Several concentrations of carbon were made for Euglena gracilis and it was found that the growth follows probability distribution curve when the data in Table 3.2.3 is plotted (Gotaas, 1954).

TABLE 3-2-2: ALGAL COMPARATIVE WEIGHTS (mg/L)
(GLOYNA, 1971)

	Theoretical Weight, mg/L	Actual Weight, mg/L
Monsoon Season	155.5	-
Post-Monsoon	91.1	169.3
Cold Weather	107.1	240.4
Summer Season	135.4	282.5

TABLE 3-2-3: CO_2 CONCENTRATION FOR ALGAL GROWTH

Concentration % CO_2	Growth
0.03	Fair Growth
0.5	Good Growth
2.0	Good Growth
4.0	Fair Growth
8.0	Poor Growth

Nevertheless, environmental conditions are important factors in the efficiencies. As for Nitrogen and Phosphorus, the latter has not been found to be a limiting factor to algal growth but the maximum algal concentration of say, 100 to 300mg/L is 10 times Nitrogen concentration (Gotaas, and Oswald 1955).

3.2.7 Depth

Some problems may be of design origin which are associated with depth of ponds. In Israel, deep ponds up to 5m in series with conventional ponds gave BOD removal of up to 80 percent with applied loadings of 100kg BOD/ha/day (Watson, 1961). There was stabilization during summer with algae present to a depth of 1m and temperature drops from 30°C at the surface to 20°C at 4m. DO at the surface layer disappeared and pH dropped from 8.8 to lesser values; nevertheless BOD removal efficiencies did not change appreciably. Probably at the greater depth on diurnal basis, the light penetrates only weakly to the lower layers of the pond and the algae cells grow more slowly during the day there, while at night thermal currents mix the various strata in the pond and a more

constant production of oxygen is thereby maintained.

A shallow lagoon of about 0.57m deep was studied at Hancock Air Force Base near Syracuse, N.Y. (Nemerow and Bryson, 1963). The primary cell of 0.7ha loaded at 146kg BOD/ha/day showed 69.3 percent BOD removal and the secondary cell of 3 hectare loaded at 12kg BOD/ha/day showed 88.3 percent BOD removal. In that study the difficulty of operating lagoons of shallow depths was highlighted. This was probably due to lack of compaction or sealing in cells resulting in the loss of liquid by percolation and evaporation to the extent that for seven of the twelve operating months there was no discharge. As a result of the operating conditions, menthane fermentation did not occur as expected inspite of the sludge build up. Growth of excess vegetation in and around the lagoon resulted with cattails, mash grass, shrubs and weeds inside the cells.

As a result of the unrestricted geographical location of area of application of stabilization ponds, there have been data from depths of ponds in Arctic and Sub-Arctic regions (Dawson and Grainge, 1969). The depth in summer were 1.22 to 2.19m while in winter they were 1.83 - 2.44m giving allowance for ice cover of up to 2.19m. This gave acceptable effluents. It is generally considered that 1.0 to 1.5m is an appropriate facultative pond depth range for tropical conditions (Fitzgeral et al., 1958; Watson, 1962 and McGarry and Pescod 1970).

3.2.8 Detention Time

Several attempts had been made to make use of detention time directly or indirectly in ponds design. Some investigators introduced the concept of ponds in series as against one single cell while some made use of baffles (Hermann and Gloyna, 1957; Marais and Shaw, 1961; Thirumurthi, 1967, 1969; Marais, 1970; Parhad et al., 1976; Kilani & Ogunrombi, 1984). As a result of the varying conditions under which they were operated it is difficult to make any correlation between detention time and areal organic load removal. However, in an ideal situation as the detention time increases, the efficiency of the reactor should increase (Thirumurthi, 1967, 1969 and Kilani, 1985).

Maximum efficiency in a series of pond system is achieved by making the detention times equal for all the ponds.(Mara, 1978). A study had reported that detention time in anaerobic pond should not be more than five days and that there is no effect on BOD removal filtered effluent with a variation in detention time of from 11 to 21 days (Parker 1959). Malina and Yousef reported a reduction of up to 99.4 percent of coliform number in 11.4 days detention time (Oswald and Gotaas 1963). Total coliform reductions were greater than total bacterial reduction. Comparisons of the total bacteria numbers with coliforms gave a correlation coefficient of 0.78. This is not all that strong. Reduction in coliform count proceeds in a regular manner with increasing detention time. With an appropriate design with regards to layout and detention period any desired level of coliform reduction can be achieved by

lagoons (Parker, 1962).

Coliform bacterial reduction rate constant, K in ponds had been found to follow Chick's Law which is a function of time according to (Meron 1963). In the study Meron obtained a rate constant K in the range of 0.070 to 0.127 giving an average of about 0.1/day. They concluded that longer detention time yield lower SS, coliform and algal numbers in the effluent. Evaporation and increased salinity results with increased detention.

In the study of two stabilization ponds in series with detention time of 20 days a 99.5 percent reduction of coliforms was recorded (Coetzee and Fourie, 1965). The reduction of E. Coli was 99.98 percent during the same period and with very high organic loading S. typhi was found to be more resistant to die off than E. Coli. From equations (3-2-10) and (3-2-12) by induction one can conclude that both first order removal rate constant, K and the effluent BOD removal, L_e are retarded with increase in detention time, t. Arthur introduced a retardation coefficient, ϕ (Arthur, 1981).

3.2.9 Shapes

Several shapes of stabilization ponds had been suggested by different reports. These shapes range from rectangular to circular ones. However, proper selection of the size and shape of a pond and the location of inlets would enhance natural mixing and load distribution (Pipes, 1961; Shindala and Murphy, 1969; Gloyna, 1971). Pipes is of the opinion that ponds should

be as nearly round or square as possible to aid in uniform mixing and load distribution. In another later report, it was suggested that ponds should be of regular shape with no coves or peninsulas to have thorough load distribution and prevent local septic areas (Porges, 1963).

Shape influences the accumulation of sludge in lagoon bottoms which also is a measure of the load distribution and mixing (Shindala & Murphy, 1969), They recommend that the shape of sewage ponds should be such that there are no narrow or elongated portions. They also recommended shapes like round, square or rectangular with a length to width ratio not exceeding 3:1. This almost agree with the ratio suggested by Mara (Mara, 1978). From the study with tracer test by Thirumurthi, it was observed that long and rectangular tank will be more efficient than square ponds, especially when sudden slugs of toxic industrial wastes are not released into the ponds (Thirumurthi, 1969). In addition to using rectangular facultative ponds, Kilani et. al introduced the idea of baffles (Kilani, 1984). They concluded that increase in the number of baffles would increase the efficiency of the pond.

Algae at the effluent has always been a problem. Mara then suggests that a deepening at the outlet end of a long narrow pond is advantageous in providing an anaerobic zone that would reduce this algae problem. This configuration seems more practical than that suggested by Middlebrook et al. (Middlebrook et al., 1978). Middlebrook et al. observed that because the

rotifiers and the microscopic crustaceans feed on algae, it is often thought that, if they are cultivated in the last cells of a series of waste stabilization ponds, they may help to reduce the algal content of the effluent.

3.2.10 Mixing

Of all the factors influencing pond behaviour, mixing is probably the most important. Mixing is induced principally by wind action although thermal mixing occurs at night to a degree. Lack of wind coupled with solar radiation normally leads to a state of stratification or non mixing in the pond. The intensity of mixing affects the kinetic rate of microbial degradation in the pond liquid , the growth of the algae, their species and concentration of algae. Distribution of the oxygen generated in the top layers to the bottom layers of the pond is principally through mixing. Good mixing always increases the capacity of the pond to take increased pollution load. Poor mixing or stratified condition always leads to the development of anaerobism in heavily loaded ponds.

Three flow models have been suggested to describe the hydraulic flow characteristics of the pond system. At one extreme the hydraulic flow pattern for the pond is assumed to be plug flow pattern. At the other extreme the pond is assumed to be a completely mixed reactor. In practice the hydraulic regime in ponds is neither a plug flow nor completely mixed flow but represents a dispersed flow pattern existing between the two extreme flow patterns. These various hydraulic

flow patterns are discussed in what follows.

3.2.1 (i) Plug Flow Reactor

This is an ideal reactor which can be either a simple tube or one packed with a catalyst or some other sort of packing. Sometimes it is referred to as a tubular, ideal tubular or piston flow reactor. A feed containing the reactants is fed continuously to the reactor inlet while the effluent containing the products and unreacted reactants is removed from the outlet. For an ideal plug flow it is assumed that there are no velocity or concentration gradients along the length of the reactor and no longitudinal (axial) diffusional mixing of either reactants or products along the reactor.

The concentrations of reactants and products (according to the assumption) vary with the axial distance, x only (Figure 3.2.1 (a)) Figure 3.2.1 (a) is Plug Flow Reactor (PFR). In the control volume (an infinitesimal volume), ΔV, the concentration is uniform. The mass balance of the reactant around the control volume from Figure 3.2.1 is:

$$n\big|_x - n\big|_{x+\ x} + rA_c\ \Delta x = \frac{\delta}{\delta t}(A_c\ \Delta xc) \qquad 3\text{-}2\text{-}23$$

$$\text{or, } -\frac{(n\big|_{x\ +\ \Delta_x} - n\big|_x)}{\Delta x} + rA_c = A_c\frac{\delta c}{\delta t} \qquad 3\text{-}2\text{-}24$$

where C is the molar concentration, x the distance from the reactor entrance, x the length of infinitesimal reactor volume, $n\big|_x$ and $n\big|_{x\ +\ \Delta_x}$ the molar flow rates of the reactant at the distances x and $x + \Delta x$ respectively and r rate of

degeneration of the reactant. In the limit as x tends to zero, the first term in 3 - 2 - 24 becomes the partial derivative of n and the equation reduces to

$$\frac{\delta n}{\delta x} + rA_c = A_c \frac{\delta c}{\delta t} \qquad 3\text{-}2\text{-}25$$

At steady state there is no concentration gradient with time within the reactor and taking the molar flow rate, n as the product of the flow rate F and molar concentration C, n = FC. Assuming this flow rate, F to be constant, then the last equation reduces to

$$-F \frac{dc}{dx} + rA_c = 0 \qquad 3\text{-}2\text{-}26$$

Since $A_c\ dx = dV$, then equation 3-2-26 when rearranged, gives

$$\int_0^V \frac{dv}{F} = \int_{c_e}^{ci} \frac{dc}{-r} \qquad 3\text{-}2\text{-}27$$

or

$$\tau = \int_{c_e}^{ci} \frac{dc}{-r} \qquad 3\text{-}2\text{-}28$$

where C_i and C_e are the influent and effluent reactant concentrations respectively. A plot of the concentration versus the inverse of the negative of degeneration rate will give τ as the area under the curve.

τ is the space time and this is the time required to process one reactor volume of feed measured at specified conditions. For a constant volume the space time is interchangeable with reaction time (Grady & Lim, 1980).

Since each packet receives no additional material such as BOD

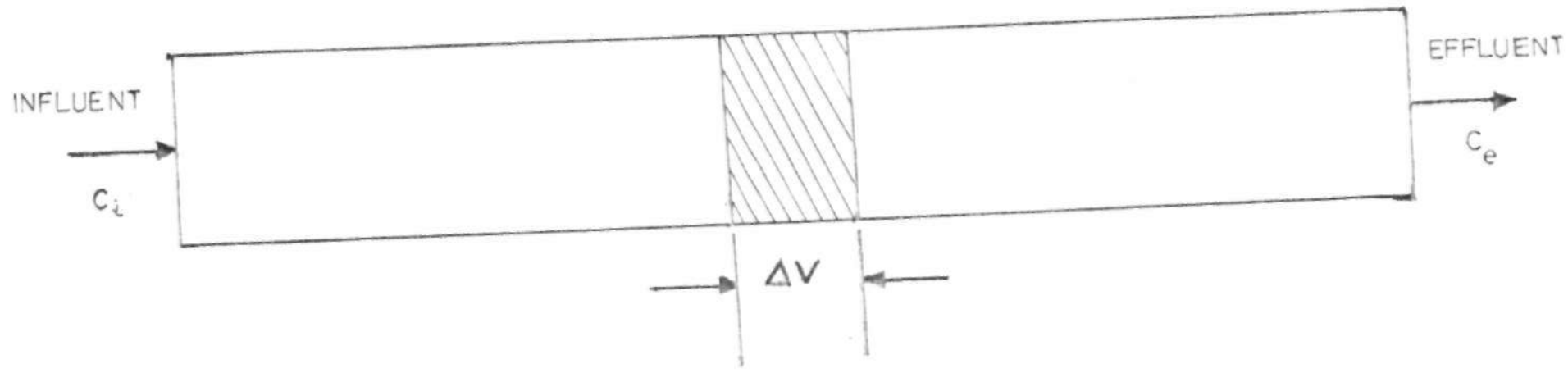


FIG 3-2-1 (a): FLOW REGIMES IN PONDS (PLUG FLOW)

and loses none to a neighbouring packet, the removal of BOD for instance within each packet is essentially a batch process so that the BOD removal in a plug flow reactor may be put as

$$C_e = C_i \; e^{-K\tau} = C_i \; e^{-kt} \qquad 3\text{-}2\text{-}29$$

where k is first order reaction rate (day^{-1}) and t the reaction (detention) time (day).

3.2.1 (ii) Completely Mixed Reactor

Completely mixed reactor is also known as Continuous Stirred Tank Reactors (CSTR), Continuous Flow Stirred Tank Reactor (CFSTR) or Backmix Reactor. It is usually equipped with baffles and a powerful mixer which is operated at sufficiently high speed so that the mixing is assumed to be perfect (that is homogeneous and instantaneous). Thus throughout the reactor there is no concentration gradient such that the entire fluid volume can be taken as the control volume. (Figure 3-2-1(b)).

The general mass balance equation on the system given in Figure 3-2-2 is

$$n_i - n_e + rV = 0 \qquad 3\text{-}2\text{-}30$$

where n_i and n_e are the molar flow rates for the input and output, respectively and V the reaction volume.

The CSTR equation is an algebraic equation and can be solved when the rate of consumption, r of reactant is known at only one concentration, that is, the effluent concentration. Hence the rate expression need not be known in analytical form.

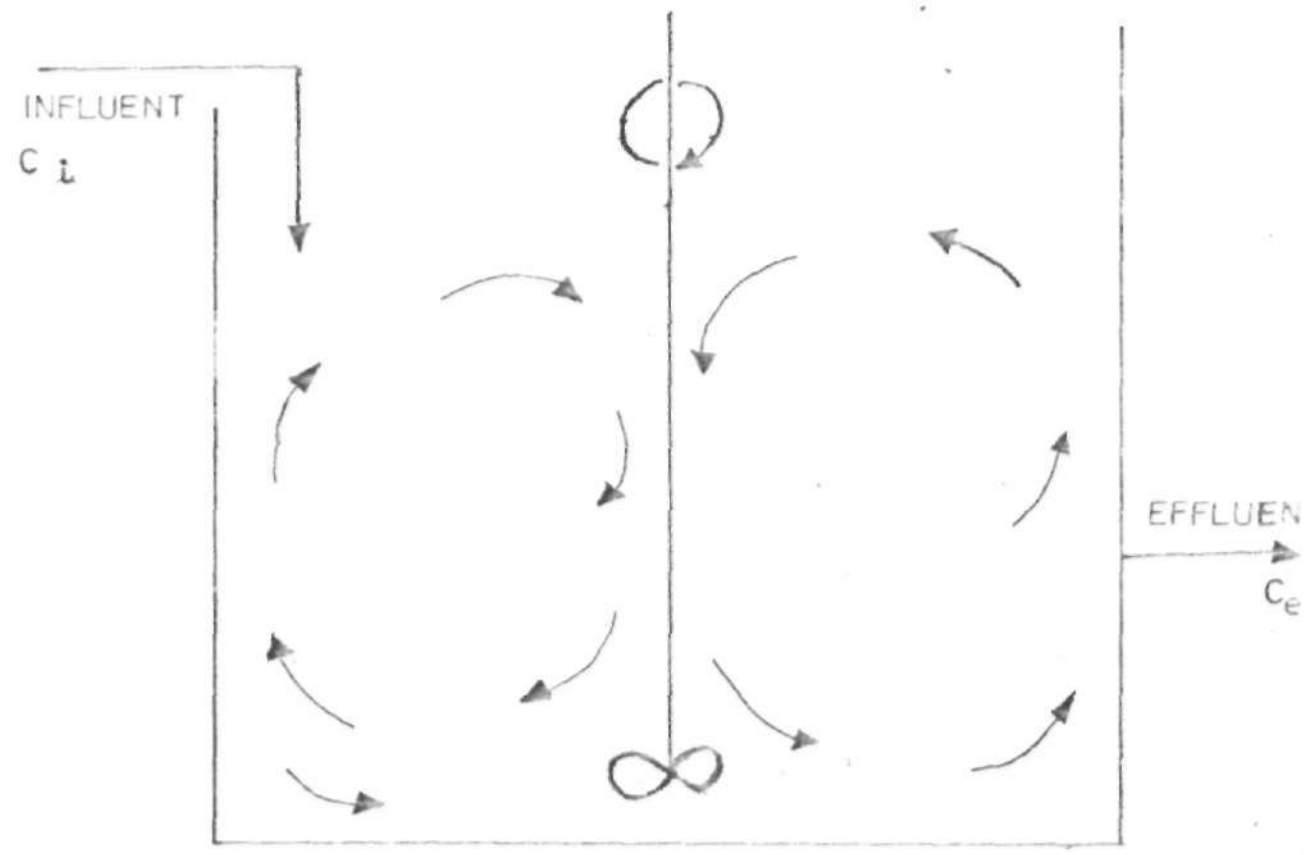


FIG 3-2-1(b): FLOW REGIMES IN POND

(COMPLETE MIXED FLOW).

For constant volume reactions, 3-2-30 can be put in the form of 3-2-26.

$$F_i C_i - FC + rV = 0 \qquad 3\text{-}2\text{-}31$$

where F_i is the molar flow rate of the influent with influent molar concentration, C_i and F and C are the molar flow rate and molar concentration respectively in the reactor and both are equivalent to the effluent values at steady state. With this steady state assumption where $F_o = F = F_e$ and $C = C_e$, 3-2-31 reduces to

$$\frac{V}{F} = (C_i - C_e)/-r = \tau \qquad 3\text{-}2\text{-}32$$

This time space, τ is the same as that defined under plug flow reactor and a plot of $\frac{-1}{r}$ versus the concentration C would give τ as the area under the curve.

Introducing the reaction rate, k and the reaction time, t as in 3-2-29, then 3-2-32 becomes, when F = Q, the mean flow rate

$$C_i Q = C_e Q + K_1 C_e V \qquad 3\text{-}2\text{-}33$$

Since $V/F = \frac{v}{Q} = \tau = t$, rearranging 3-2-33 and substituting t, then 3-2-33 will be of the form

$$\frac{C_e}{C_i} = \frac{1}{1 + k_1 t} \qquad 3\text{-}2\text{-}34$$

3.2.1 (iii) Dispersed Flow Reactor

In this model some degree of backmixing is superimposed upon

plug flow of fluid with a magnitude assumed to be independent of the position in the reactor and expressed by the longitudinal or axial dispersion coefficient, D. This coefficient is analogous to the coefficient of molecular diffusion in Fick's law of diffusion. (Figure 3-2-1(c)).

$$\frac{\delta}{\delta x}\ (uC - DA_c \frac{\delta c}{\delta x} = A_c\ \delta c/\delta t \qquad 3\text{-}2\text{-}35$$

or

$$D\frac{\delta^2 C}{\delta x^2} - \frac{u\delta c}{\delta x} - \frac{\delta c}{\delta t} = 0 \qquad 3\text{-}2\text{-}36$$

where:

A_c = Cross-sectional area available for flow.

In dimensionless form, putting:

$$Z = \frac{x}{L} \text{ and } \Theta = \frac{t}{L/(F/A)} = \frac{t}{\tau}$$

$$\frac{D}{uL}\ \frac{\delta^2 C}{\delta z^2} - \frac{\delta C}{\delta Z} - \frac{\delta c}{\delta \Theta} = 0 \qquad 3\text{-}2\text{-}37$$

where

D/uL = dispersion number, d and u = F/A.

When d = 0 there is no axial dispersion and therefore plug flow exists. And when it is infinitely large complete back-mixing exists and the reactor behaves as a CSTR. Furthermore with the boundary conditions:

$$C\ (Z.0) = 0 \qquad 3\text{-}2\text{-}38$$

$$C\ (0.\Theta) = C_o/u + (D/uL)\ C/d\ Z \qquad 3\text{-}2\text{-}39$$

and

$$dC/dz\ (1.\Theta) = 0 \qquad 3\text{-}2\text{-}40$$

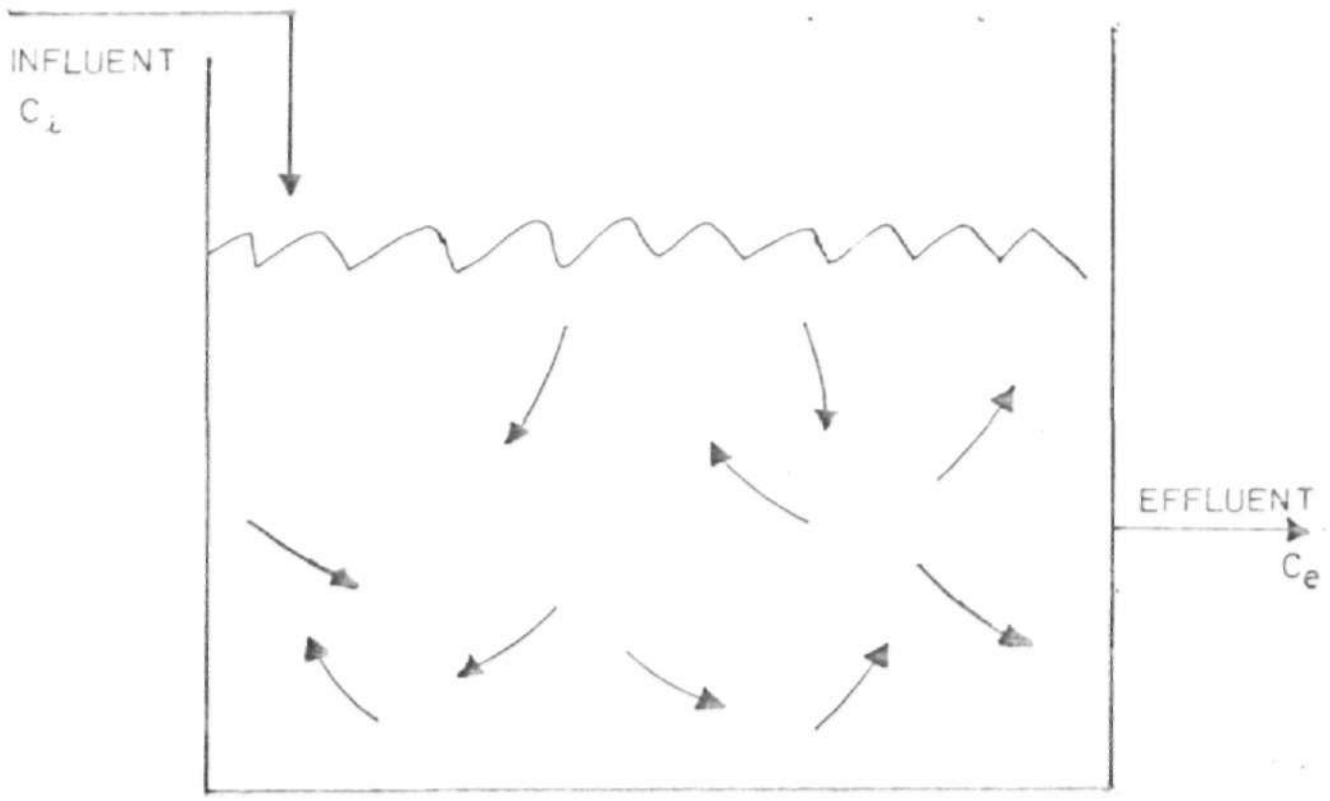


FIG 3-2-1 (c): FLOW REGIMES IN POND

(DISPERSED FLOW).

3-2-37 is difficult to solve analytically, so the solution must be sought numerically. For small deviation from plug flow (i.e when d is small) an approximate analytical solution is possible. And at the effluent, the tracer concentration is given by:

$$C/C_o = F(\theta) = \tfrac{1}{2}\ 1\text{-erf}\left(\frac{1-\theta}{2\theta^{0.5}\ (D/uL)^{0.5}}\right) \qquad 3\text{-}2\text{-}41$$

where erf is the error function which is defined as

$$\text{erf}(x) = \frac{2}{\pi 0.5} \int_o^x \exp(-y^2)\, dy \qquad 3\text{-}2\text{-}42$$

Solution of which can be obtained in most standard mathematical table.

From 3-2-41 the F(θ) function for small values of the dispersion number is characterized by the mean residence time ($\tau = L/(F/A)$) and the dispersion number.

For any kinetics, the steady state mass balance can also be put as:

$$\frac{D}{uL}\ \frac{d^2C}{dz^2} - \frac{dC}{dZ} + r = 0 \qquad 3\text{-}2\text{-}43$$

with the boundary conditions:

$$\frac{dC}{dZ} = 0, \quad \text{at} \quad Z = 1$$

and

$$C = D/uL\ (dC/dZ) = C_o \text{ at } Z = 0$$

Normally a numerical technique has to be used to solve 3-2-43 and its boundary conditions. However, for first order reactions $r = -kC$, it can be solved analytically to provide the solution at the exit as done by Wehmer and Wilhelm

(Wehmer and Wilhelm, 1959) (Appendix A).

$$\frac{C_e}{C_1} = \frac{4ae^{\frac{1}{2d}}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}} \qquad \ldots.3\text{-}2\text{-}44$$

where:

$a = (1 + 4ktd)^{\frac{1}{2}}$

$d = D/uL$ = dispersion number

D = Coefficient of dispersion, (L^2T^{-1})

u = mean fluid velocity, $(L\ T^{-1})$

L = mean path length, tank length, (L)

These equations allow calculations of the effluent concentration from a reactor with axial dispersion within which a first order reaction is occuring.

The Dispersion Index varies from zero for plug flow to infinity for completely mixed flow. Thus, the hydraulic properties of ponds can be compared by using the Dispersion Index of various ponds which is usually estimated by using tracer technique. The use of equation 3-2-44 for the design of pond is cumbersome and therefore the Thirumurthi chart (Figure 3-2-2) which was developed from it is found convenient to use (Thirumurthi, 1969).

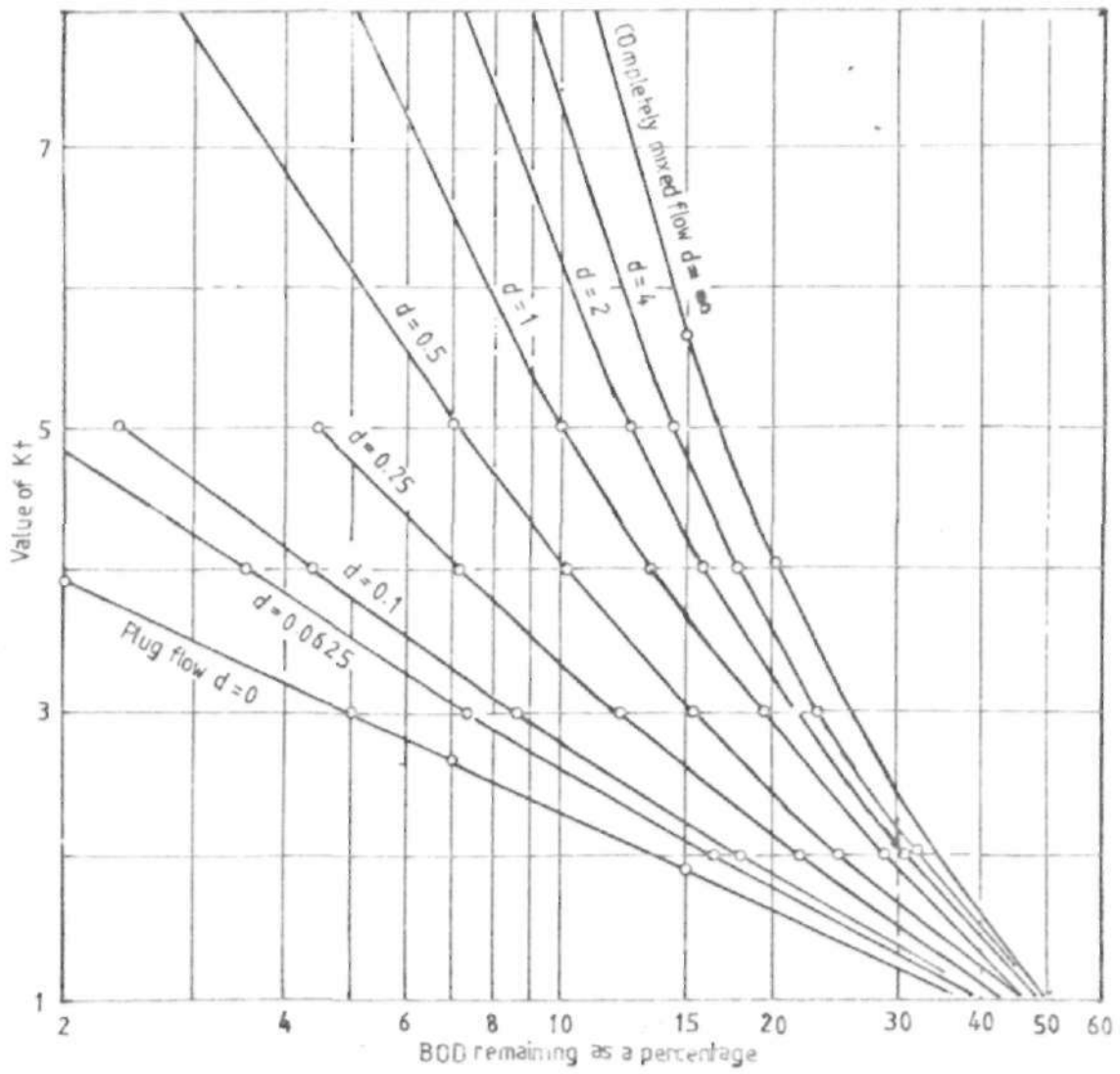


FIG 3-2-2 THIRUMURTHI CHART FOR WEHNER-WILHELM EQUATION.

For first order chemical reactions, the volume of a non ideal reactor is bigger than that of an ideal plug flow reactor having the same efficiency and can be related as in 3-2-45. (Levenspiel, 1962)

$$\frac{V}{V_p} = (1 + kt_g d) \quad \ldots \quad \ldots \quad 3\text{-}2\text{-}45$$

$$kt_g d > 0 \quad \ldots \quad \ldots \quad 3\text{-}2\text{-}46$$

where: V = volume of the non-ideal (dispersed flow) reactor

V_p = volume of ideal plug flow reactor

k = First order reaction constant

d = dispersion number index.

3.3.0 TRACER TEST TECHNIQUE

The hydraulic characteristics of a stabilization pond may be examined by injecting a tracer material into the inlet and observing its appearance in the effluents. The flow through curve defining the tracer concentration in the effluent as a function of time can be plotted as shown in Figure 3.3.1.

Methods for estimating the hydraulic parameters from the tracer test data are given in Appendix B.

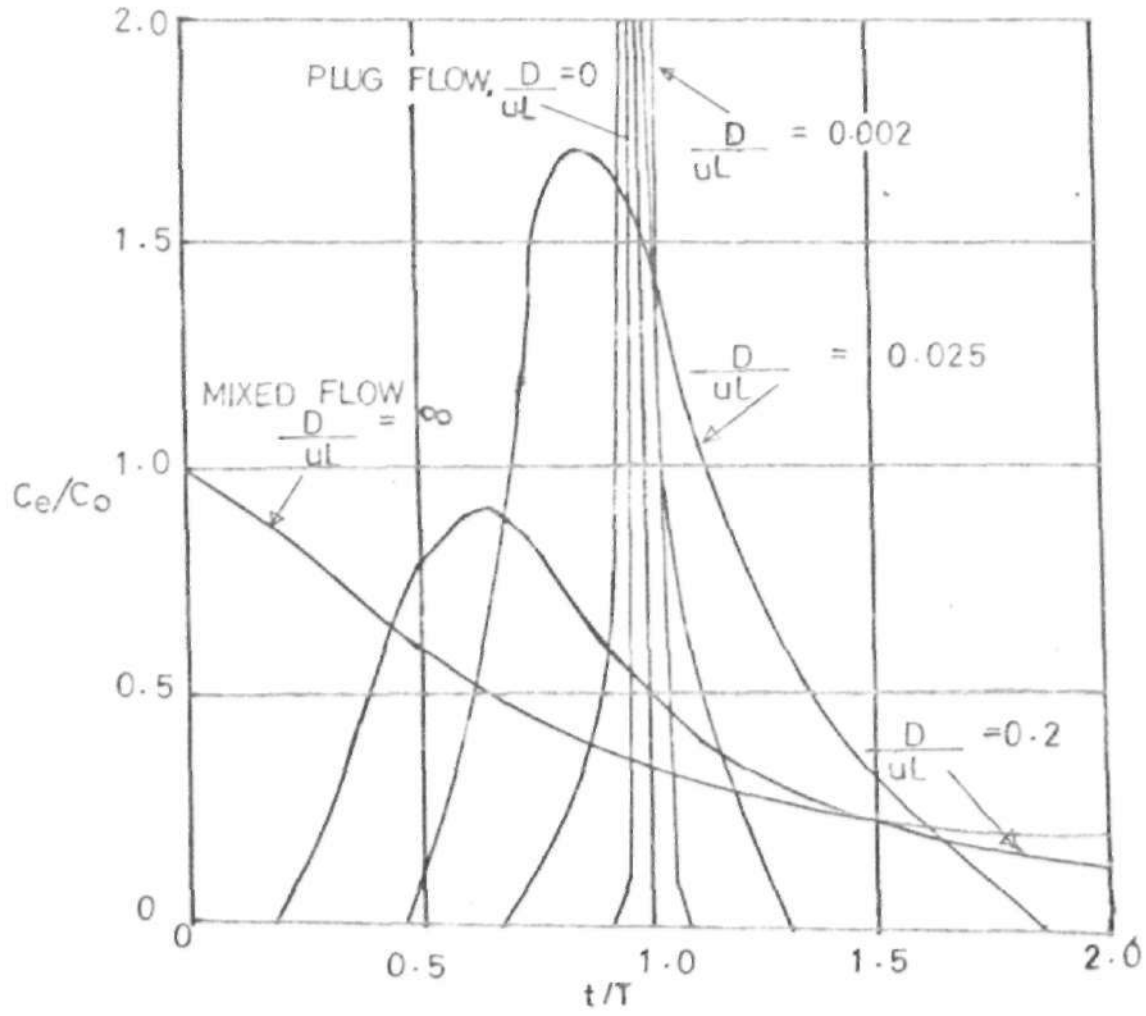


FIG 3-3-1 : THE FLOW -THROUGH CURVE (C - CURVE)

CHAPTER 4

4.0 STABILIZATION POND DESIGN

At present, the design formulae are based on either empirical or semi empirical (rational) methods. Unfortunately most of the few available data for stabilization pond apply only to temperate regions; modification is needed to make them applicable to the hot climate (Hermann and Gloyna, 1958; Marais and Shaw, 1961; Arceivala, 1973; Oswald, 1963; Porgess, 1963 and Thirumurthi, 1969).

Among the design considerations for stabilization pond are such factors as the pond geometry, size, numbers, base embankment and both inlet and outlet structures (Gloyna, 1971 and Mara, 1978). Length to breadth ratios of 3 to 1 or 2 to 1 have been recommended (Gloyna, 1976). For easy maintenance it is desirable to have at least two ponds in parallel. In order to guide against possible ground water pollution, the bottom of the pond should be impermeable or sealed with materials such as polythene sheeting, linings of clay, bitumen or asphalt especially when seepage exceeds 10 percent (Marais, 1961; Gloyna, 1976 and Mara, 1978).

An embankment of 3 to 1 is usually suitable in most soil conditions and both erosion due to surface wave action and vegetation growth can be prevented by placing slabs on the embankment (Gloyna, 1976). Operational performance needs to be monitored occasionally hence the inflow and outflow are measured with venturi flume and V-notch respectively.

4.1.0 Facultative Ponds

Various design methods based on empirical formulae, statistical analysis and rule of thumb are available for facultative ponds. The empirical method includes:

(a) Global Environmental Design Basis or the Surface Loading Procedure

(b) Gloyna's Empirical Formula

(c) India Approximate Procedure

(d) Solar Radiation Method

(e) McGarry and Pescod Formulae

(f) Marais and Shaw Procedure.

(a) The Global Environmental Design Basis

Table of acceptable organic loadings in facultative ponds (Table 4.1.1) successfully used in various environments is given by Gloyna (Gloyna, 1971). The use of this table assumes that (i) the total evaporative and seepage loss is not greater than the influent volume (ii) the per capita BOD contribution is 50g BOD_5 per day and (iii) the wastewater generation per capita day is 100 litres. According to Gloyna, caution is needed when using this procedure for design purpose and recommends that the values should be used only where limited information precludes the use of more refined methods.

(b) Gloyna's Empirical Formula

This is an empirical formula based on the operational results from many small laboratory experiments, larger pilot scale studies and several full scale ponds in which the pond volume is related to temperature, influent BOD and wastewater flow

TABLE 4-1-1: GENERALIZED BOD LOADING PER UNIT AREA PER DAY UNDER VARIOUS CLIMATIC CONDITIONS (GLOYNA, 1971).

SURFACE LOADING (kg BOD/ha) PER DAY	POPULATION (PER ha)	DETENTION TIME (DAYS)	ENVIRONMENTAL CONDITIONS
Less than 10	Less than 200	More than 200	Frigid zones, with seasonal ice cover, uniformly low water temperatures and variable cloud cover.
10 - 50	200 - 1000	200 - 100	Cold seasonal climate, with seasonal ice cover and temperature summer temperatures for short season.
50 - 150	1000 - 3000	100 - 33	Temperate to semi-tropical, occasional ice cover, no prolonged cloud cover.
50 - 350	3000 - 7000	33 - 17	Tropical uniformly distributed sunshine and temperature, and no seasonal cloud cover.

(Gloyna, 1971). The expression is given as

$$V = (3.5 \times 10^{-5}) N_q L_a \theta^{(35-Tm)} \qquad 4\text{-}1\text{-}1$$

where:

V = pond volume (m^3)

N = contributing population

q = per capita wastewater flow (L/day)

θ = temperature coefficient = 1.085

T_m = average water temperature in the coldest month ($^{\circ}C$)

L_a = ultimate Biochemical Oxygen Demand (mg/L).

The formula has been updated to include factors such as toxicity and sulphide contents (algal toxicity factor) as in 4-1-2 (Gloyna, 1976).

$$V = (3.5 \times 10^{-5}) \alpha\beta N_q L_a \theta^{(35-T_m)} \qquad 4\text{-}1\text{-}2$$

where α and β are coefficients representing toxicity and sulphide contents in the ponds recpectively.

The Gloyna's Equations 4-1-1 and 4-1-2 major set back is that they are based entirely on data collected from ponds operating in temperate zones. The reference temperature of $35^{\circ}C$ is considered too conservative and an extrapolation instead of more reliable interpolation is required for field and out of the temperate zones application.

A further modification suggested by Middlebrooks may not equally be applicable to hot climates due to the assumption that the maximum visible solar radiation prevailing at pond

locations is 250 langleys thereby restricting its application to areas where light is limiting (Middlebrooks 1982). Middlebrooks modification is given as

$$\frac{V}{q} = 3.5 \times 10^{-2} C_i (1.099)^{SE/250 (35-T)} \qquad 4\text{-}1\text{-}3$$

where C_i is the influent BOD or COD (mg/L) and S_E the visible solar radiation at elevation E (Langley).

(c) Indian Approximation Procedure

This procedure relates the areal BOD loading to the latitude. An increase of 24kg BOD/ha-day for every 40 degree change in latitude produces a range of loading rate from 150 to 325kg BOD/ha-day.

Extrapolation outside India are not recommended. The values are just approximate and are based on an altitude of sea level, clear skies for 75 percent of the year and a pond depth of 0.9 to 1.5 metres. The influence of latitude on the pond performance has not yet been fully assessed world wide.

(d) The Solar Radiation Method

This method was developed by Oswald and Gotaas based on the following factors (Oswald and Gotaas, 1957):

(i) *average monthly maximum and minimum solar radiation*, S_o.

S_o = minimum visible radiation + (max - min)λ 4-1-3

where:

λ = the sky clearance factor

$$\lambda = \frac{\text{actual hours of sunlight}}{\text{total possible sunlight hours}}$$

(ii) average monthly sky clearance factors

(iii) algae energy requirement and utilization efficiency, and

(iv) oxygen production rate by algae.

As a result of the influence of altitude, the visible radiation values need to be corrected, such that

$$S_E = S_o (1 + 0.00925E) \qquad 4\text{-}1\text{-}4$$

where, S_o and S_E are the visible radiation at sea level and elevation, E respectively (g Cal/cm^2 day), and E the elevation above sea level.

According to Oswald and Gotaas, the mass of algae produced is given as

$$\text{Mass of algae} = \frac{0.06 \times S_E}{6 \times 10^6 \times 10^{-8}} \qquad 4\text{-}1\text{-}5$$

The oxygen production by algae is estimated between 1.26 to 1.75g/g of algae produced with an average of 1.50g/g of algae. Therefore introducing this average value into 4-1-5 the rate of oxygen production, O_p is given as

$$O_p = 1.5\ S_E \qquad 4\text{-}1\text{-}6$$

In warm climates O_p as high as 500 kg/ha-day have been obtained, however, 200 kg/ha-day is the average realistic figure especially for design.

Different design approaches relating the oxygen produced by the photosynthetic activity of algae to either BOD_5 or ultimate BOD removed or applied to pond are also suggested (Siddiqi & Handa, 1971).

$$\text{Ultimate BOD load} = L_f \times O_p \qquad 4\text{-}1\text{-}7$$

where

L_f is the load factor

Oswald and Gotaas suggest a lower limit of 0.44 for L_f and upper limit of 8.0 to maintain facultative conditions.

Arceivala et al. proposed that the ultimate BOD load to the pond should not exceed the oxygen produced by algae, and although this ignores the reduction of BOD by sedimentation and anaerobic decomposition, it provides a safe design basis (Arceivala, et al 1970). Mara suggests a safety factor not greater than 1.07 of the oxygen produced to give the design load for ultimate BOD (Mara, 1975). This factor is included to compensate for under utilization of the oxygen produced for organic stabilization. However, no account is taken of BOD reduction by sedimentation. Any theoretical relationship between load and algae oxygen should include natural surface reaeration as an oxygen source in addition to the sedimentation phenomena.

(e) McGarry and Pescod Formula

McGarry and Pescod carried out a survey of ponds operating in tropical and temperate zones and indicated that the areal BOD removal, λ_r in primary facultative ponds may be estimated through the knowledge of areal BOD loading, λ_i as given by equation 3-2-13.

The maximum feasible loading of the ponds is again related to the mean monthly ambient temperature, as in equation 4-1-8.

$$\lambda_i = 11.2\ (1.054)^{T_m} \qquad 4\text{-}1\text{-}8$$

where:

T_m = the mean monthly ambient temperature (^oF)

λ_i = maximum areal BOD loading.

For safety purposes a safety factor of 1.5 is proposed by Mara (Mara, 1978). The results of the modified version of the method were compared with other results from other methods and it was noticed that the comparison is only valid for equal temperature.

An alternative design formula for λ_i, from regression analysis is given in the form of equation 3-2-15

$$\lambda_i = 20\ T_m - 120 \qquad 4\text{-}1\text{-}9$$

and from Mara's suggestion, a non linear equation relating areal loading, λ_i and maximum areal loading, $\hat{\lambda}_i$

$$\hat{\lambda}_i = 1.5\ \lambda_i \qquad 4\text{-}1\text{-}10$$

And the areal BOD loading is related to the surface area as

$$\lambda_i = 10 C_i Q A^{-1} \qquad 4\text{-}1\text{-}11$$

where C_i is the BOD loading (mg/L), Q the flow rate (L/day) and A surface area (hectare).

The design surface area is obtained by substituting equation 4-1-8 into equation 4-1-11.

Then,

$$A = C_i\ Q/(0.75(1.054)^{T_m}) \qquad 4\text{-}1\text{-}12$$

or from 4-1-9

$$A = C_i\ Q/(2_{T_m} - 12) \qquad 4\text{-}1\text{-}13$$

As far as temperature effect is concerned both McGarry and Pescod, and Mara's design equations are still acceptable for hot climate. However, any other factors that influence pond performance except temperature and the areal loading are beyond the scope of their approaches.

(f) Marais and Shaw Procedure

Marais and Shaw derive the fundamental differential equation governing the concentration of BOD and faecal bacteria in ponds, give various solutions to the equation and establish the relationship between the kinetic activity of a river and ponds in series (Marais and Shaw, 1961).

Experimental evidence is presented to verify and indicate the limitations of the theory within which framework a criterion is developed to determine the maximum loading for aerobism and anaerobism.

The formulated theory is applied to the development of a design procedure for a series of ponds with a little consideration given to the kinetics of recirculation. And among the assumptions made are that the reduction in concentration takes place according to monolmolecular law and that the mixing of the influent with the pond content is instantaneous and complete.

Design procedure of Marais and Shaw consist of

(i) Estimation of the flow, the BOD and faecal bacterial concentrations in the influent to the primary pond

$$C = 1.2 \frac{b}{q} \; 10^{10} \qquad 4\text{-}1\text{-}14$$

where:

b = the BOD contribution per person per day (mg)

q = effluent flow per person per day (L) and

C = the total BOD concentration in the pond (mg/L).

(ii) Select the pond depth, d and from the depth determine the maximum total BOD in the pond

$$C = \frac{600}{2d+8} \qquad 4\text{-}1\text{-}15$$

(iii) Determine the detention time, t of the first pond from

$$C = \frac{C_i}{0.17t + 1} \qquad 4\text{-}1\text{-}16$$

or

$$t = \left(\frac{C_i}{C} - 1\right)\frac{1}{0.17} \qquad 4\text{-}1\text{-}17$$

where C_1 is the influent BOD (mg/L).

(iv) Determine the surface area of the pond from the relationship.

$$A = \frac{Qt}{46.8d} \qquad 4\text{-}1\text{-}18$$

where A is the area in m^2 and Q daily flow rate in litres.

(v) The second and third pond depths are selected with suitable surface areas to give 7 days detention time each.

The major set back with this method is that the influence of depth has great impact on it. It is suggested that the area/depth ratio should not fall below one thousand (Marais and Shaw, 1961). Some of the assumptions such as complete and instantaneous mixing may not be totally valid.

(g) Other popular group of design formulae, the rational methods is made up of (i) the first order kinetics and (ii) the Thirumurthi method.

(i) The first order kinetics.

This method is based on the basic principle of completely mixed reactors in which BOD_5 removal follows first order reaction and is given as equation 4-1-19 (Marais, 1966 and Gloyna, 1976).

$$\frac{C_e}{C_i} = \frac{1}{V/Q} \int_{t=o}^{\infty} e^{-k_T t} \; e^{-(Q/V)t} \; dt \qquad 4\text{-}1\text{-}19$$

where C_i and C_e are the influent and effluent BOD respectively (mg/L), K the reaction rate (1st order) and temperature dependent in accordance with the relationship

$$K_T = K_{35} \times 1.085^{(T-35)},$$

and K_{35} has been found to be 1.2 in South Africa

t = detention time (days)

V = volume of pond (m^3), and

Q = the flow rate (L/day).

Integrating 4-1-19 and solving in terms of a detention factor, the following expression is obtained

$$\frac{C_e}{C_i} = \frac{1}{K_T t + 1} \qquad 4\text{-}1\text{-}20$$

For a batch reactor, the BOD is degraded according to 4-1-21

$$C_e = C_i \; e^{-k_T t} \qquad 4\text{-}1\text{-}21$$

And for a fixed percentage reduction it can be shown that for single ponds, the ratios of the degradation constants are equal to the ratios of the detention times thus

$$\frac{K_T}{K_{35}} = \frac{t_T}{t_{35}} = \theta^{(35-T)} \qquad 4\text{-}1\text{-}22$$

where:

θ = temperature coefficient ($\theta = 1.085$).

This method is based on the assumption of a completely mixed reactor, whereas most pond systems are far from this condition. Hence the method tends to overestimate the reactor removal efficiency although a judicious choice of the value of K_T can compensate for this especially when based on local operative experience in other pond systems.

(ii) Thirumurthi Method

This is another rational method based on non ideally mixed reactor. An important parameter in this system is mixing and the degree of mixing is evaluated by the value of d, the dispersion number which seldom exceeds 1.0.

As a result of the importance of hydraulic patterns in ponds which are influenced by factors like the temperature, shape rate constant and loadings, a tracer test is always carried out to evaluate the mixing pattern (Thirumurthi & Mashashibi, 1967; Thirumurthi, 1969).

Thirumurthi comes up with a design chart. The product of the

rate constant and detention time is plotted against percentage BOD remaining for different values of dispersion number, d (Figure 3-2-2).

The resulting equation for this model is as given by equation (3-2-44). The approximated value is given by

$$\frac{C_e}{C_i} = \frac{4ae^{(1-a)/2d}}{(1+a)^2} \qquad 4\text{-}1\text{-}23$$

Traditionally the evaluation of the first order reaction rate, K values has always been limited to the influence of temperature as can be observed in equation (4-1-22). However, Thirumurthi pointed out that other factors should be taken into consideration. He suggested that a standard value of the first-order BOD removal coefficient should be evaluated for a set of assumed standard environmental conditions and that corrections should be made to the value whenever the field conditions deviate from the assumed standard environment. The standard first order BOD removal rate constant, K may be corrected for temperature, organic loading and toxicity as follows (Kilani, 1985):

$$K_1 = K_2 C_{Te} C_o C_{Tox} \qquad 4\text{-}1\text{-}24$$

where:

K_1 = design first order BOD removal coefficient

K_2 = first order BOD removal coefficient corresponding to the actual pond's environmental conditions

C_{Te} = correction factor for temperature

C_{Tox} = correction factor for industrial toxic chemicals

C_o = correction factor for organic loading

C_{Te} can be estimated from equation (4-1-25)

$$C_{Te} = \theta^{(T_m - 20)} \qquad 4\text{-}1\text{-}25)$$

Equation (4-1-25) is similar to equation (4-1-22) except that Thirumurthi obtained a value of 1.036 for θ (Thirumurthi, 1974). Thirumurthi had earlier developed a design equation correlating organic load, λ_i with BOD removal coefficient, K_i (Thirumurthi, 1969). The developed correlation for plug flow condition was:

$$\log_{10} \frac{\lambda_{i2}}{\lambda_{i1}} = 12\{(K_1)_2 - (K_1)_1\} \qquad 4\text{-}1\text{-}26$$

where subscripts 1 and 2 denote any two corresponding values. For a standard environment, λ_s and K_s can be substituted for λ_{i2} and $(K_1)_2$ respectively. Hence equation (4-1-26) becomes

$$\log_{10} \frac{\lambda_s}{\lambda_{i1}} = 12\{K_s - (K_1)_1\} \qquad 4\text{-}1\text{-}27$$

If all other environmental conditions are kept constant, the corresponding factors for organic load C_o may be related to K_1 values as

$$C_o = \frac{K_1}{K_s} \qquad 4\text{-}1\text{-}28$$

substituting equation (4-1-28) into (4-1-27) and simplifying gives:

$$C_o = 1 - \frac{0.083}{K_s}\left\{\log_{10} \frac{\lambda_s}{\lambda_{i1}}\right\} \qquad 4\text{-}1\text{-}29$$

A value of λ_s = 67.2 kg/ha/day had been suggested by Thirumurthi.

The error in equation 4-1-23 is negligible for small values of d and only becomes significant when d exceeds 2.0.

Among the limitations of this model are:

(i) the design criteria will suffer from the usual limitations related to all model based research and some inevitable approximations;

(ii) the effects of percolation, rainfall and wind agitation are difficult to simulate and judge; a correlation factor needs to be introduced at times to have a conservative criteria;

(iii) the data and the resulting criteria are more meaningful only at steady state condition; and

(iv) accurate evaluations of some parameters such as the reaction rate K and the dispersion number, d are not easy to obtain.

4.1.1 Anaerobic Pond

Formulation of anaerobic pond design is mainly empirical although with a reasonable theoretical basis. A design formulation is given as

$$C_e = \frac{C_i}{K_1 \left(\frac{C_e}{C_i} \right)^n t + 1} \qquad 4\text{-}1\text{-}30$$

where C_i and C_e are the influent and effluent BOD respectively, t detention time for completely mixed system (days), K_1 BOD removal constant, base e (per day) and n an exponent.

The values of K_1 and n had been given as 6 and 4.8 at pond liquid temperature of about 25°C respectively for Zambian conditions (Gloyna, 1971). According to Gloyna the following design considerations are necessary for successful anaerobic pond operation at 2 to 5 day detention time, 3 to 5m depth, surface and volumetric loadings of BOD in the range of 1000 - 6000 kg BOD/ha-day and 0.19 - 0.24 kg BOD/m^3-day respectively.

In a later development it was pointed out by Gloyna that (i) the inorganic load must be based on ultiamte BOD values as such 4-1-30 becomes

$$C_e = \frac{C_1}{K_1 \left(\frac{C_e}{C_1}\right)^n t + 1} \qquad 4\text{-}1\text{-}31$$

where C_i' is influent ultimate BOD;
(ii) a fraction of the influent BOD remains in the liquid substrate of the pond while the rest settles to the bottom as sludge;(iii) mixing occurs and effluent BOD is equal to the pond BOD (completely mixed); (iv) there are no losses of liquid in the system; (v) reaction constants for the liquid substrate and the sludge layer are temperature dependent; and (vi) the fraction of the BOD lost in the sludge by fermentation either returns to the pond substrate or leaves the pond as gas (Gloyna, 1976).

Temperature of the pond contents is very critical. It has been found that ponds which continue to operate satisfactorily during winter months usually have a temperature above 30°C with an

excellent cover of grease and inert solids. If the temperature drops below 10 or 15°C gas production usually drops to minimum. At this point, the pond simply behaves just as a settling basin. The pond functions better when deep, 3 to 5m than when shallow. However, consideration should be given to the aquifer to avoid groundwater pollution. Also the temperature of such groundwater may lower the temperature of the sludge at the bottom. Since this is where a major portion of the gasification occurs, if the temperature drops below 10 to 15°C, the efficiency of solids and BOD reduction in the pond would also be reduced. Wind action has much less effect on the pond contents and the scum layer. (McKinney, 1982).

The effluent, from anaerobic pond usually consists of significant nutrients as a result of high organic loading. The effluent therefore needs additional treatment before discharge (White, 1970; Gloyna, 1976, and Metcalf and Eddy, 1979).

4.1.2 Maturation Pond

Maturation ponds are designed to provide for secondary effluent polishing and seasonal nitrification (Metcalf and Eddy, 1979). Operationally the residual biological solid are endogeneously respired and ammonia is converted to nitrate using the oxygen from surface reaeration and from algae. A detention time of 18 to 20 days is suggested as the minimum period to provide complete endogeneous respiration. This detention time seems to be the principal factor in maturation pond design.

The ponds are always arranged in series depending upon the degree of efficiency required in the reduction of faecal bacteria. The governing equation for maturation pond design is for bacterial removal

$$N_e = \frac{N_i}{(K_T t_1 + 1)(K_T t_2 + 1)\ldots(K_T t_n + 1)} \qquad 4\text{-}1\text{-}32$$

or

$$\frac{N_e}{N_i} = \frac{1}{\prod_{o}^{n} (K_T t_n + 1)} \qquad 4\text{-}1\text{-}33$$

where N_i and N_e = influent and effluent faecal coliform count per 100ml respectively.

K_T = faecal coliform removal rate constant, at a reference temperature, T^oC (day^{-1}).

K_T = $2.6 \times 1.19^{(T-20)}$, for temperature, T between 5 and 21^oC,

t = detention time (days) (Metcalf and Eddy, 1979).

n = number of ponds in series.

CHAPTER 5

5.0 EXPERIMENTAL INVESTIGATIONS

5.1.0 Material

Five rectangular galvanized steel tanks were used as laboratory model ponds, each having a working capacity of 125 liters. The layouts of the ponds are as shown in Figures 5-4-1 to 5-4-5. The ponds differed only in the number of baffles. Pond A had no baffle and served as the control model pond B had 6 baffles, pond C had 9 baffles, pond D had 12 baffles while pond E had 15 baffles. These are round-the-end baffle configurations which force the waste water to follow a snake-like path in the horizontal plane through the ponds. Three 40 watts flourescent lamps located approximately 100cm over the ponds simulated the solar energy source. These gave a light intensity of about 1,700 lumen/m^2. A 12 hour-on and 12 hour-off light cycle was maintained throughout the period of the investigations. The synthetic milk waste used for treatment performance study was Fussels condensed milk. The chemical composition of Fussel condensed milk is as given in Table 5-1-1. The amount of milk, W_m required to give the desired BOD of each feed is obtained from Figure 5-2-2 which was initially prepared.

5.2 Method

Two types of tests were conducted. The first test was the tracer test, the purpose of which was to evaluate the hydraulic properties of the ponds. The second test was conducted to evaluate the organic removal efficiency of treating a synthetic milk waste.

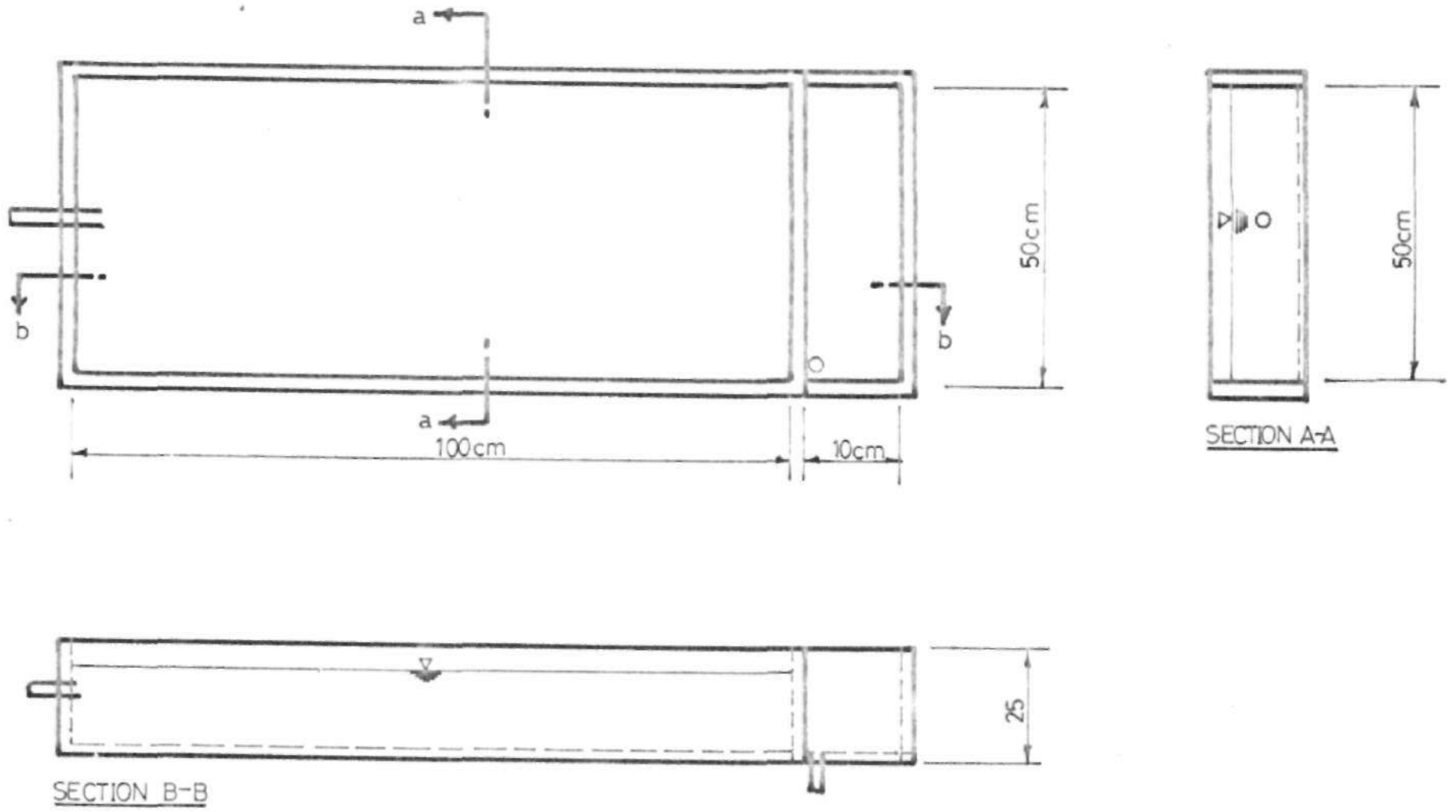
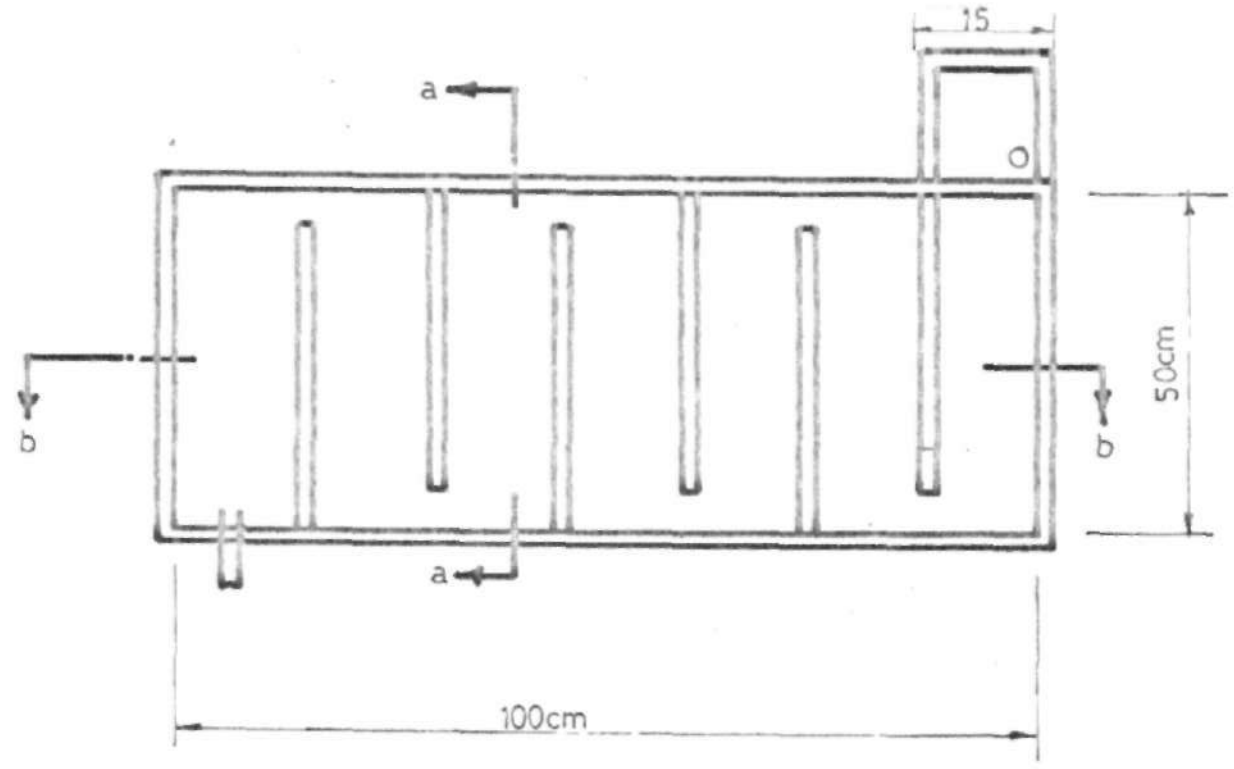
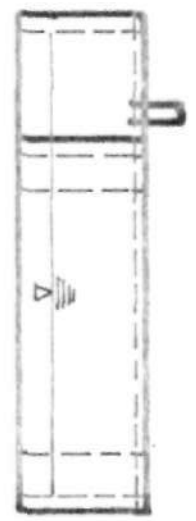
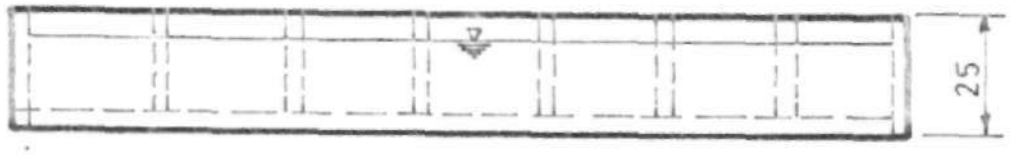


FIG 5-1-1 - MODEL OF PLAIN FACULTATIVE STABILIZATION POND (CONTROL MODEL TANK A)





SECTION A-A



SECTION B-B

FIG 5-1-2 MODEL OF 6 - BAFFLED FACULTATIVE STABILIZATION POND (TANK B)

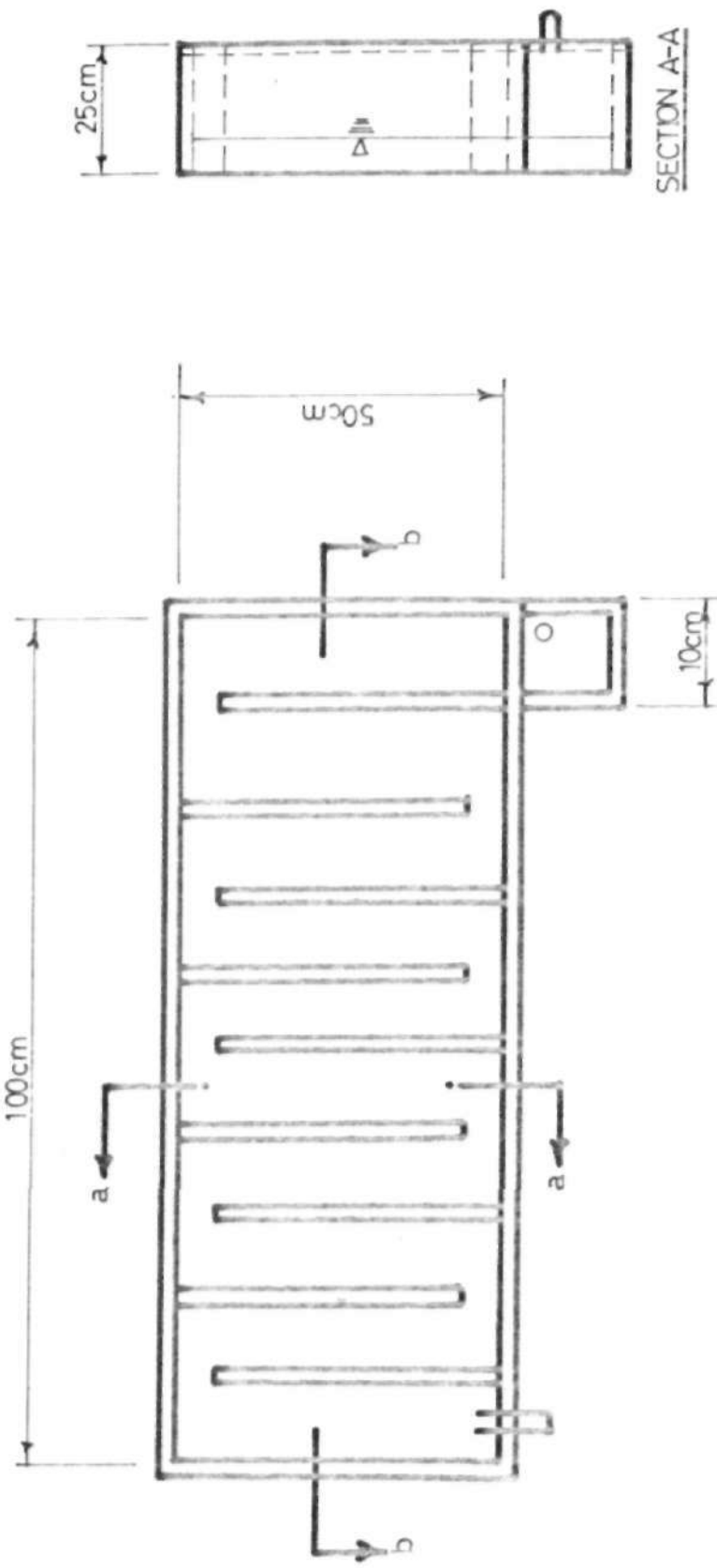


FIG 5-1-3 MODEL OF 9- BAFFLED FACULTATIVE STABILIZATION POND (TANK C)

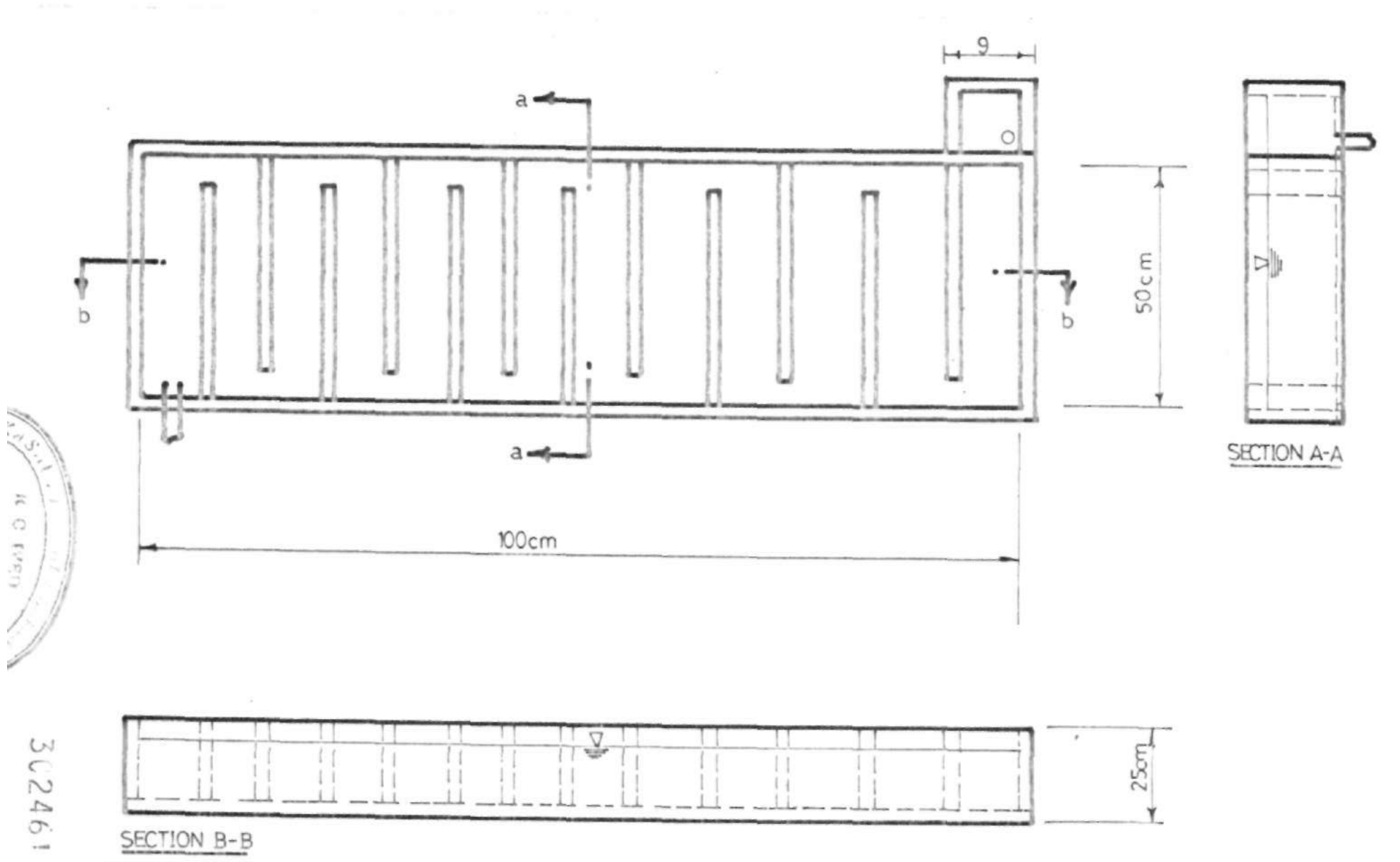


FIG 5-1-4 MODEL OF 12- BAFFLED FACULTATIVE STABILIZATION POND (TANK D)

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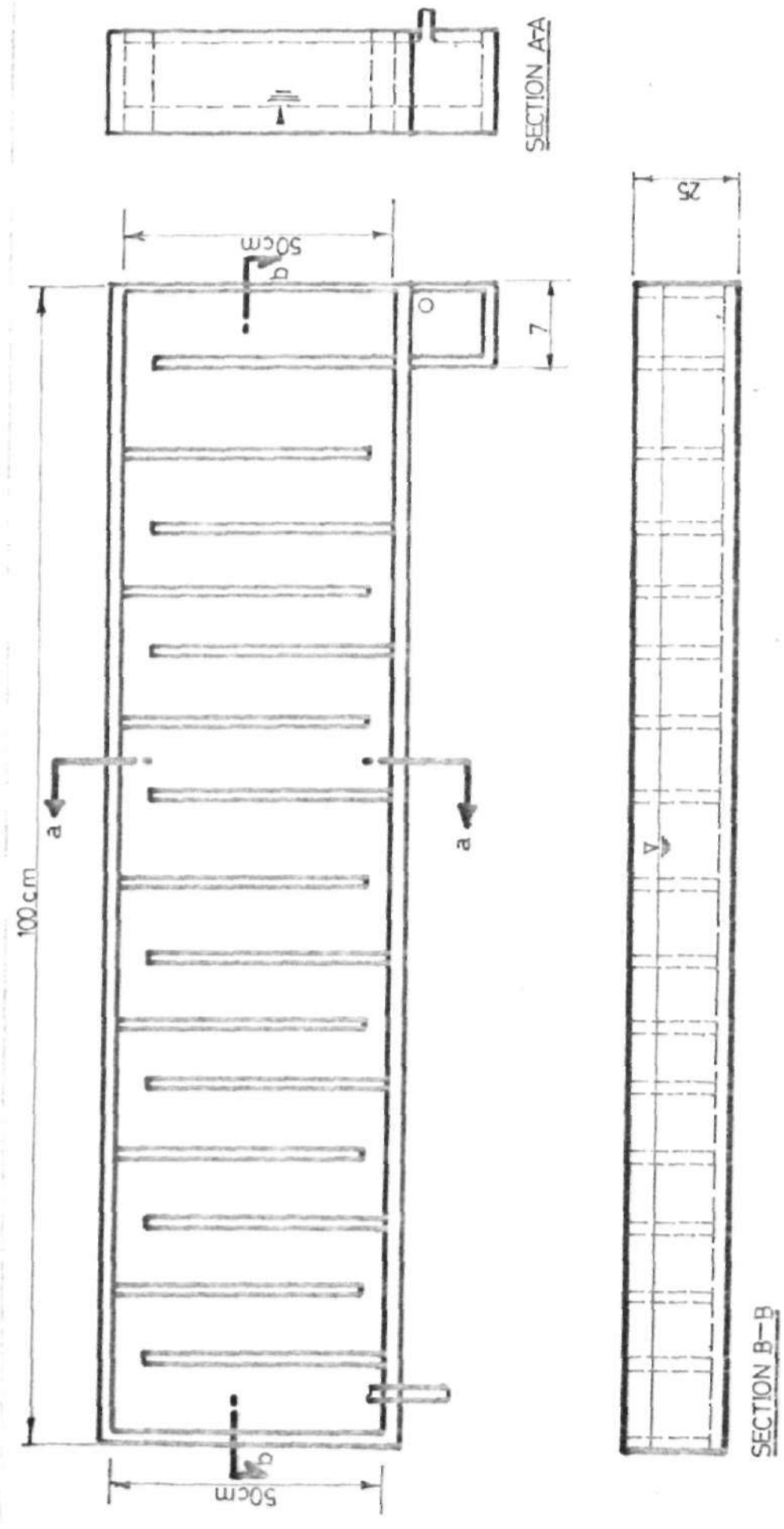


FIG 5-1-5 MODEL OF 15 '- BAFFLED FACULTATIVE STABILIZATION POND (TANK E)

TABLE 5-1-1: CHEMICAL COMPOSITION OF FUSSELS CONDENSED MILK (ALUKO, 1976)

CONSTITUENTS	PERCENTAGE BY WEIGHT
Protein	10.20
Sucrose	44.00
Lactose	14.80
Minerals	2.30
Fat	0.20
Moisture	28.50

5.2.1 Tracer Test

The ponds were filled with tap water and the desired flow rate was maintained by pumping tap water from a reservoir using peristatic pumps. After the pumps had maintained steady state flow, 25 mL of Indigo Carmine solution of known concentration was added to the inlet of each pond. Samples were collected from the outlet of the ponds daily and analysed for the concentration of the dye. This was accomplished by measuring the transmittance of the samples using a Spectronic 20 model and obtaining the corresponding concentrations from the calibration curve which was initially prepared (Figure 5-2-1). The tracer recovery curves were prepared by plotting concentrations against time. From these curves and using methods suggested in previous studies the hydraulic characteristics of the ponds were estimated (Kilani and Ogunrombi, 1984). These includ the dispersion index (d), index of short circuiting (α_s), index of dead volume (δ) and actual (mean) detention time (t_g) for each run. Appendix B shows expression for calculating these parameters. The experiments were carried out at four different flow rates of 17.86, 11.36, 8.33 and 6.25 L/day for each pond. From the result obtained the best operational conditions in terms of the number of baffles, minimum dispersion index and the detention time was established.

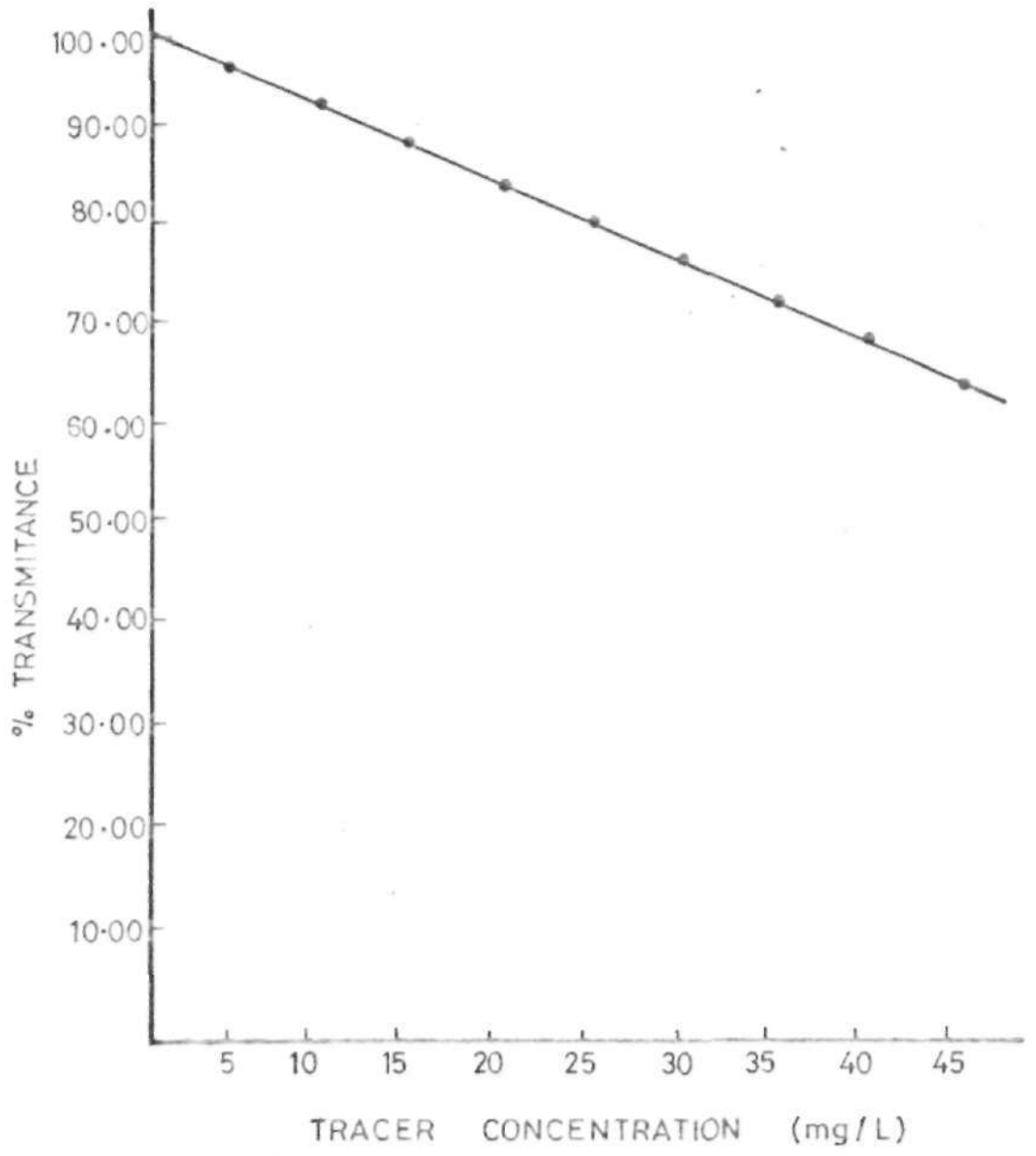


FIG 5-2-1: STANDARD CALIBRATION CURVE FOR TRACER SOLUTION

5.2.2 Treatment Performance Tests

The best operational conditions established in the tracer tests were used for the treatment performance tests. These were carried out by using pond E with 15 baffles and 15 days detention time and the plain pond as control.

In order to avoid unstable pH condition and also to conserve the synthetic milk waste it was decided to prepare a standard curve for the weight of the synthetic milk waste needed to give any predetermined organic loading (Figure 5-2-2). With this curve only the amount of milk needed was taken and prepared as feed, the remaining part was refrigerated at about $4 \pm 1^{o}C$. Refrigeration sometimes had been found to enhance further stability (Thirumurthi, 1967). 50L feed in an aspirator was pumped at about 17L/day flow rate into the two ponds operating in parallel. Thus the feed would last just about 3 days. So in addition to the above advantages of minimum pH depression and conservation of feed, the problem of evaporation became negligible.

The average liquid content weight of each tin of milk used was 434g. 1 litre of water was put in 9 one liter bottles. In each bottle different amounts of milk were put as indicated below (Table 5-2-1). The corresponding BOD_5 values are also indicated in the table.

To ensure that the pH would be around 8.0, small amount of NaOH was always added (Aluko, 1976).

TABLE 5-2-1: WEIGHTS OF CONDENSED FUSSEL MILK WITH CORRESPONDING BOD_5

Bottle No.	1	2	3	4	5	6	7	8	9
BOD_5(mg/L)	217	434	651	868	1085	1302	1519	1736	2170
Weight of milk (mg)	23	40	61	79	100	110	154	172	217

From these sample solutions with DO meter and standard BOD bottle test, the corresponding BOD values were determined. Concurrently with the same solution, the COD values of organic loading corresponding to those of each BOD values were also determined in accordance with Standard Method (Table C-2-1). Due to the advantage that under this study, one is able to control the organic loading, a correlation coefficient was sought for COD and BOD such that because of the rapidity with which COD test can be performed, a quick COD test is all one needs initially as a guide when new feeds are prepared before the conventional long 5-day BOD result is available.

With this preliminary test two things were achieved. First one was able to get the standard curve for the amount of milk to give a desired BOD (Figure 5-2-2). Three runs were done and the mean gave a BOD - milk concentration linear regression equation of

$$W_m = 0.08\ BOD + 5.27 \qquad 5\text{-}2\text{-}1$$

$$r = 1.0$$

W_m = Milk weight added in 5OL aspirator (mg).

r = Correlation coefficient

Secondly correlation coefficients, r of BOD - COD of each of the runs were obtained with the mean of the regression equations given as (Figure 5-2-3).

$$BOD = 0.66\ COD + 4.60 \qquad 5\text{-}2\text{-}2$$

$$r = 1.0$$

Initially, each pond was filled with tap water and seeded with the synthetic milk waste and raw sewage from the main campus of Ahmadu Bello University, Zaria, Nigeria. The contents of the ponds were thoroughly mixed and left for some days for algae suspension to develop. The experiments were carried out under a continuous flow condition using peristatic pumps. The synthetic milk waste was pumped from the storage tank at a constant flow rate of 8.33 L/day giving detention time of 15 days for each experiment (Figure 5-2-4). The experiments were carried out at five different aerial organic loadings of 33, 50, 100, 150 and 200 kg BOD/ha-day.

Sampling of the effluent for physical and chemical analyses began after the ponds were operated for a period equal to the theoretical detention time when the initial contents of the ponds appeared to have been completely flushed out. The performance of the ponds was evaluated by the analyses of both the influent and effluent samples for BOD, COD and solids. The BOD_5 and COD measurements for both filtered and unfiltered effluent samples were made in accordance with the "Standard Methods". The filtration of the effluent samples to remove algae was accomplished by using Whatman Number GF/C filter paper. The BOD was determined at 24°C, an average temperature of the ponds liquid contents and an average temperature considered appropriate for BOD determination under the Tropical Humid Climate with seasonal rainfall (Onuoha, 1980).

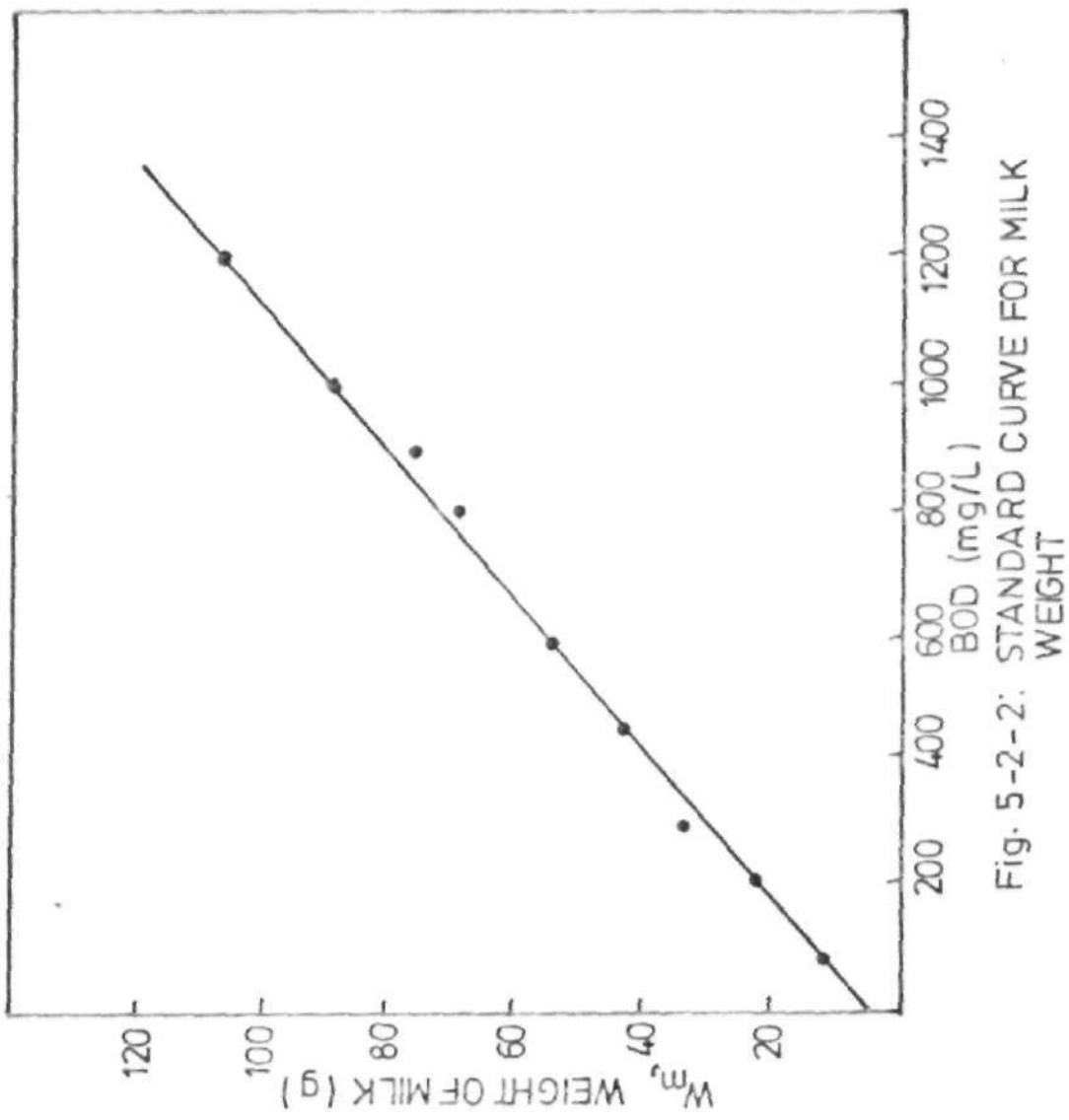


Fig. 5-2-2: STANDARD CURVE FOR MILK WEIGHT

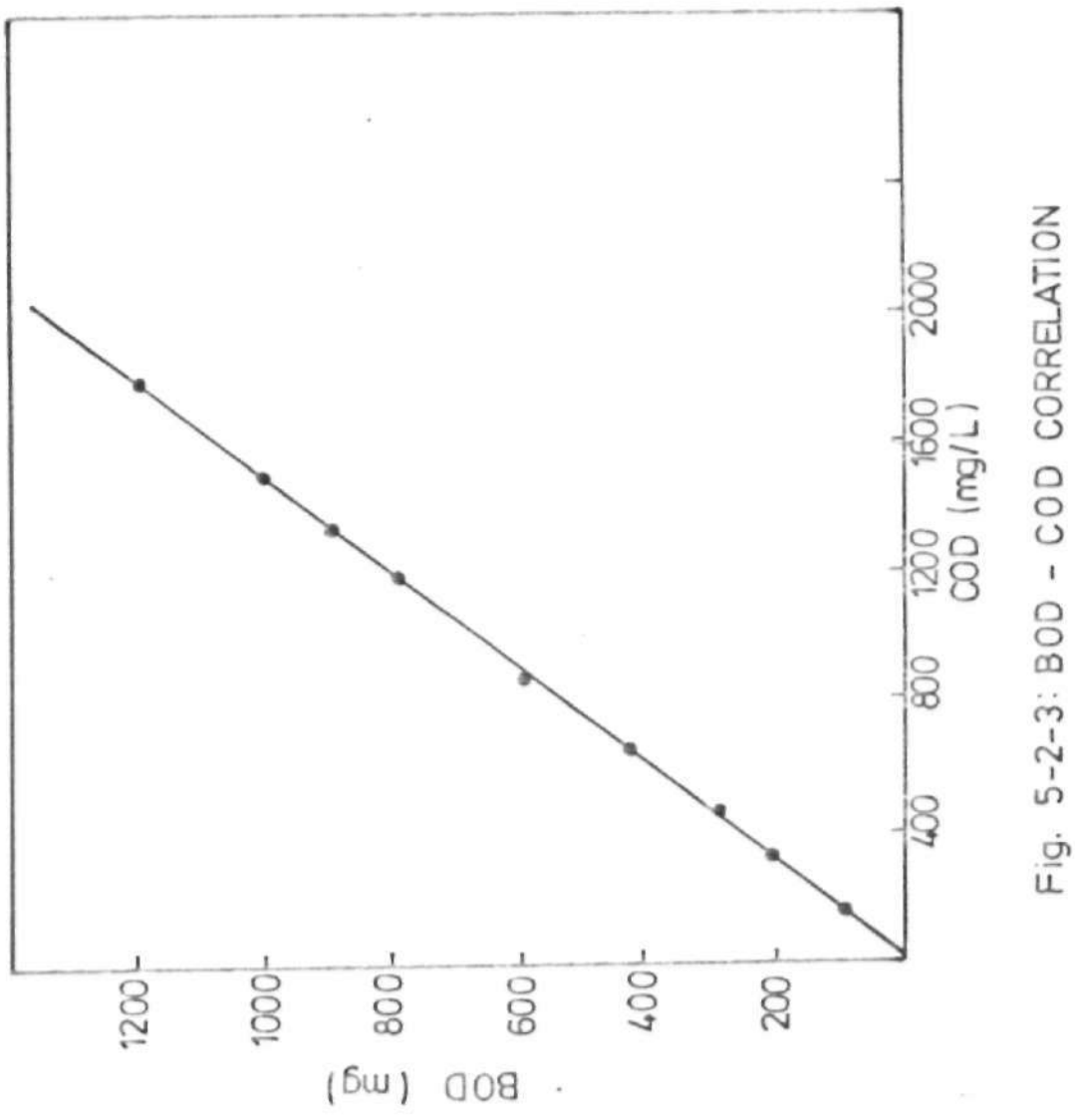


Fig. 5-2-3: BOD - COD CORRELATION

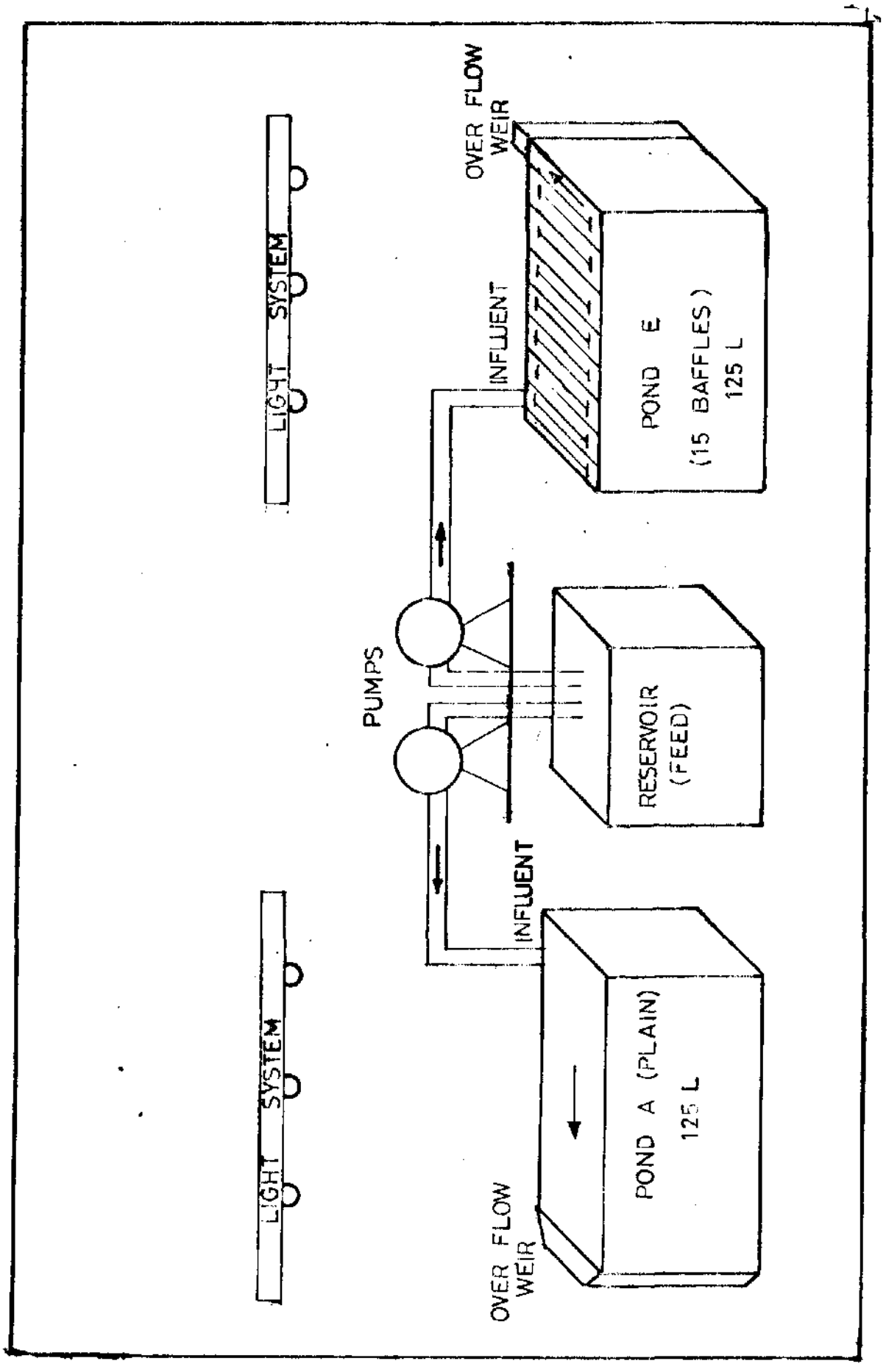


FIG 5-2-4: LABORATORY MODEL PONDS

The analyses of Dissolved Oxygen (DO), suspended solids (SS), pH and alkalinity of the samples were also made in accordance with the Standard Methods. The diurnal changes in DO, alkalinity and pH of the ponds during treatment were also measured at a depth of about 5cm at the center of the ponds. The DO was measured using DO meter model YSI 5700 and the pH was measured by using a Pye Unican model 292 MK-2 pH meter. The diurnal variations in both DO and pH were observed by measuring these parameters bi-hourly over a period of 24 hours. The steady state condition was maintained for about 22 days during which sufficient data were collected.

The average values of the measured parameters at steady state condition were used to evaluate the treatment performance. The experiments were performed at the room temperatures between 21°C and 23°C. Also, microscopic examination of the ponds were carried out in order to identify the dominant algal flora and to determine the algal and bacteriological counts in the ponds. (C-2-7).

Algae were counted after proper dilution had been made using counting chamber. In order to inhibit the algal movements $CHCl_3$ (chloroform) was used (Anyam, 1984). The bacteriological counts for Total Coliform (TC), Faecal Coliform (FC) and Faecal Streptococci (FS) were enumerated using filtration method (McKinney, 1962, 1982; Mara, 1974 and Irving et al, 1983). Filtration methods is always more convenient and faster than either the Most Probable Number (MPN) or the Pour Plate techniques (Mara, 1974 ,(a)).

CHAPTER 6

6.0 RESULTS

This chapter consists of the results of the experimental investigations and are presented under three separate sections. These include the mixing characteristics of the ponds, the performance tests for evaluating the efficiencies of the ponds in terms of organic removals, changes in the ponds treatment parameters such as Alkalinity, Dissolved Oxygen (DO), and pH while the third section is on the estimation of the bacterial die-off and algal concentrations in the ponds during the treatment.

6.1.0 Mixing Characteristics

Figures 6-1-1 through 6-1-4 show the tracer recovery curves at different theoretical detention times of 7, 11, 15 and 20 days respectively. Also Table 6-1-1 shows the hydraulic characteristics of the ponds which were calculated from the information obtained from the tracer test curves. These include the following:

t_n = Nominal (Theoretical) Detention Time (days)

t_i = Time of initial appearance of tracer at the effluent (days)

t_p = time to reach peak or maximum concentration (days)

t_s = Standard Detention Time (days)

t_g = Time to reach centroid of the curve or Mean (Actual) Detention Time (days)

t_η = Standard Detention Time Efficiency

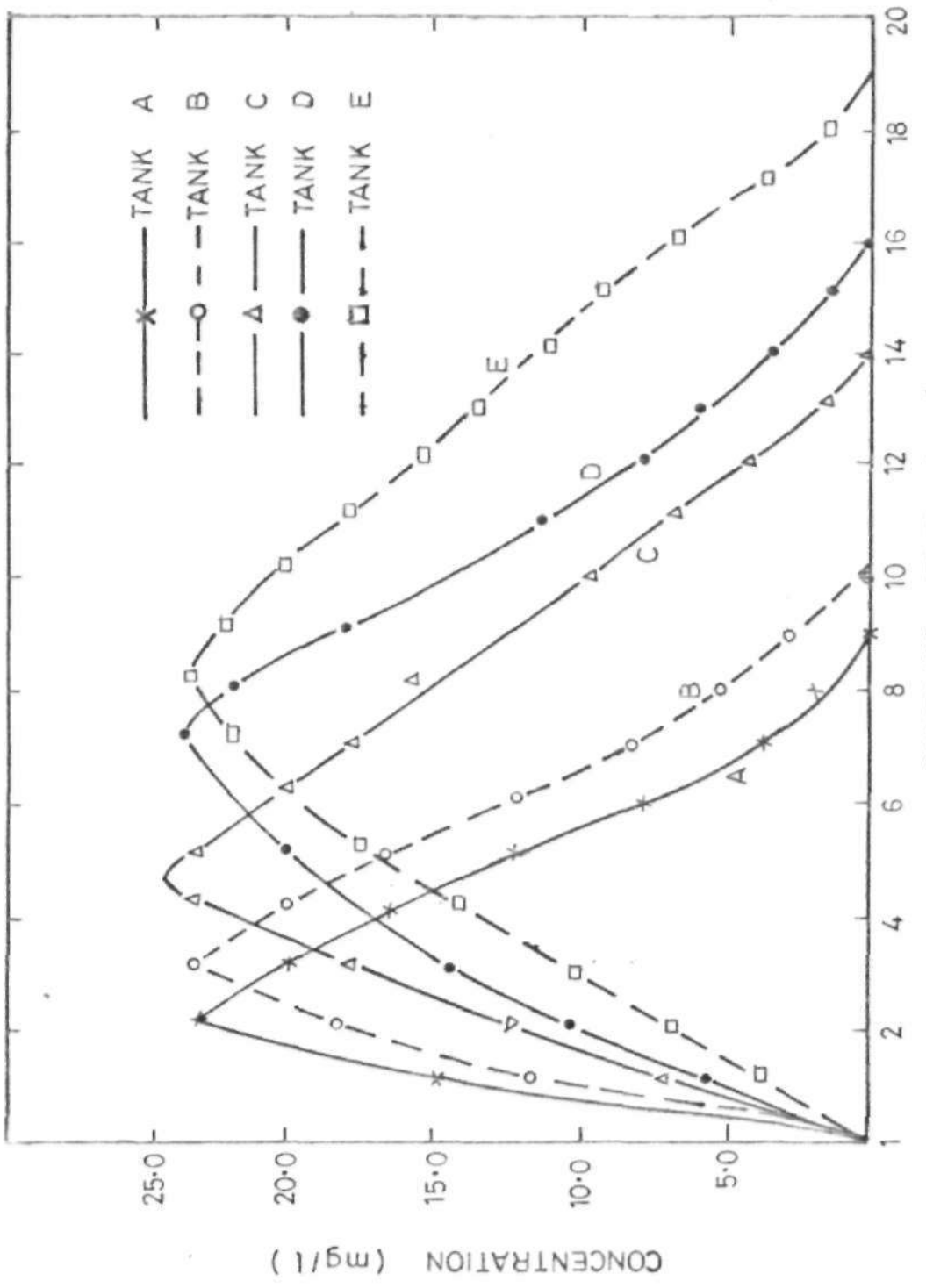


FIG 6-1-1: TRACER TEST CURVES No 1 (7 DAY)

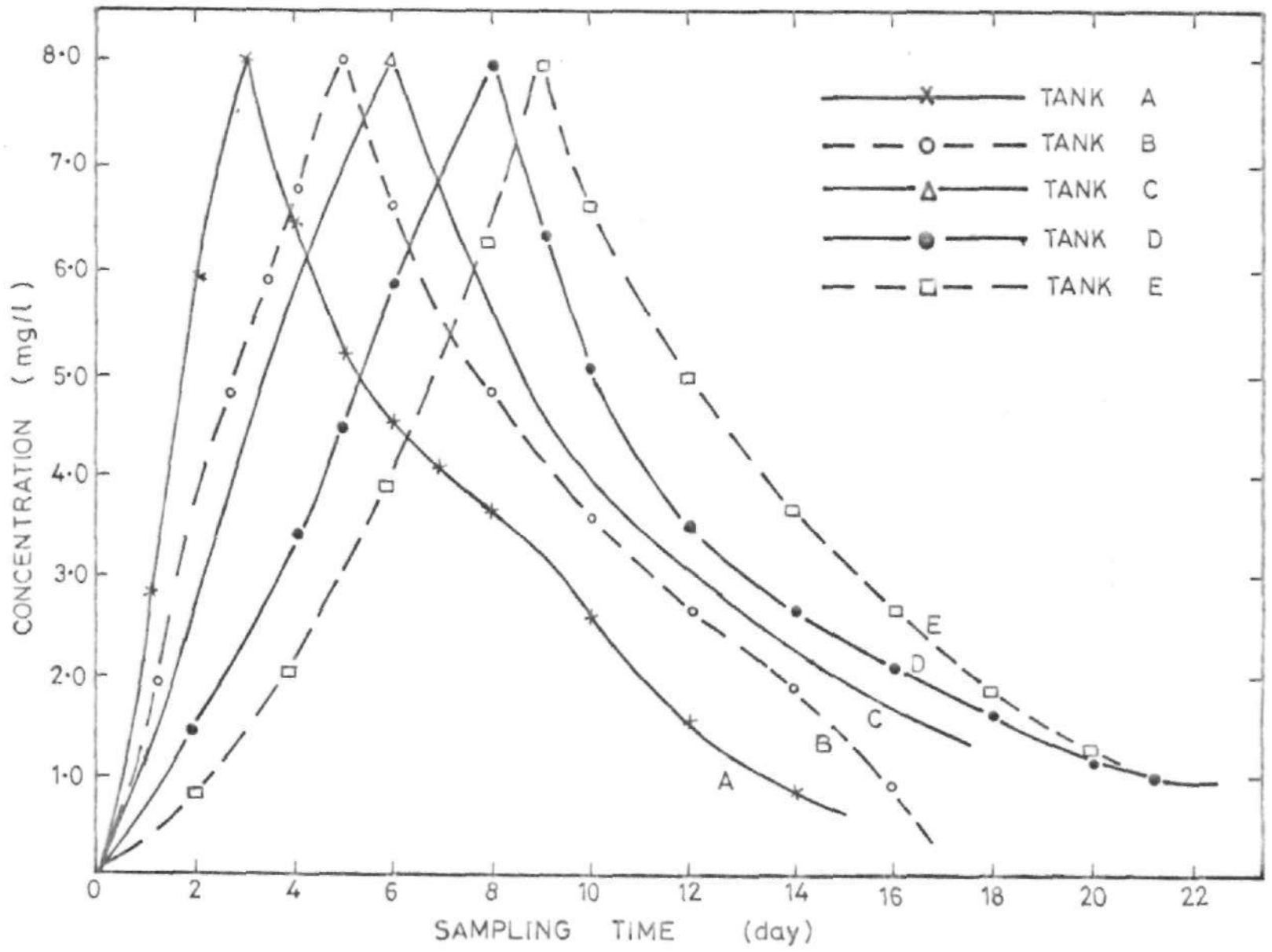


FIG 6-1-2: TRACER TEST CURVES No 2 (11 DAY)

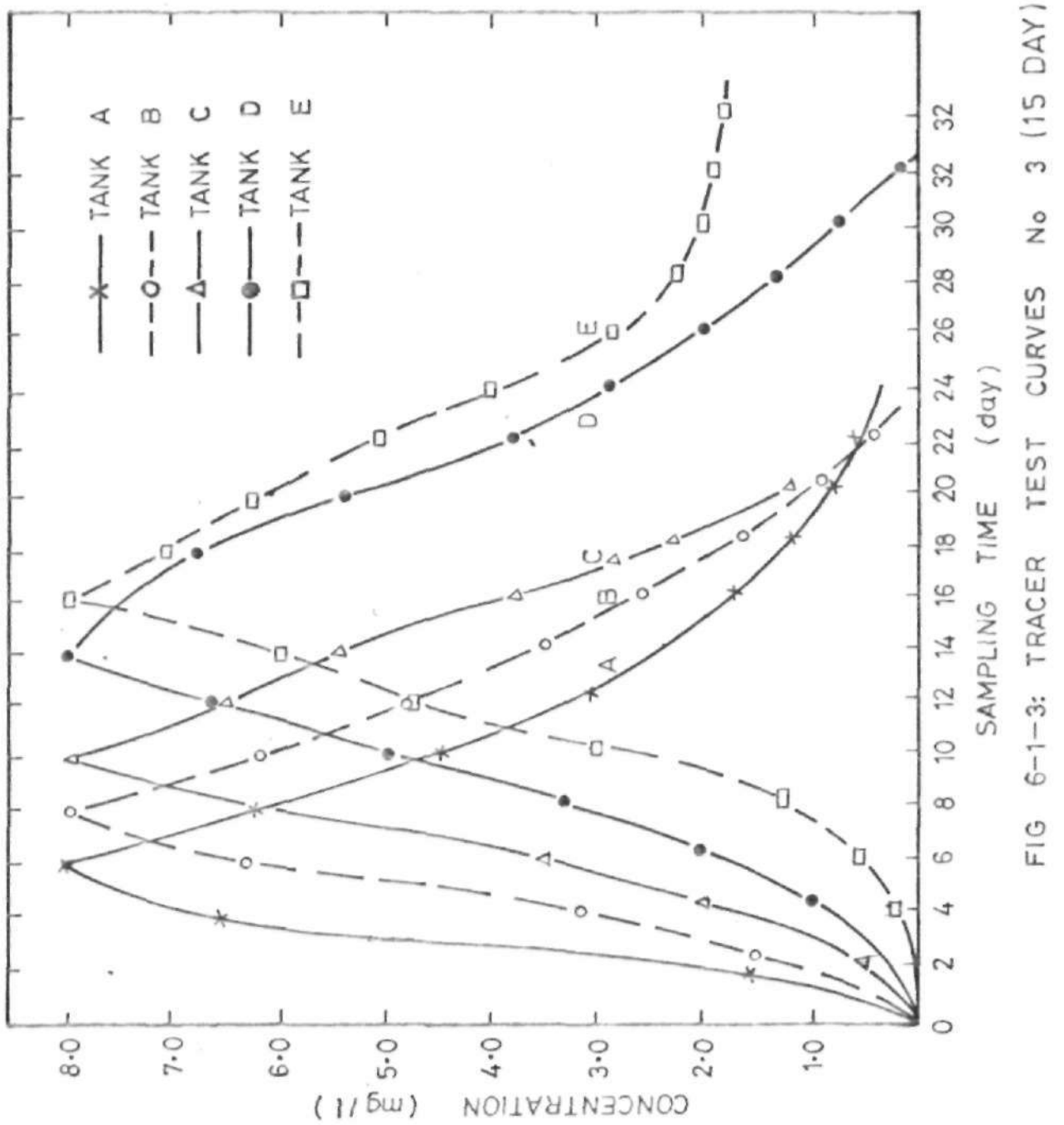


FIG 6-1-3: TRACER TEST CURVES No 3 (15 DAY)

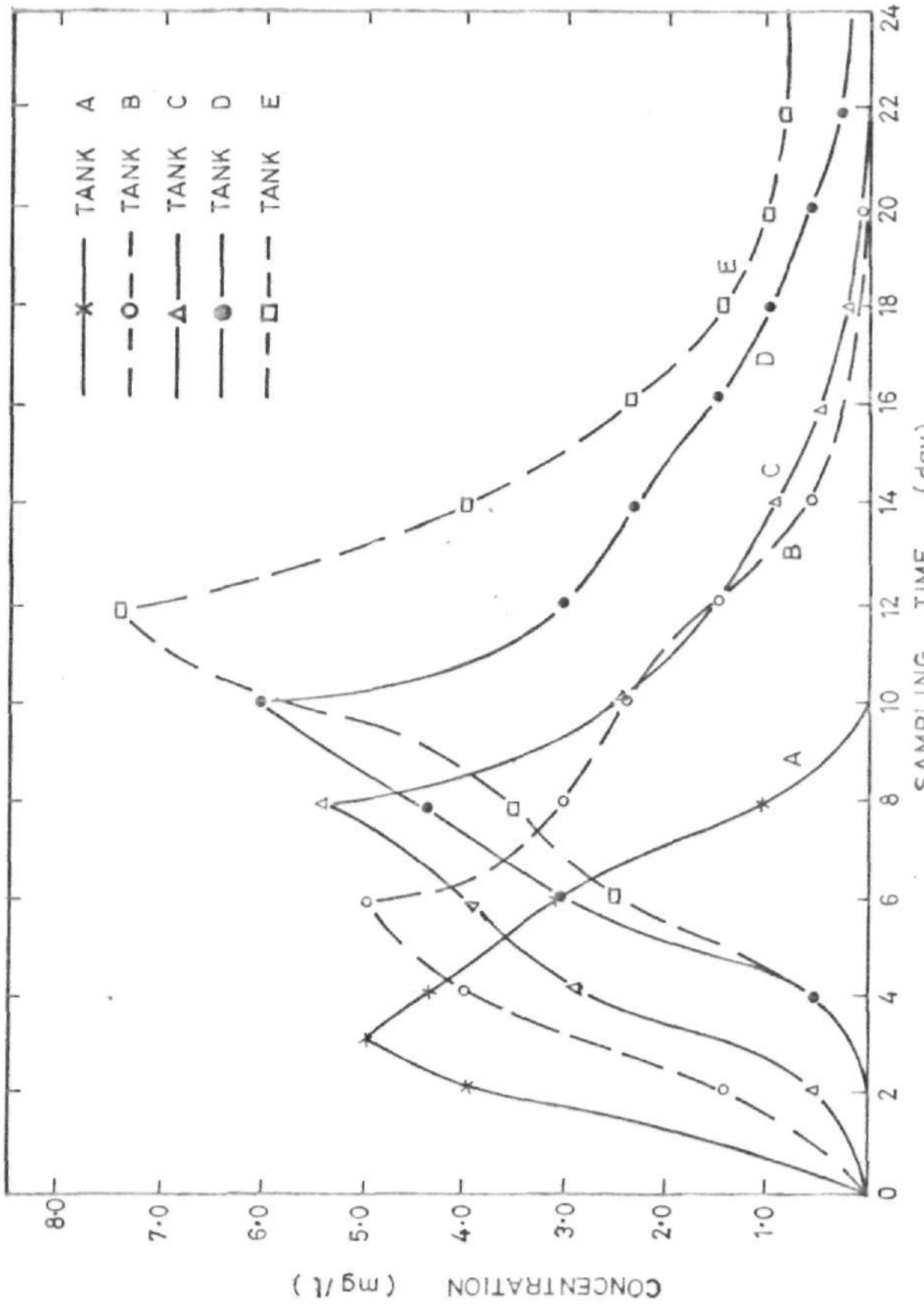


FIG 6-1-4: TRACER TEST CURVES No 4 (20 DAY)

TABLE 6-1-1: SUMMARY OF THE TRACER TEST DERIVED DATA

TEST NO.	PONDS	t_n (days)	t_i (days)	t_p (days)	t_s (days)	t_g (days)	t_η (%)	α_d	δ (%)	α_s	ψ	d
				P	A	R	A	M	E	T	E	R S
1	A	7	1	2.00	2.11	3.37	30.00	0.48	52.00	0.410	70	0.178
	B	7	1	3.00	2.58	4.07	37.00	0.58	42.00	0.260	53	0.159
	C	7	1	4.50	3.88	5.93	55.00	0.85	15.00	0.240	17	.133
	D	7	1	7.00	4.64	6.98	66.00	1.00	02.00	-0.003	0	.126
	E	7	1	8.00	6.07	8.83	87.00	1.26	-26.00	-0.094	-23	.111
2	A	11	1	3.00	3.96	5.85	36.00	0.53	47.00	0.487	61	.160
	B	11	1	5.00	7.93	7.10	72.00	0.65	35.00	0.296	43	.146
	C	11	1	6.00	5.33	7.95	48.00	0.72	28.00	0.245	16	.136
	D	11	1	8.00	6.80	9.67	62.00	0.88	12.00	0.173	13	.116
	E	11	1	9.50	8.44	10.90	77.00	0.99	01.00	0.128	1	.094
3	A	15	1	6.00	5.88	8.53	39.00	0.57	43.00	0.297	55	.151
	B	15	2	8.00	7.67	9.89	51.00	0.66	34.00	0.191	41	.114
	C	15	2	10.00	8.96	11.26	60.00	0.75	25.00	0.112	28	.100
	D	15	2	14.50	12.82	16.00	85.00	1.07	-09.00	0.094	-7	.060
	E	15	4	19.00	17.92	19.79	102.00	1.32	-32.00	0.040	-28	.030
4	A	20	1	3.00	2.88	4.17	14.00	0.21	79.00	0.281	131	.174
	B	20	2	6.00	4.95	7.12	25.00	0.35	64.00	0.157	95	.154
	C	20	2	8.00	6.29	8.29	31.00	0.41	59.00	0.156	83	.136
	D	20	4	10.00	8.78	10.50	44.00	0.53	48.00	0.048	62	.112
	E	20	4	12.00	9.90	11.74	50.00	0.59	41.00	0.022	52	.099

α_d = Index of dead space

δ = Percentage Dead space (volume)

α_s = Index of short-circuiting

ψ = Percentage Difference of actual and nominal Detention times

d = Dispersion Number (Index)

Details of the sample calculations of some of the parameters are given in Appendix B.

Figures 6-1-5 through 6-1-14 show some of the relationships between the derived mixing characteristics of the ponds.

6.2.0 Performance Test

The analysis of the results of the tracer tests showed that Pond E with 15 baffles had the best operating conditions in terms of the mixing characteristics of the ponds. Consequently, this pond together with Pond A without baffle (control pond) were used for the treatment performance test. Also for the same reason the theoretical detention time of 15 days was maintained throughout the performance test by keeping both the ponds' volume and the hydraulic load constant.

Figures 6-2-1 (a) through 6-2-5 (f) show the results of analysis of the effluent parameters for BOD_5, COD, Suspended Solids (SS), Alkalinity, Dissolved Oxygen (DO) and pH. Tables 6-2-1 (a) to 6-2-1 (e) give the analyses of the milk waste feed and the effluents from the ponds together with their performances.

Table 6-2-2 gave the average values of performance parameters.

Figures 6-2-6 (a) to 6-2-6 (e) show the diurnal variations of both pH and DO. The relationship between some of the effluent parameters are shown in Figures 6-2-7 through 6-2-18.

6.3.0 Biological Features

Table 6-3-1 shows the summary of the variations in algal concentration and the bacterial die-off percentage for both plain and the baffled ponds.

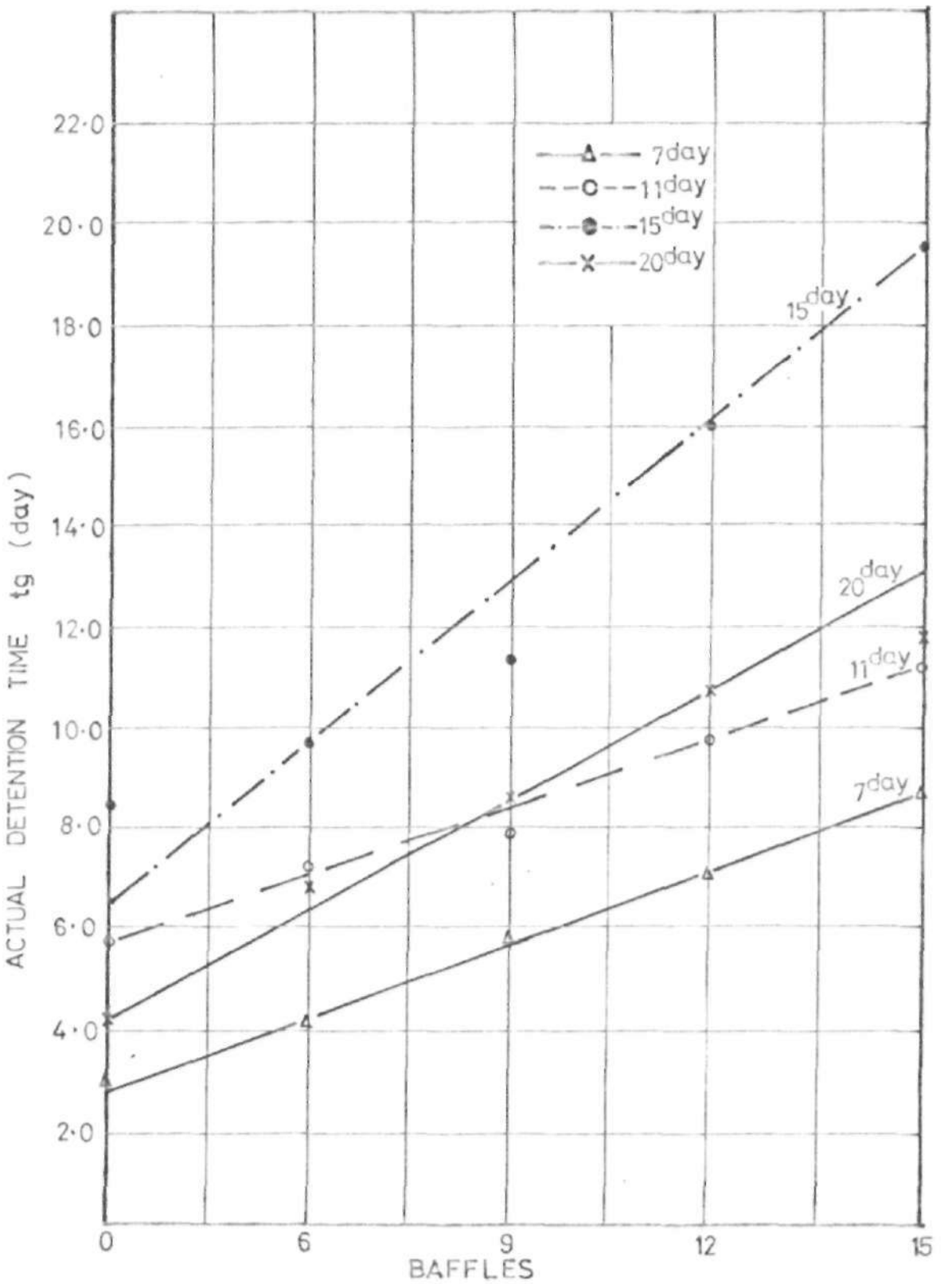


FIG 6-1-5 ACTUAL DETENTION TIME Vs NO OF BAFFLES.

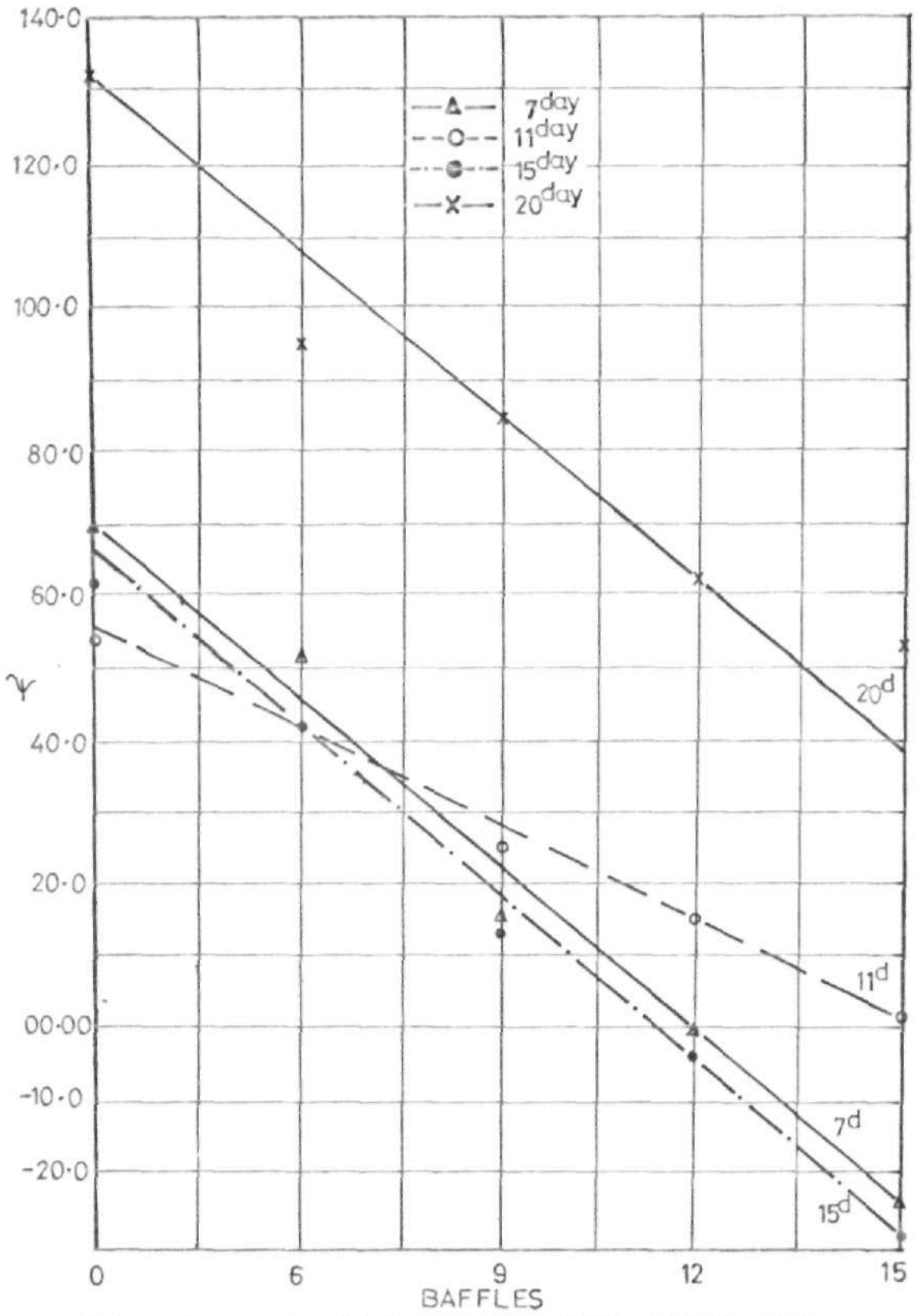


FIG 6-1-6 PERCENTAGE DIFFERENCE ACTUAL AND NOMINAL DETENTION TIME Vs NO OF BAFFLES

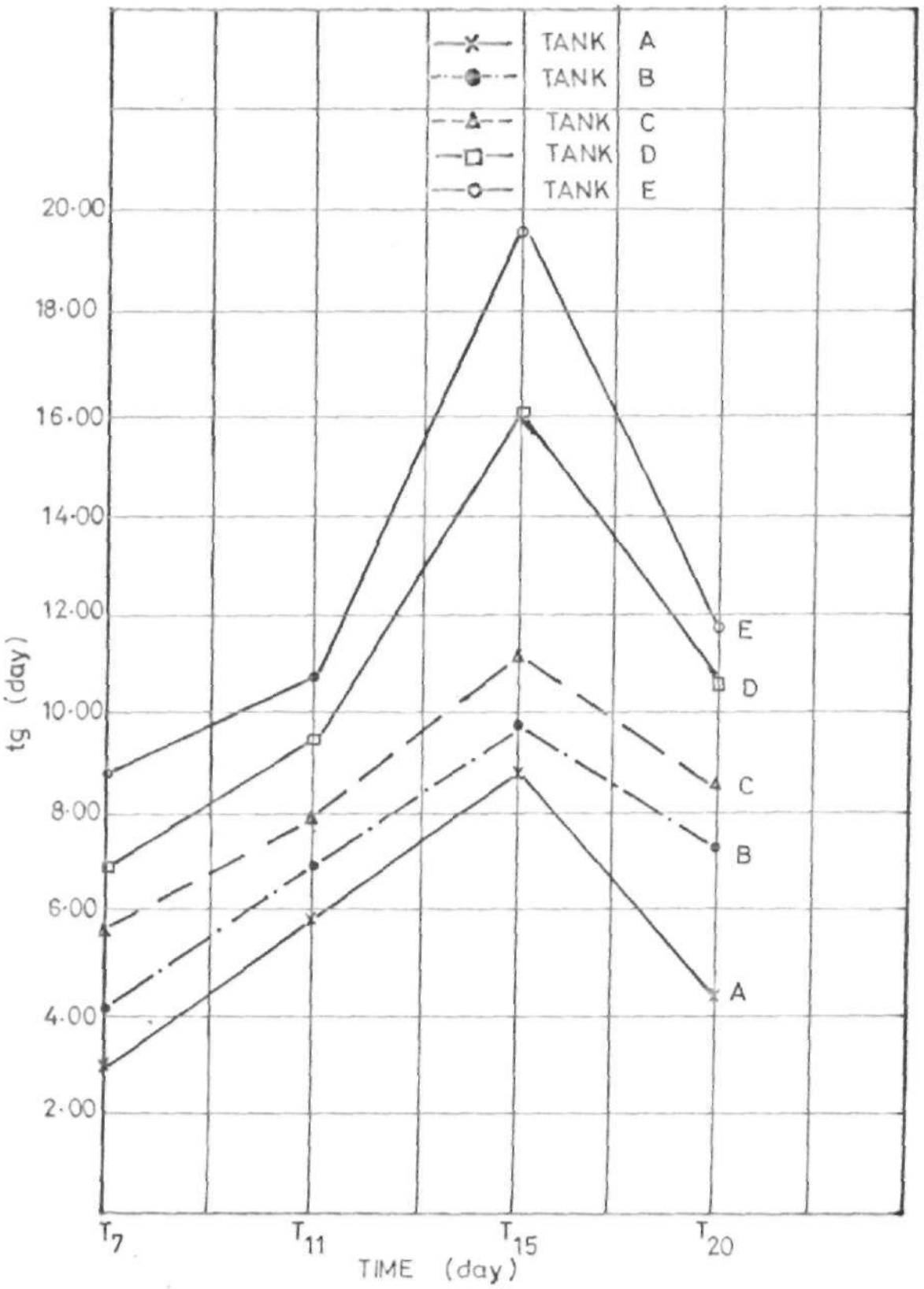


FIG 6-1-7 : ACTUAL DETENTION TIME Vs NOMINAL DETENTION TIME

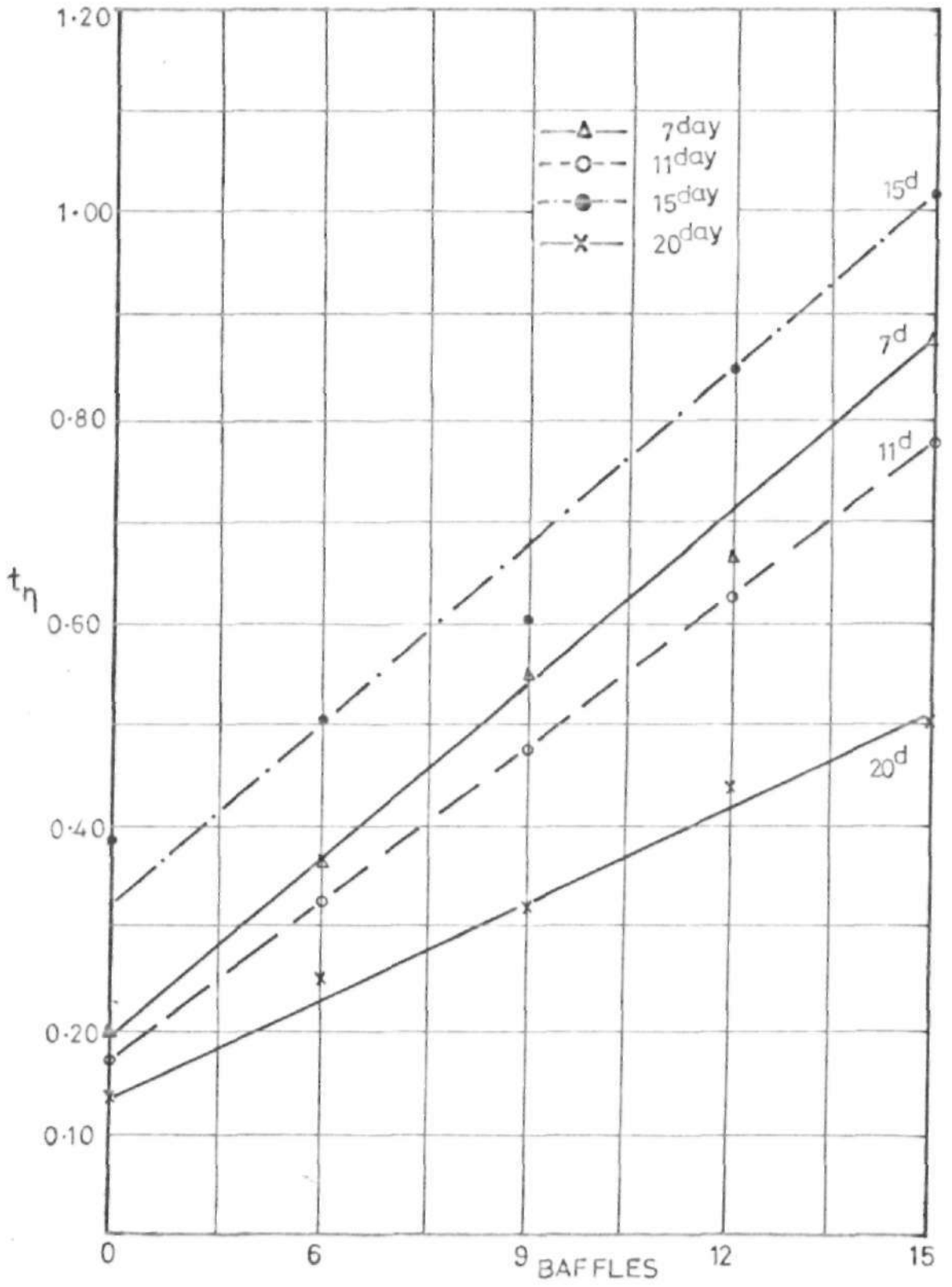


FIG 6–1–8: STANDARD DETENTION TIME EFFICIENCY

Vs NO OF BAFFLES

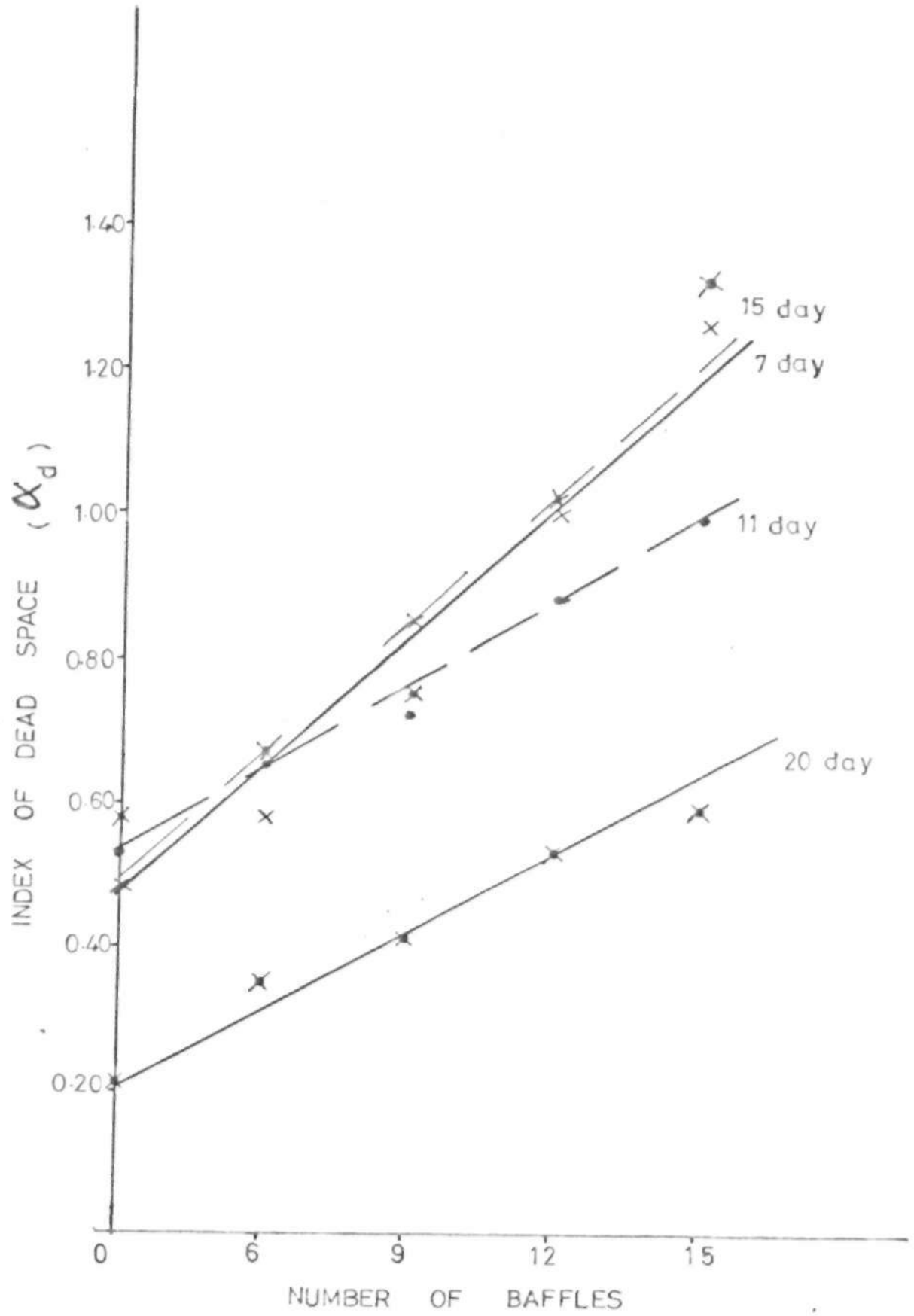
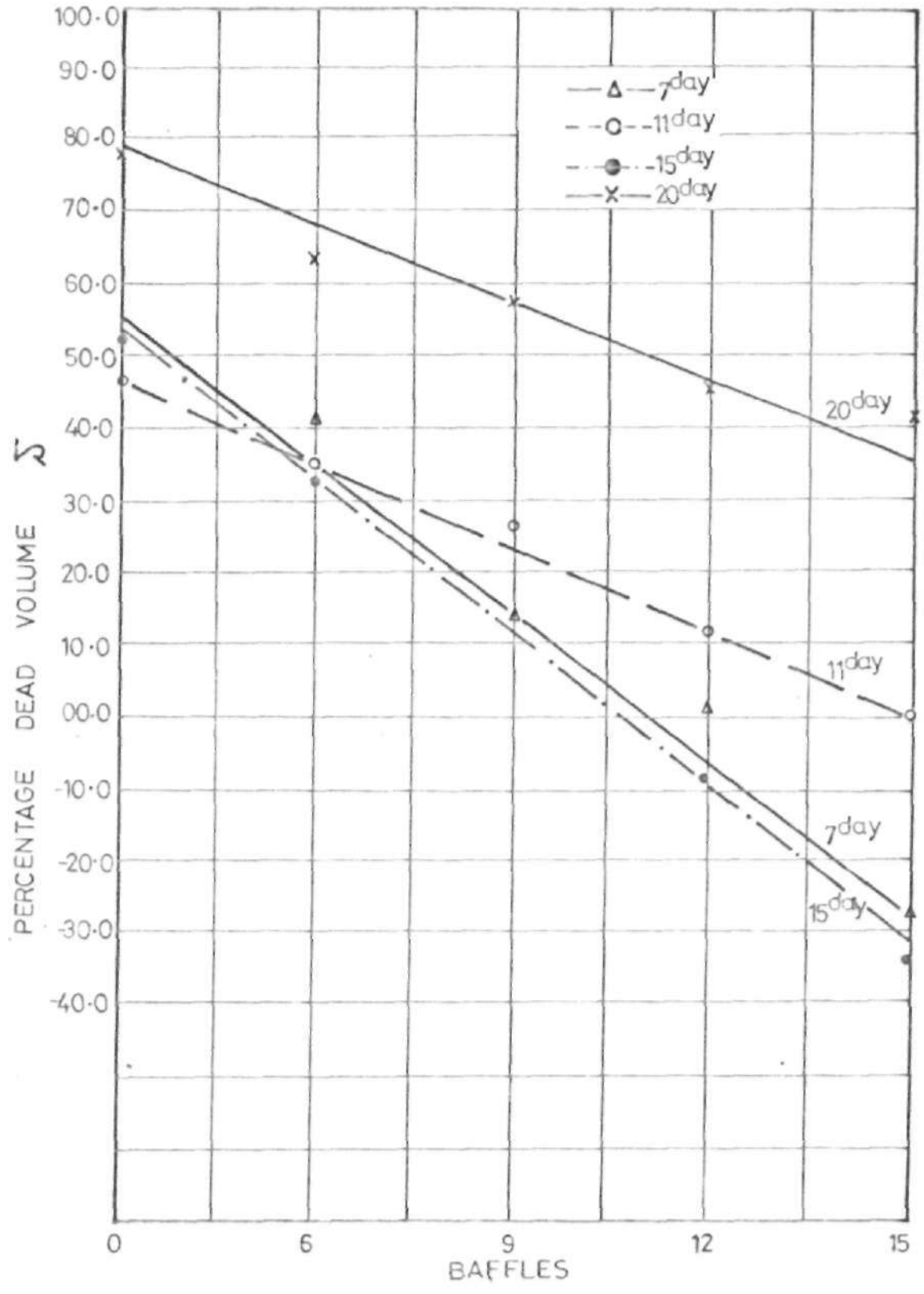


FIG 6-1-9 : INDEX OF DEAD SPACE VS NO OF BAFFLES



FIG· 6-1-10 PERCENTAGE DEAD VOLUME Vs NO OF BAFFLES

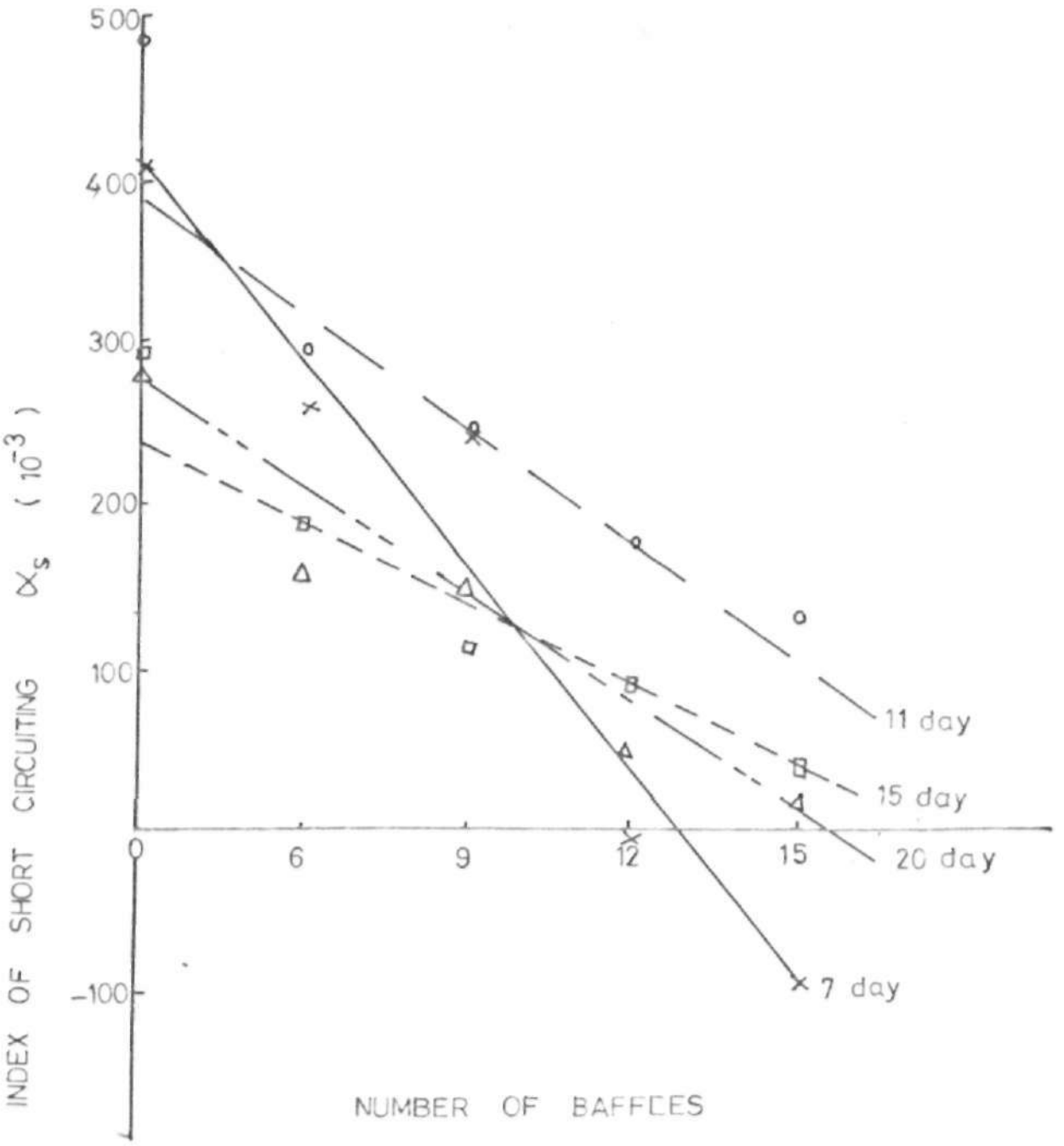


FIG 6-1-11 INDEX OF SHORT CIRCUITING α_s VS NUMBER OF BAFFLES

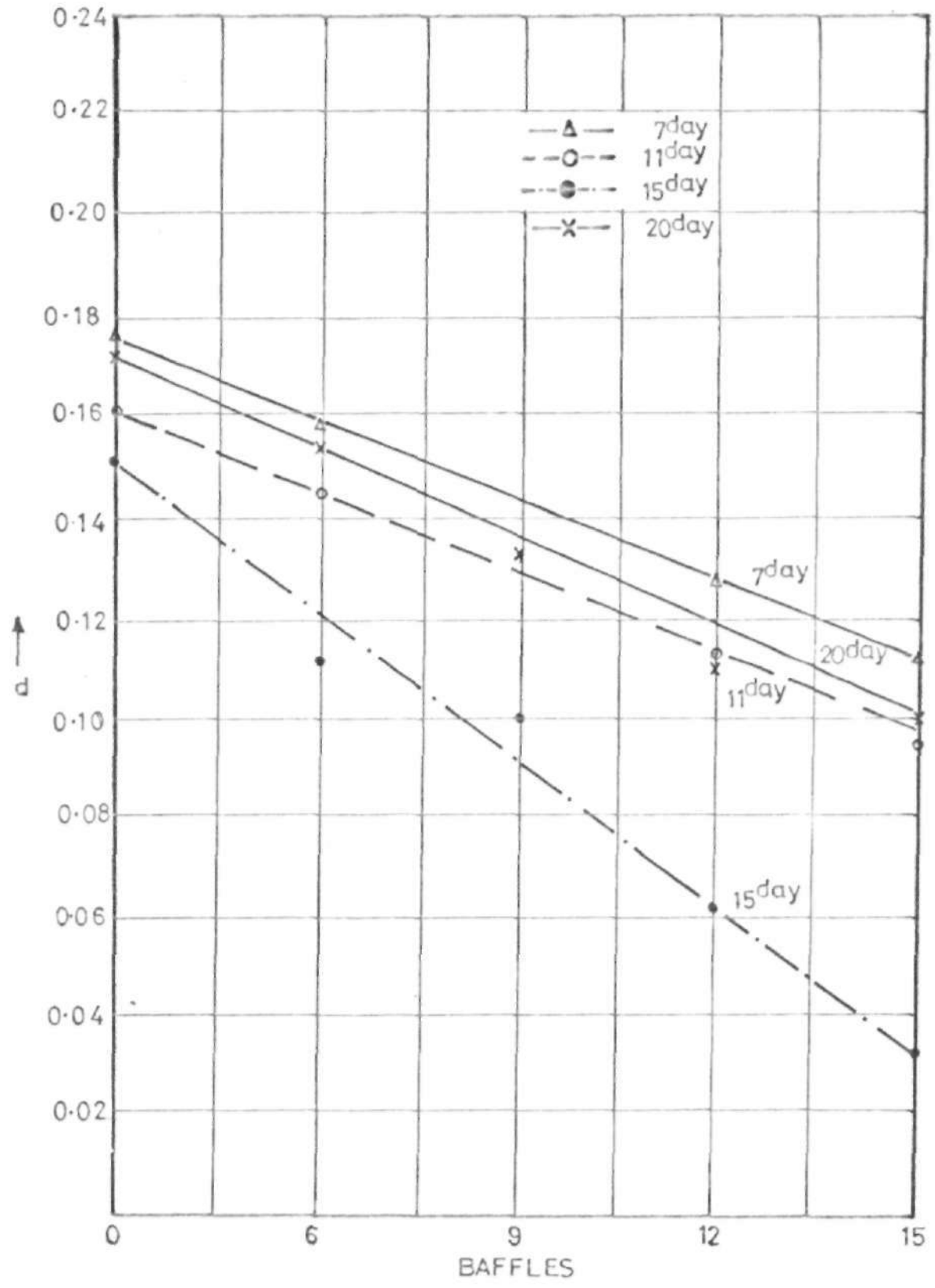


FIG 6–1–12 DISPERSION NUMBER, d Vs NO OF BAFFLES

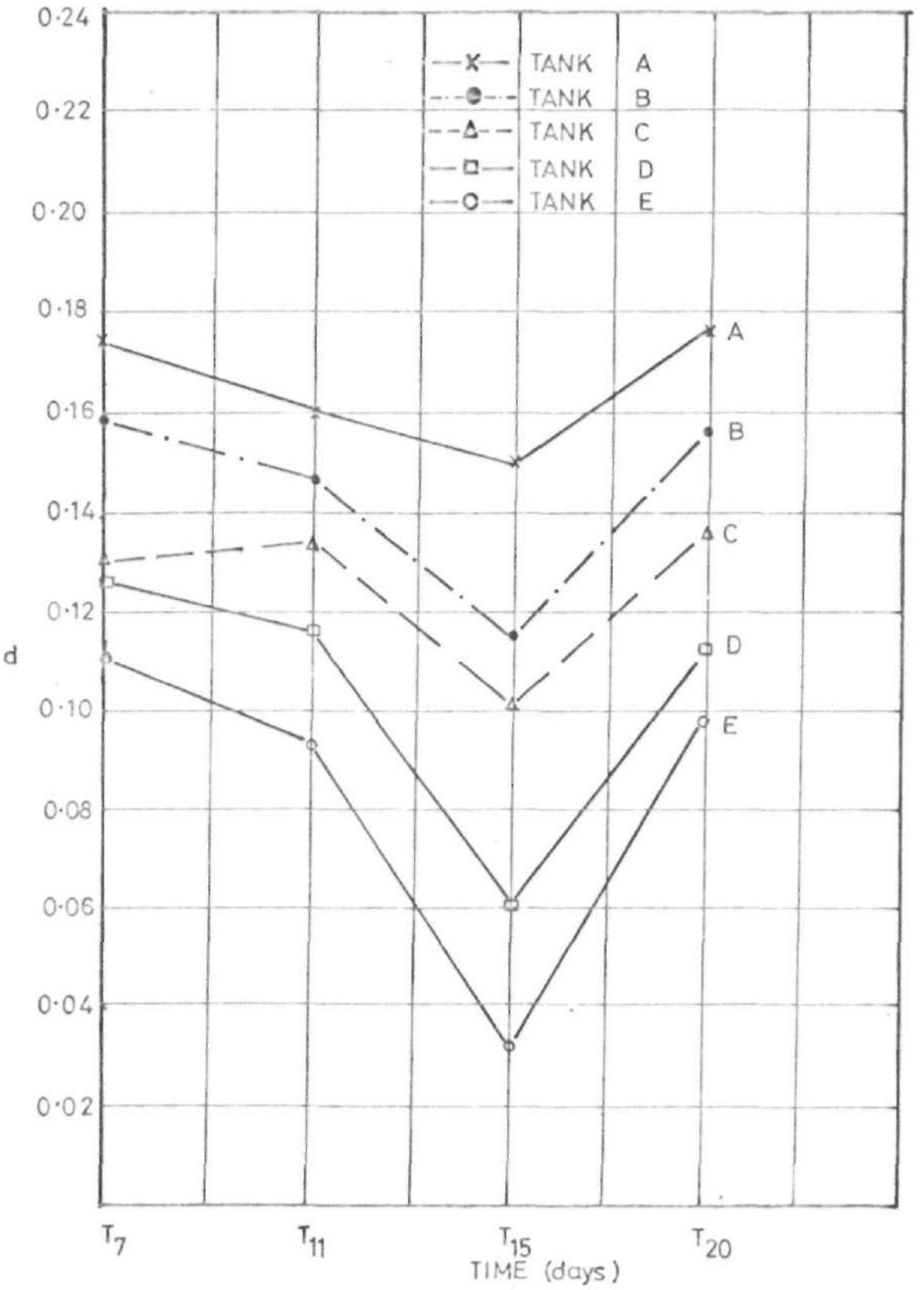


FIG 6-1-13 DISPERSION NUMBER, d Vs DETENTION TIME

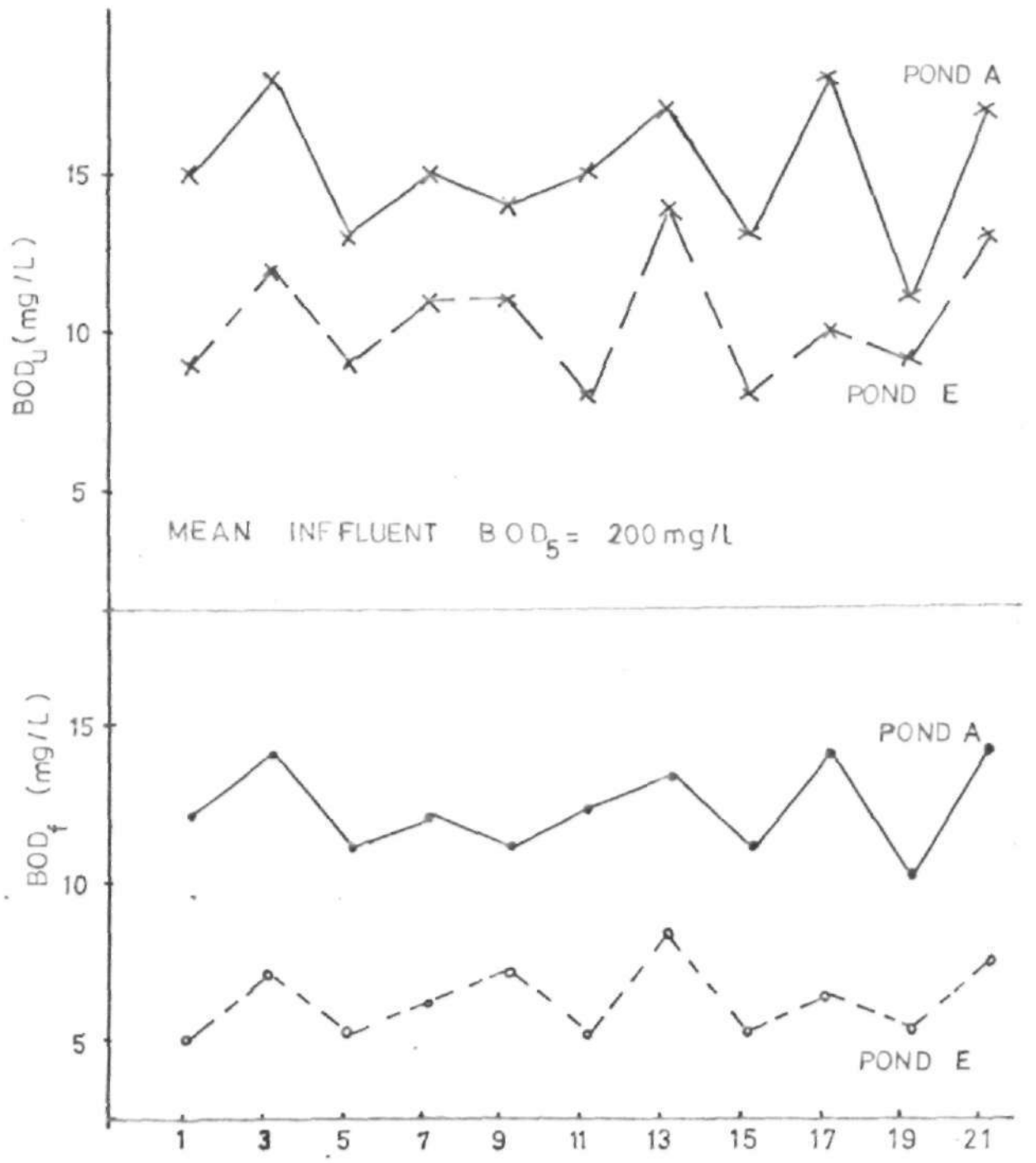


FIG 6-2-1(a): VARIATIONS IN EFFLUENT B O D_5 WITH TIME (TEST 1).

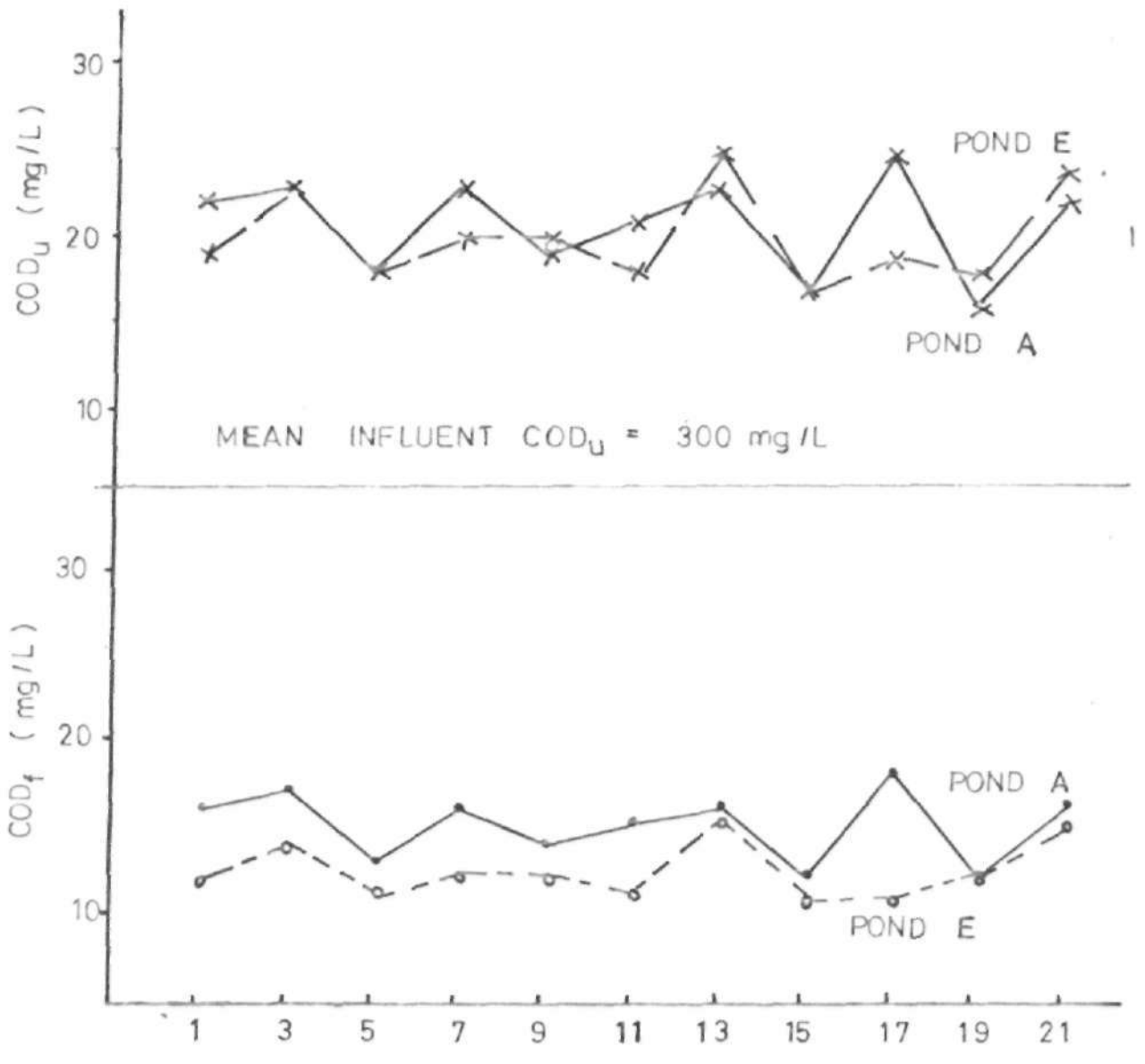


FIG 6-2-1 (b): VARIATIONS IN EFFLUENT COD WITH TIME (TEST 1).

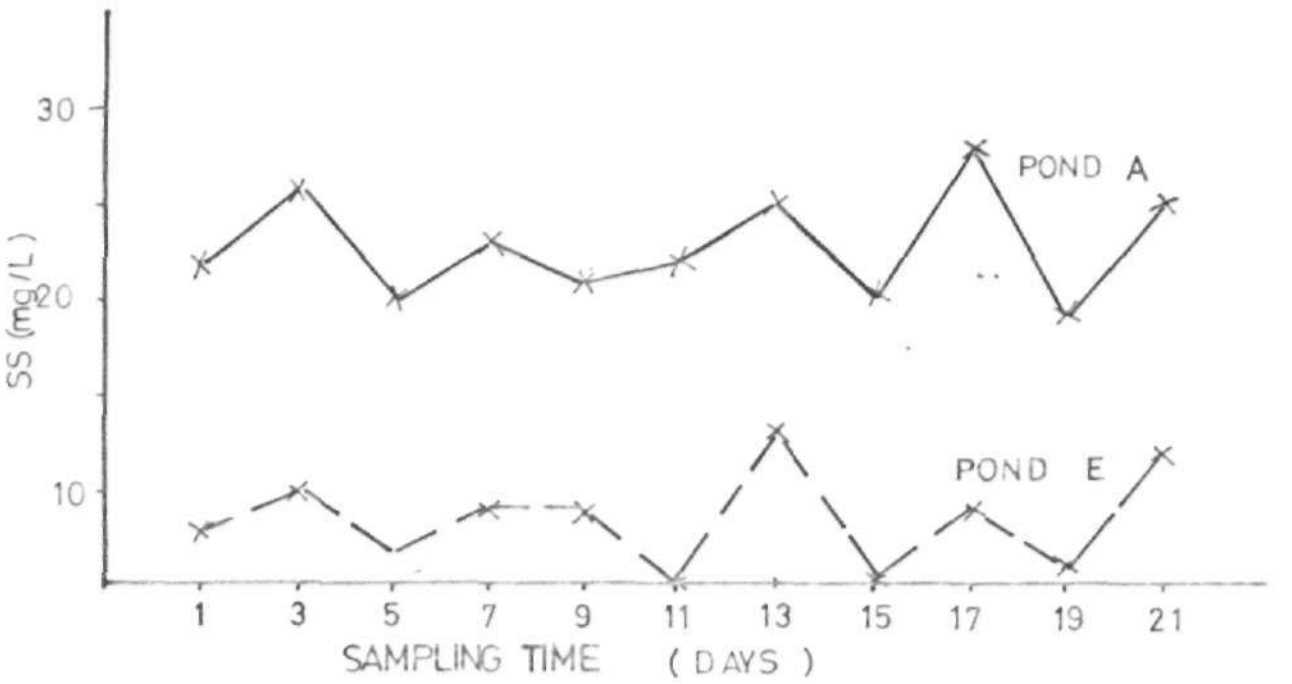


FIG 6-2-1 (c): VARIATIONS IN SUSPENDED SOLIDS (SS) WITH TIME

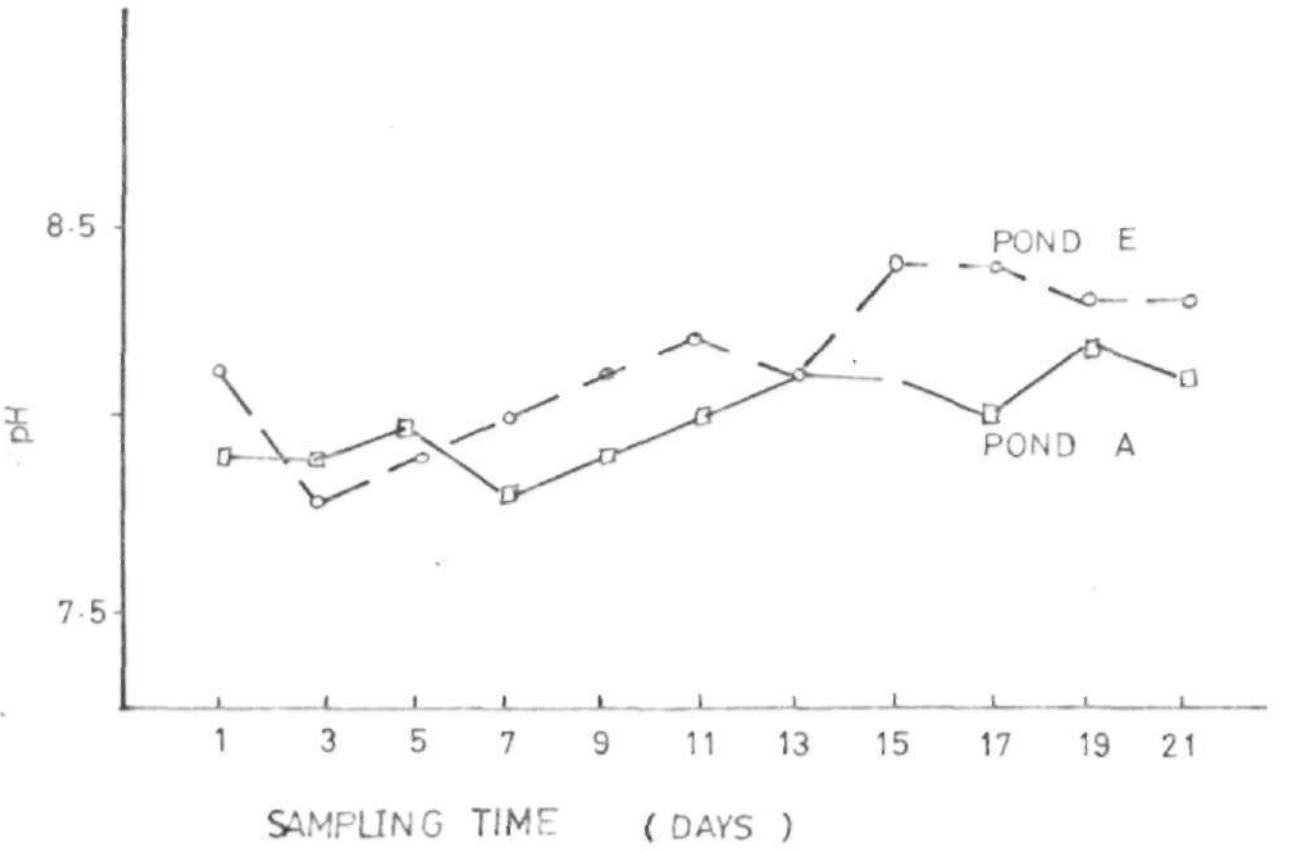


FIG 6-2-1(d): VARIATIONS IN EFFLUENT pH WITH TIME (TEST 1)

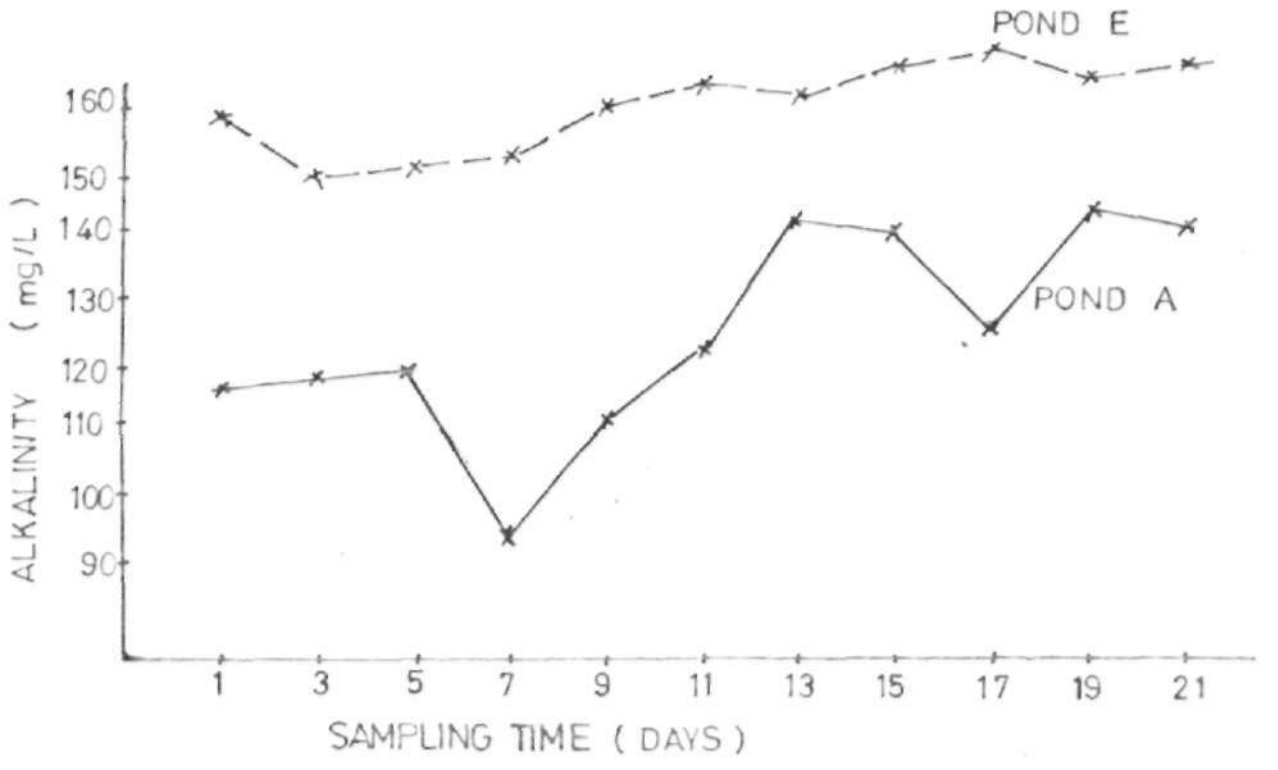


FIG 6-2-1 (e): VARIATIONS EFFLUENT ALKALINITY WITH TIME (TEST 1)

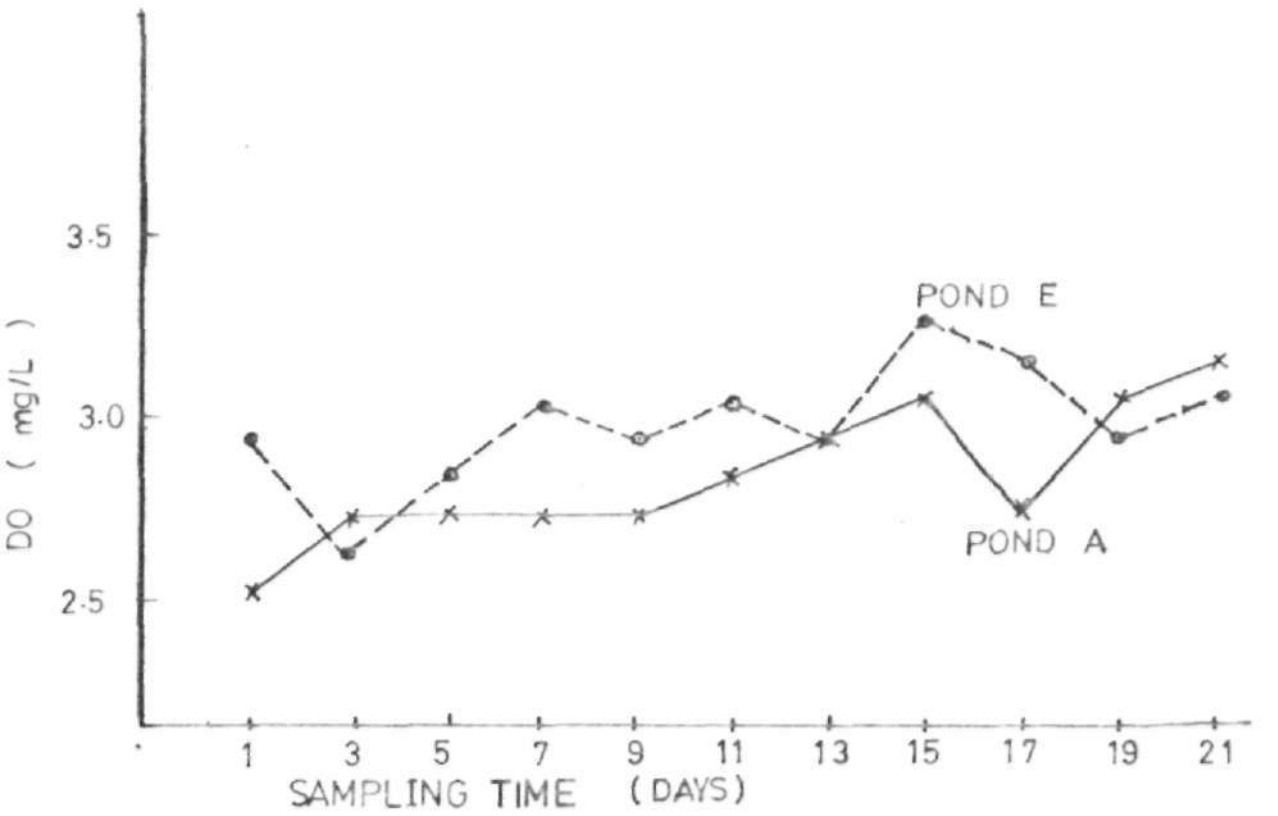


FIG 6-2-1 (f): VARIATIONS IN EFFLUENT DO WITH TIME (TEST 1).

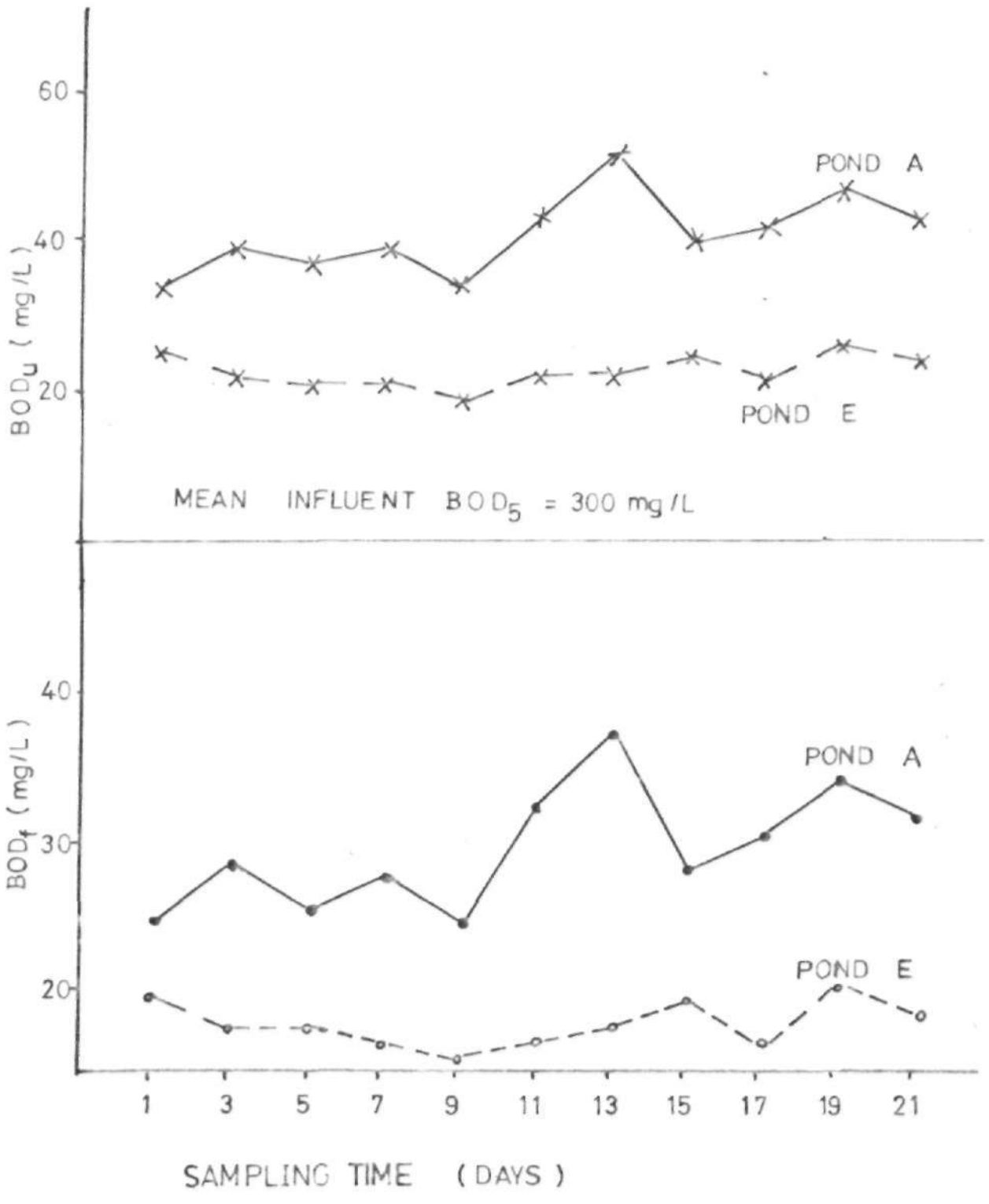


FIG 6-2-2-(a): VARIATIONS IN EFFLUENT BOD$_5$ WITH TIME (TEST 2)

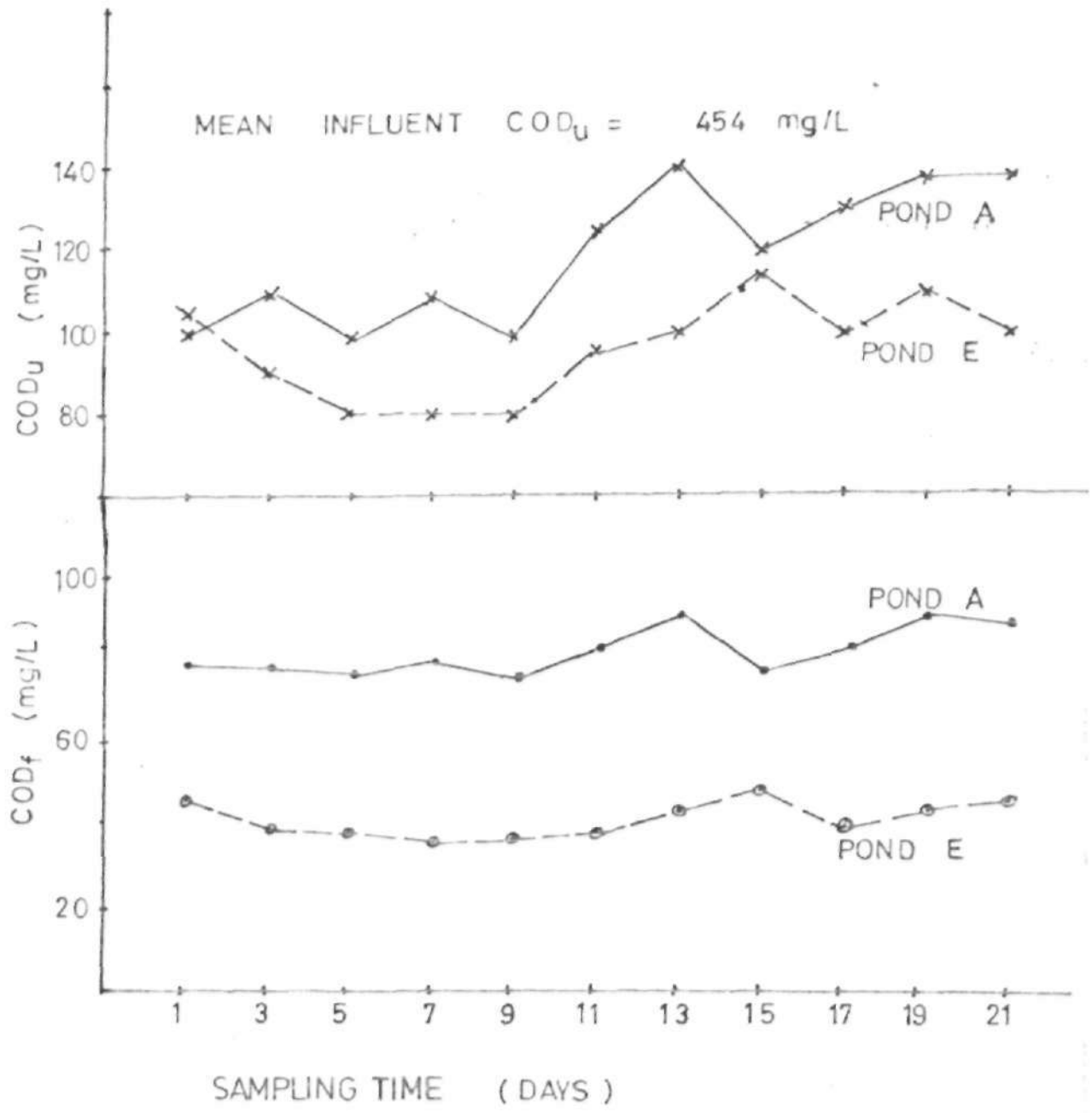


FIG 6-2-2(b): VARIATIONS IN EFFLUENT COD WITH TIME

(TEST 2)

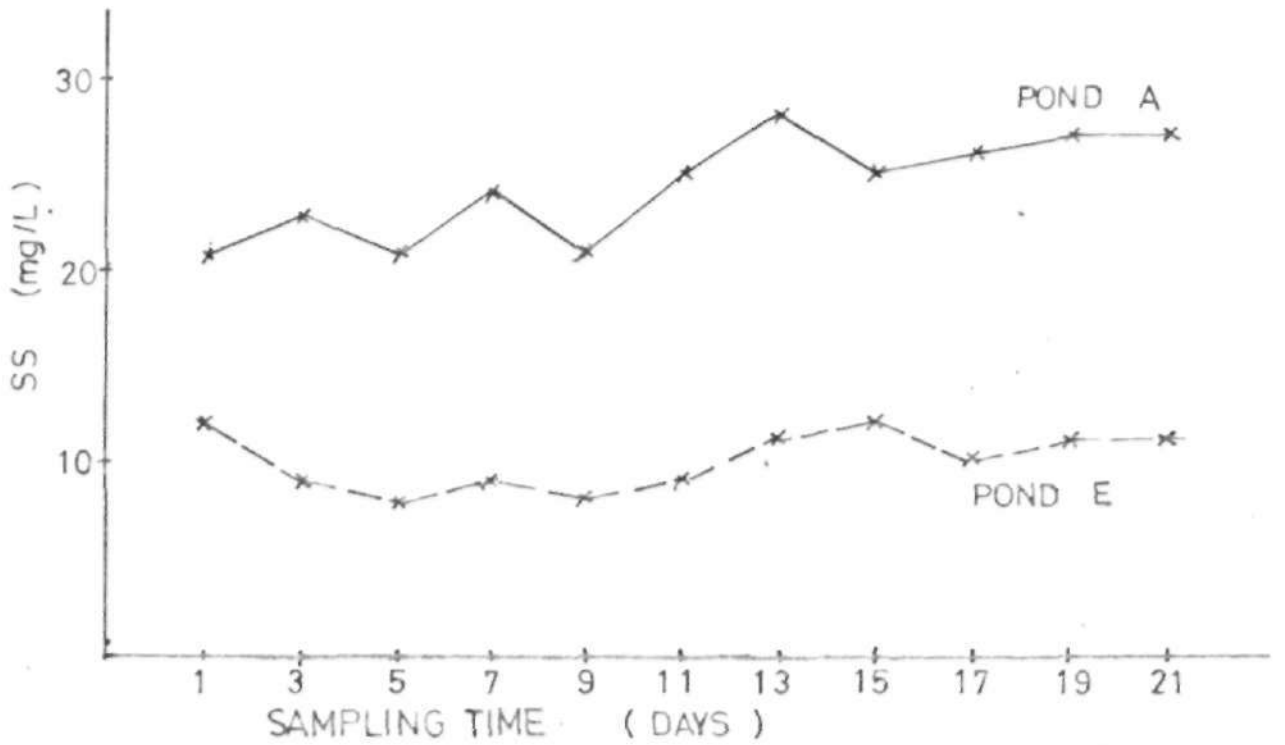


FIG 6-2-2 (C): VARIATIONS IN EFELUENT SUSPENDED SOLIDS SS WITH TIME (TEST 2)

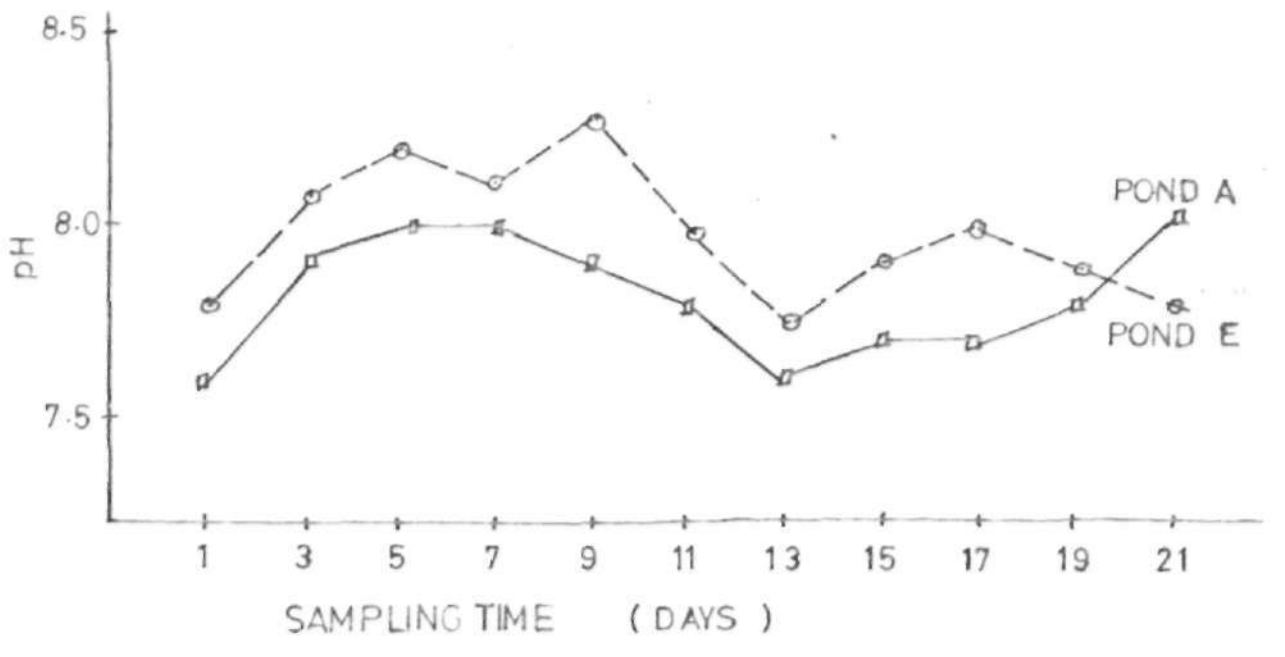


FIG 6-2-2 (d): VARIATIONS IN EFFLUENT pH WITH TIME (TEST 2)

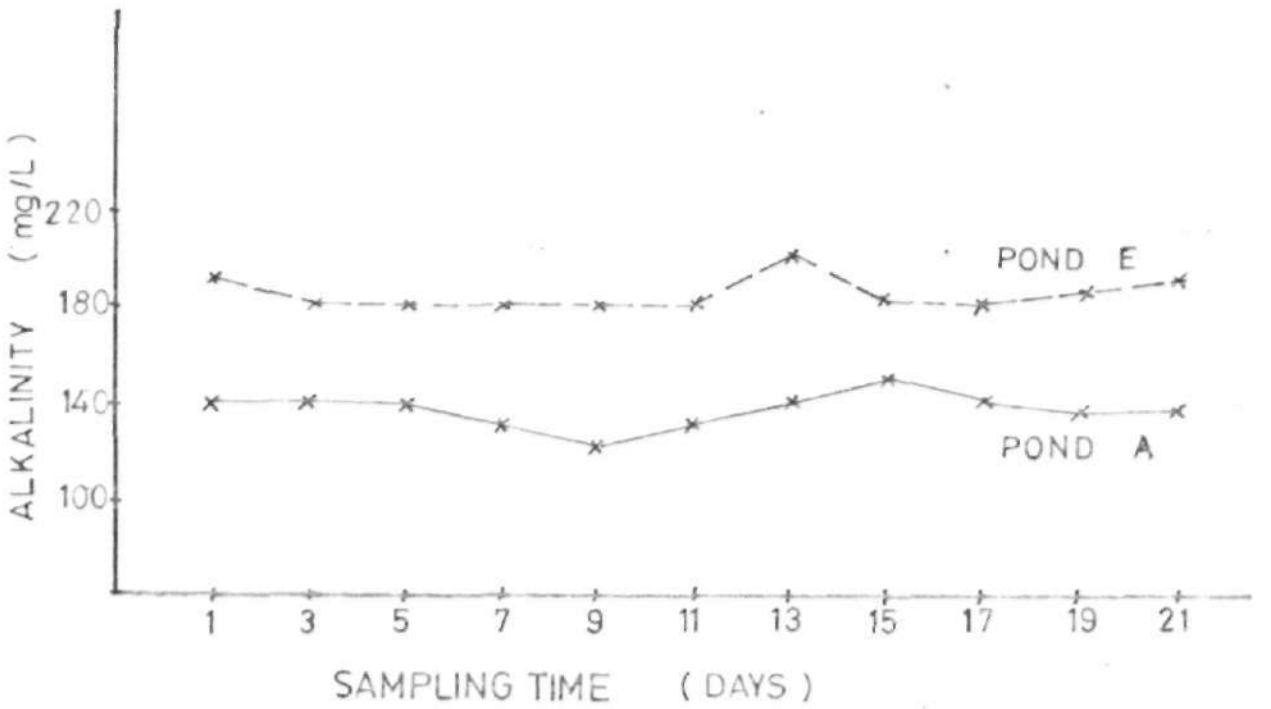


FIG 6-2-2 (e): VARIATIONS IN EFFLUENT ALKALINITY WITH TIME (TEST 2)

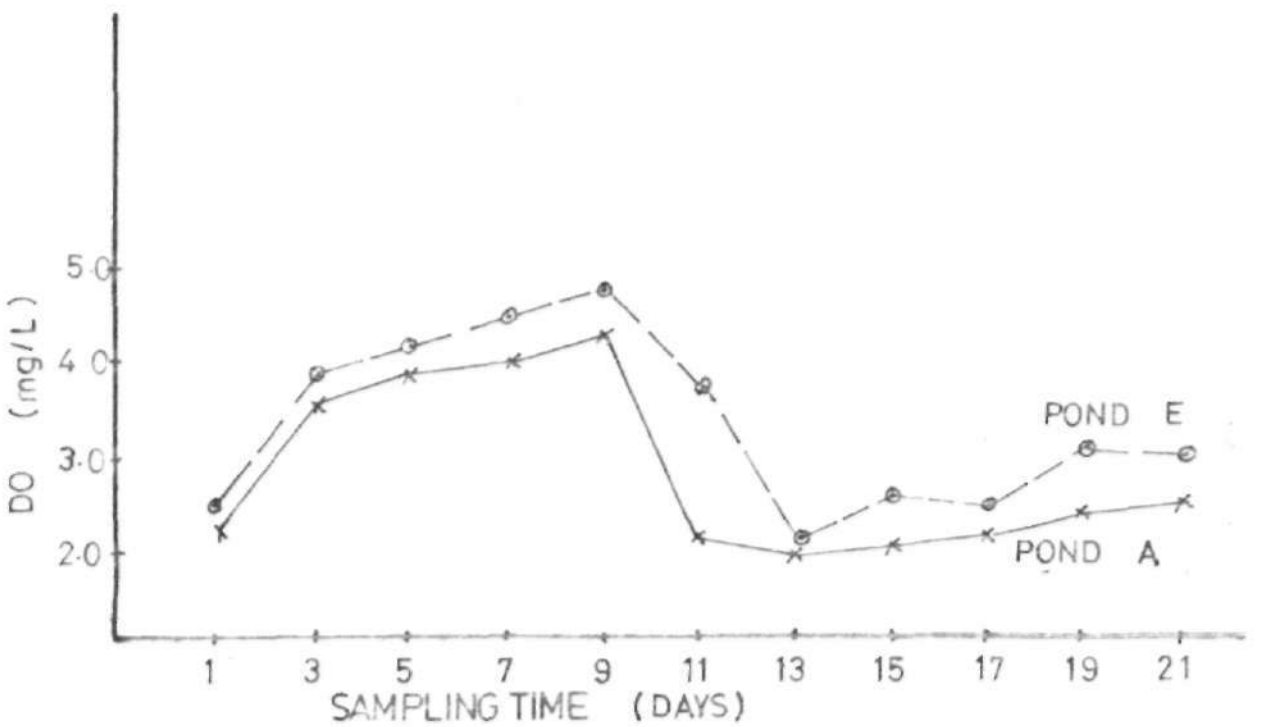


FIG 6-2-2 (f): VARIATIONS IN EFFLUENT DO WITH TIME (TEST 2)

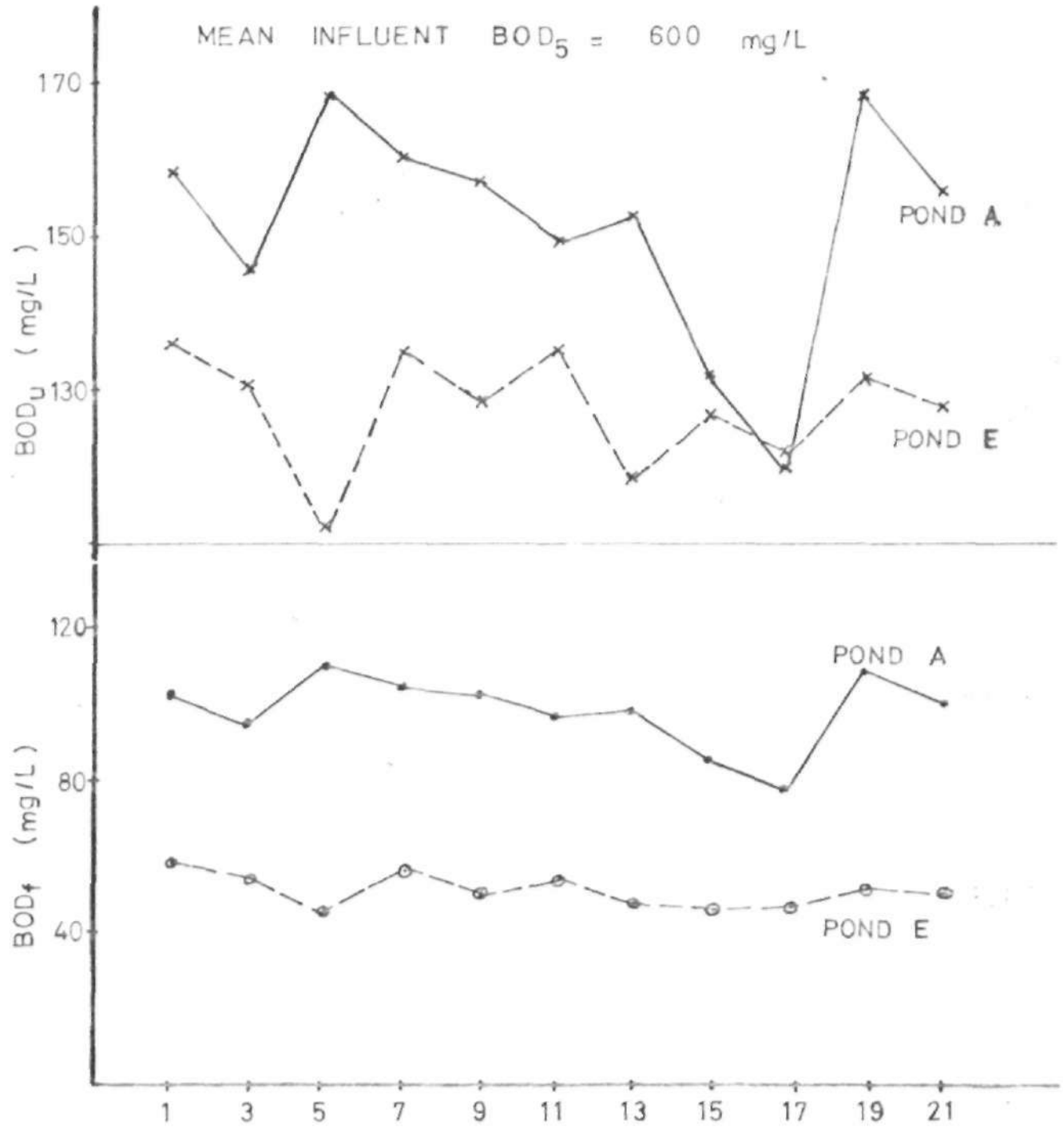


FIG 6-2-3(a): VARIATIONS IN EFFLUENT BOD_5 WITH TIME.
(TEST 3)

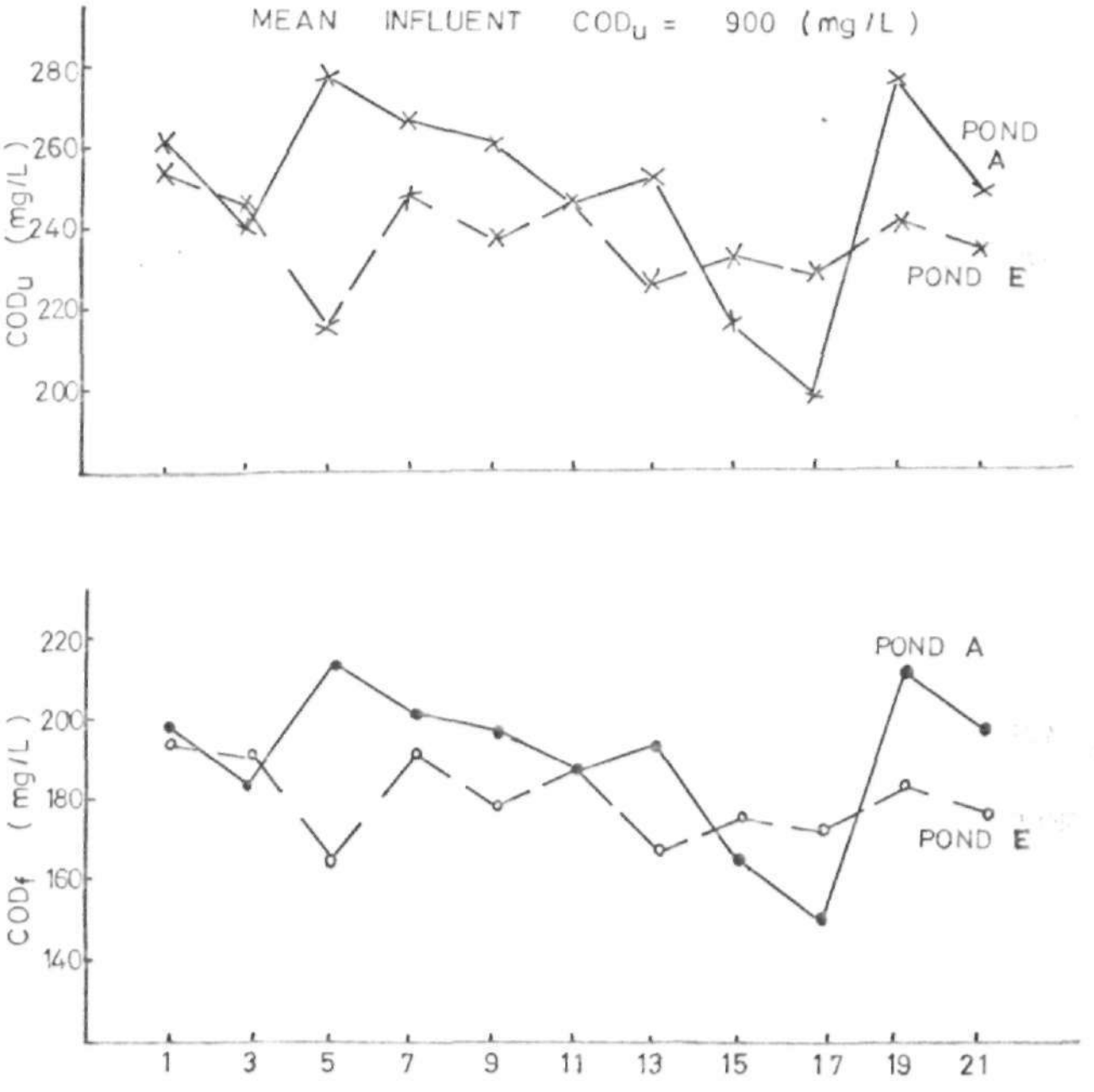


FIG 6-2-3(b): VARIATIONS IN EFFLUENT COD WITH TIME (TEST 3)

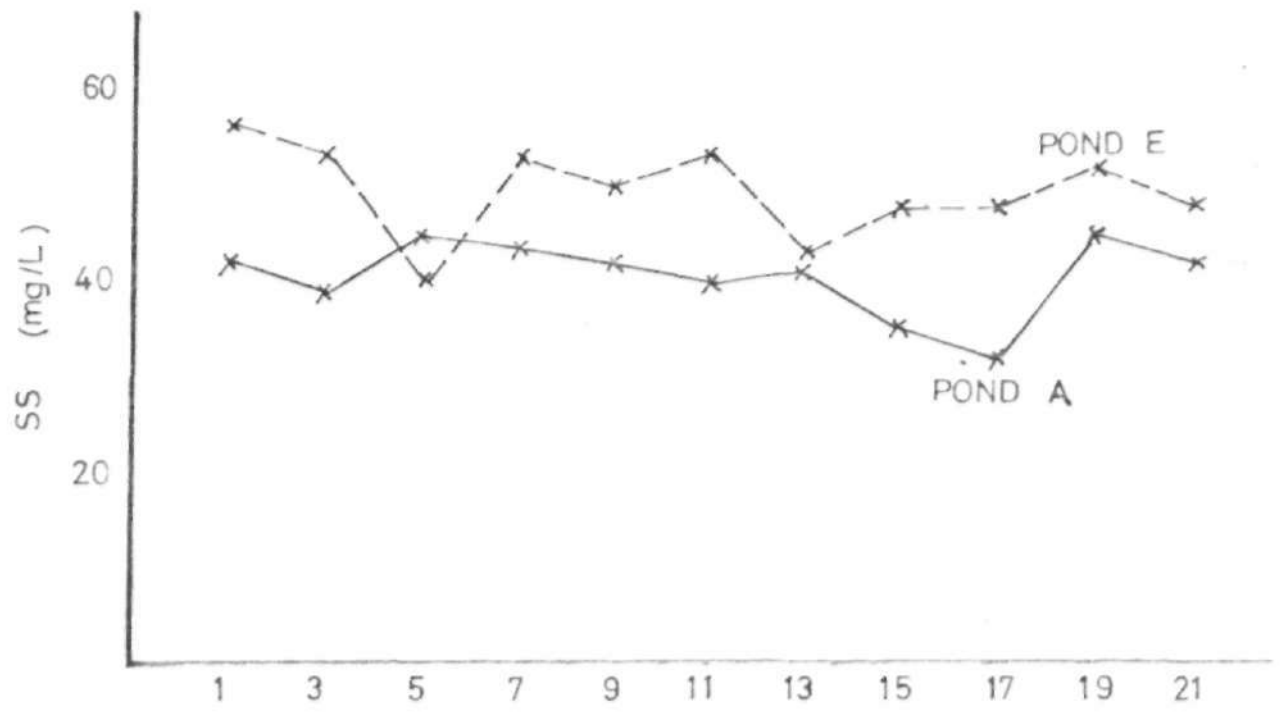


FIG 6-2-3(c): VARIATIONS IN EFFLUENT SUSPENDED SOLID (SS) WITH TIME (TEST 3)

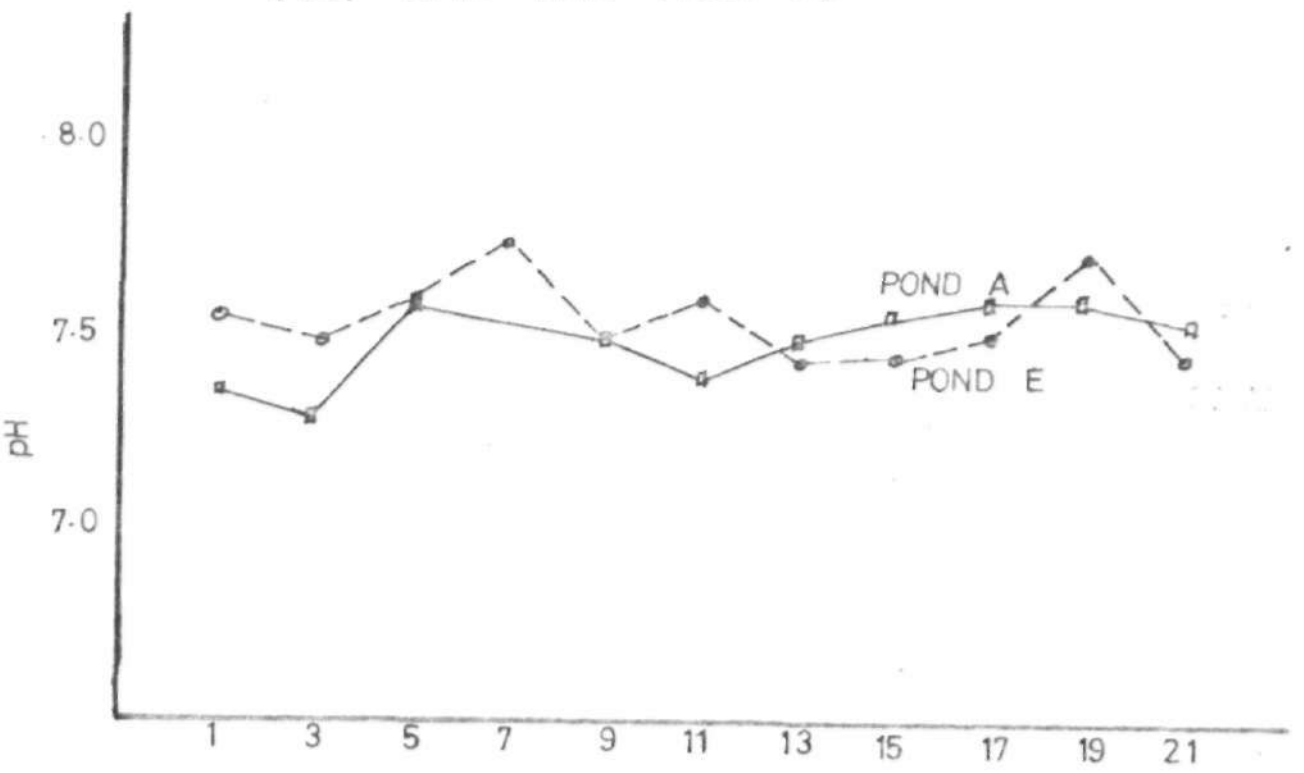


FIG 6-2-3(d): VARIATIONS IN EFFLUENT PH WITH TIME (TEST 3)

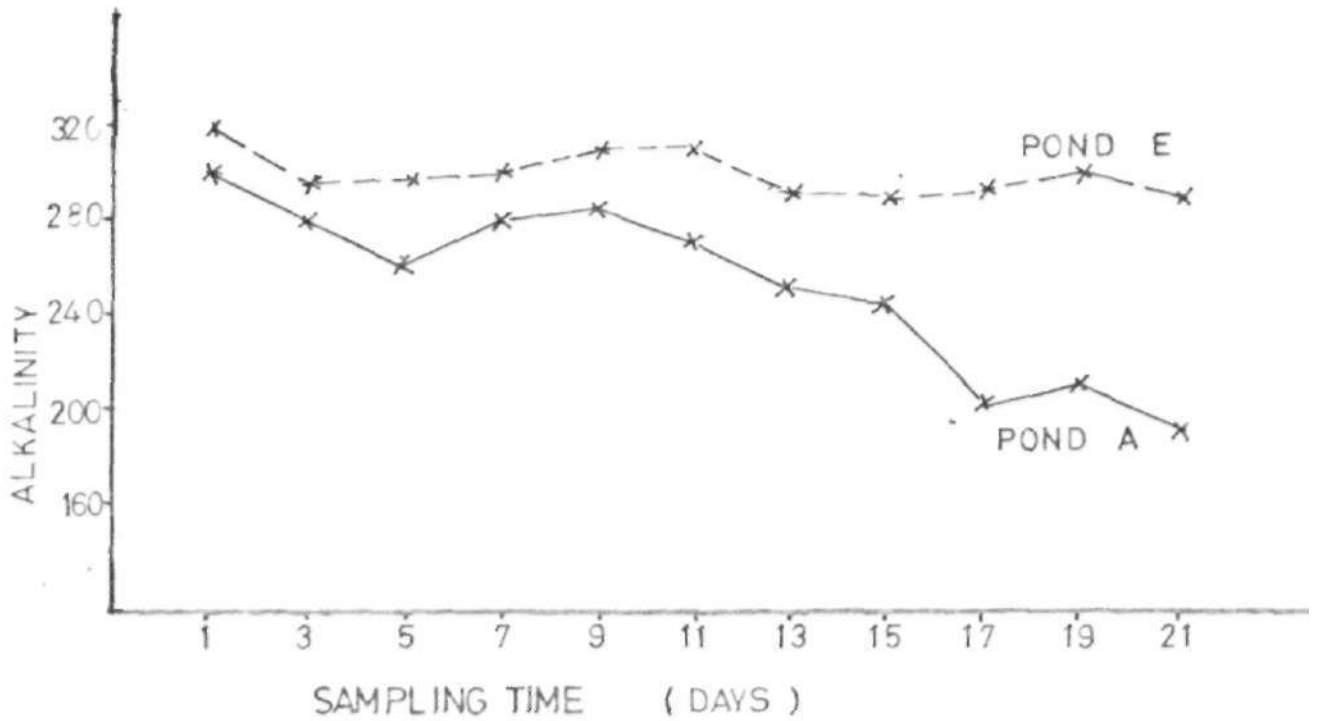


FIG 6-2-3(e): VARIATIONS IN EFFLUENT ALKALINITY WITH TIME (TEST 3)

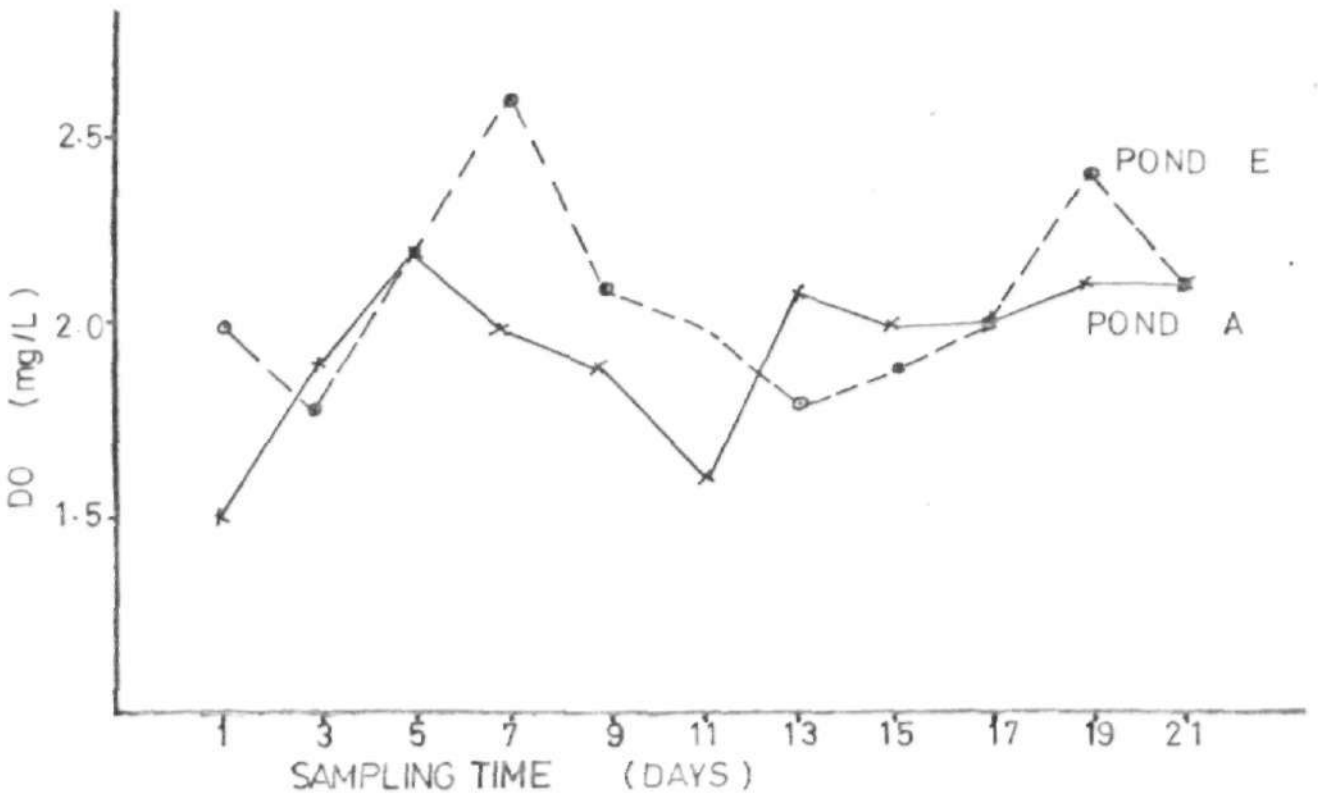


FIG 6-2-3(f): VARIATIONS IN EFFLUENT DO WITH TIME (TEST 3)

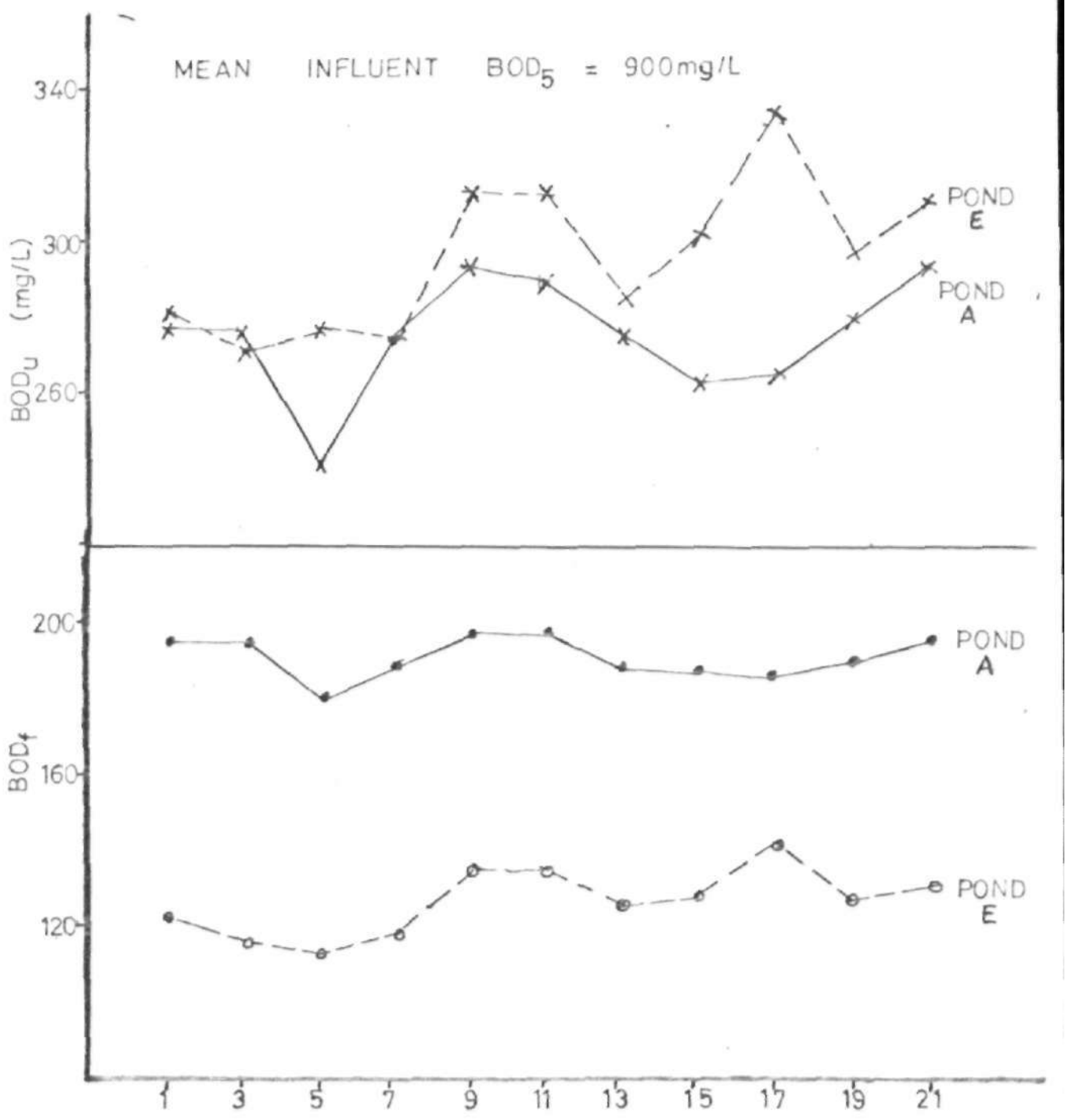


FIG 6-2-4(a): VARIATIONS IN EFFLUENT BOD_5 WITH TIME

(TEST 4)

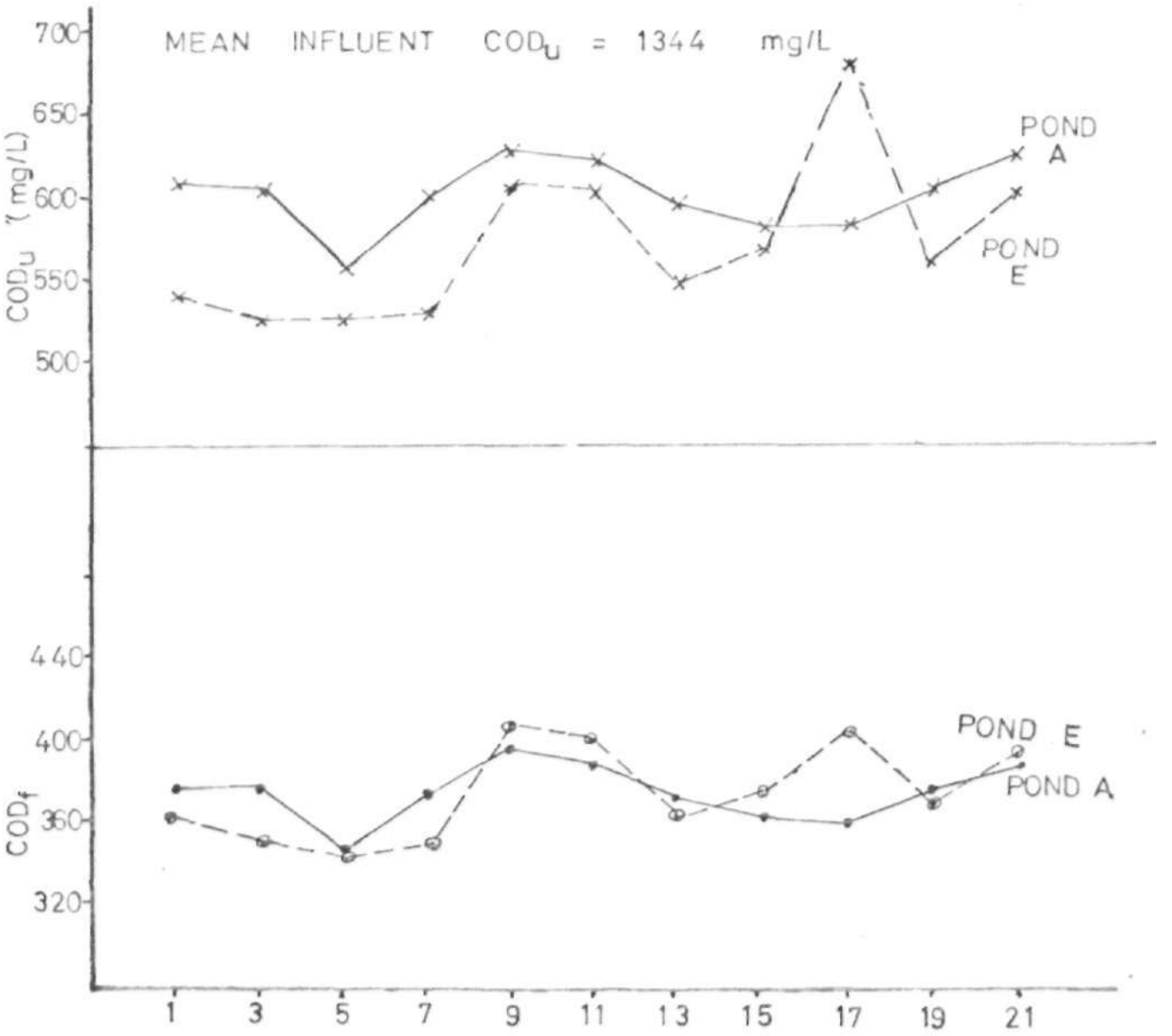


FIG 6-2-4(b): VARIATIONS IN EFFLUENT COD WITH TIME

(TEST 4)

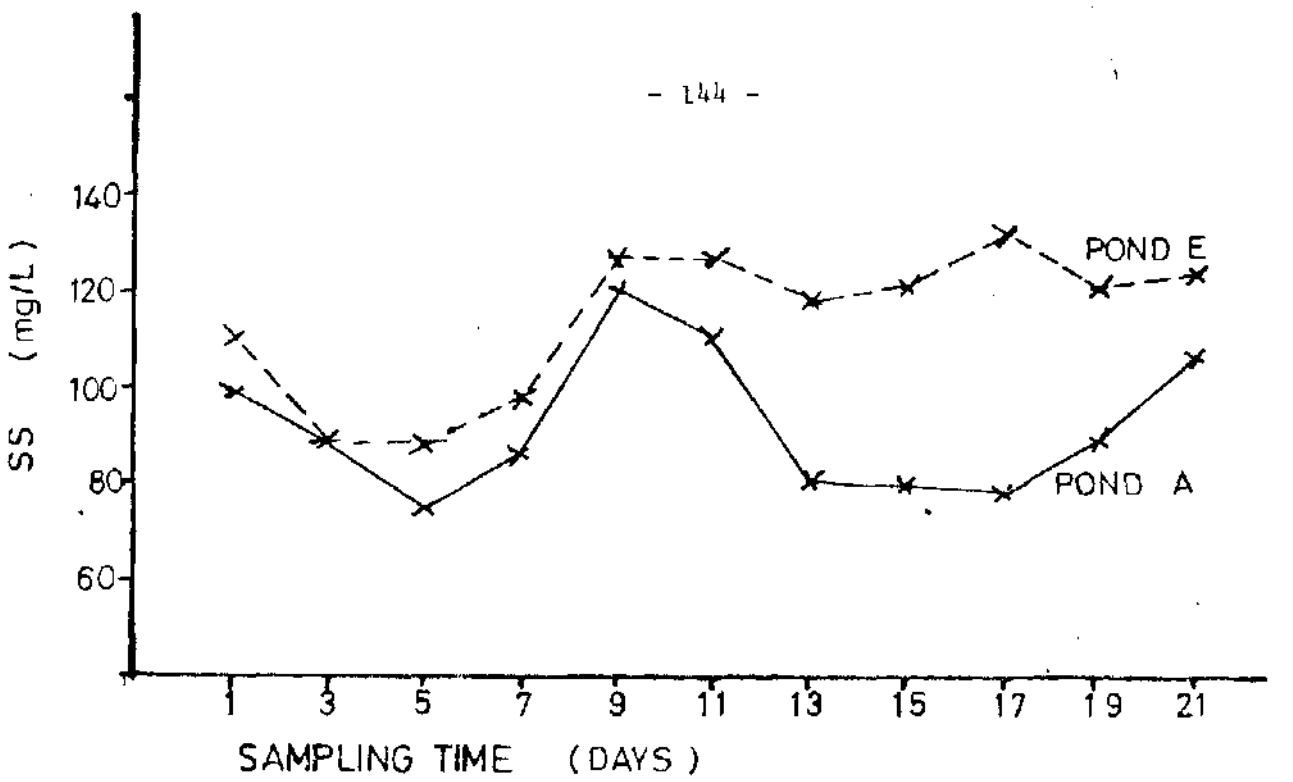


FIG 6-2-4 (c): VARIATIONS IN EFFLUENT SUSPENDED SOLIDS SS WITH TIME (TEST 4)

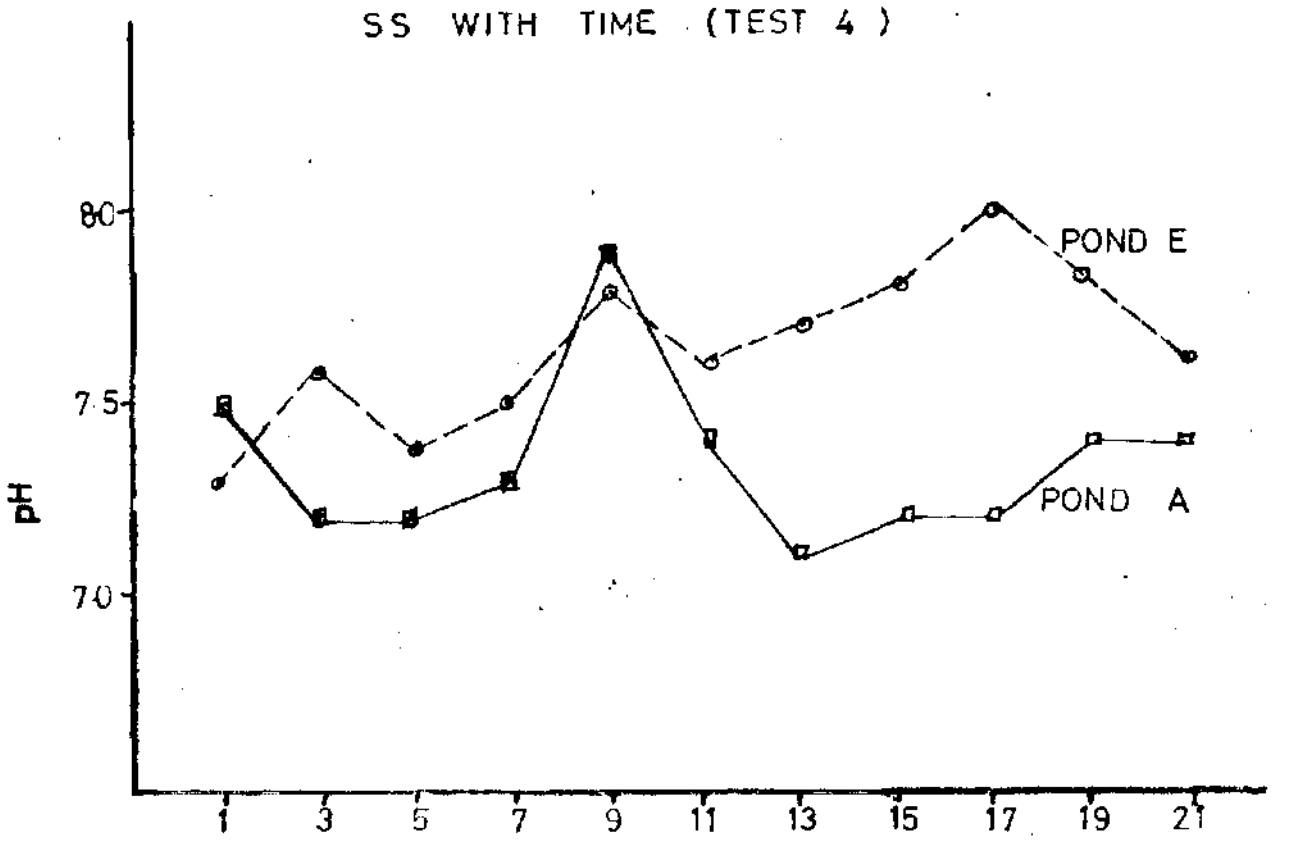


FIG: 6-2-4 (d) VARIATIONS IN EFFLUENT pH WITH TIME (TEST 4)

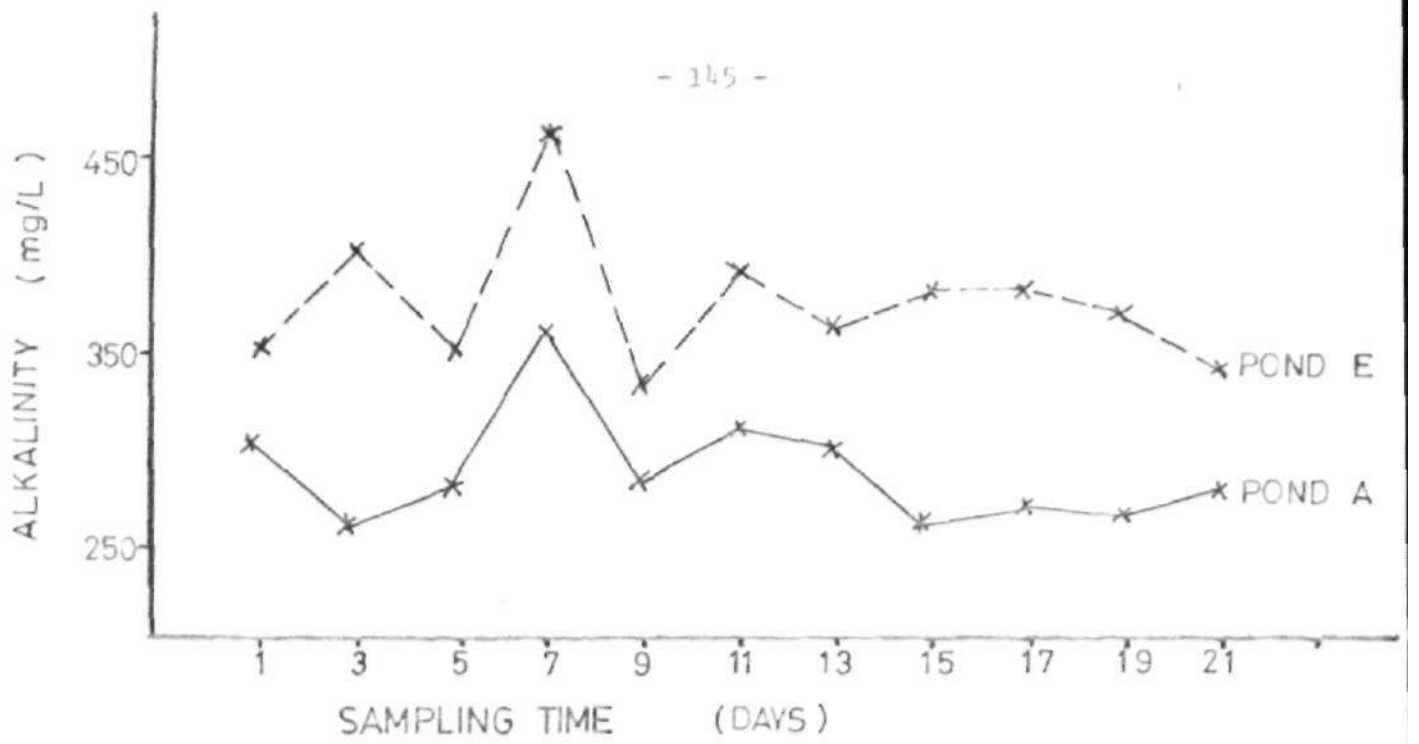


FIG 6-2-4 (e): VARIATIONS IN EFFLUENT ALKALINITY WITH TIME (TEST 4)

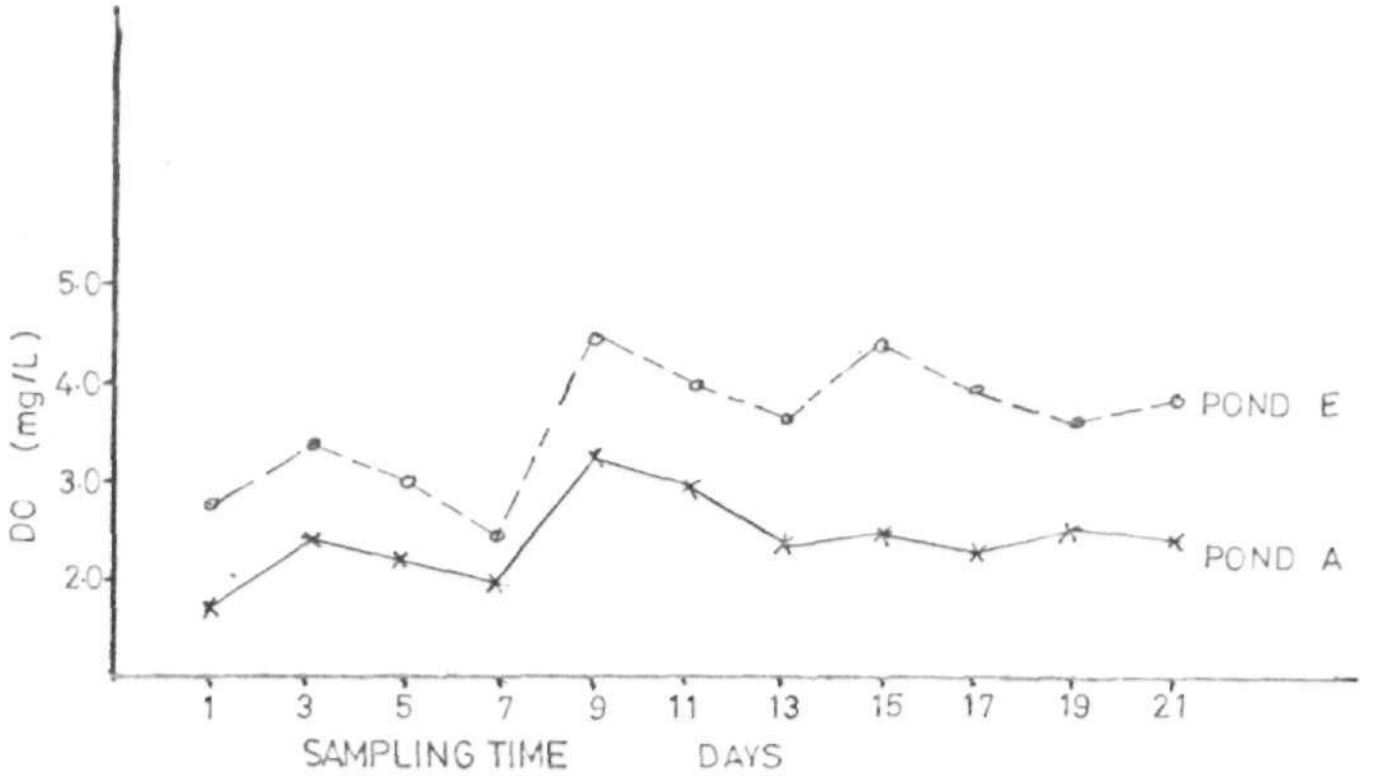


FIG 6-2-4 (f): VARIATIONS IN EFFLUENT DO WITH TIME (TEST 4)

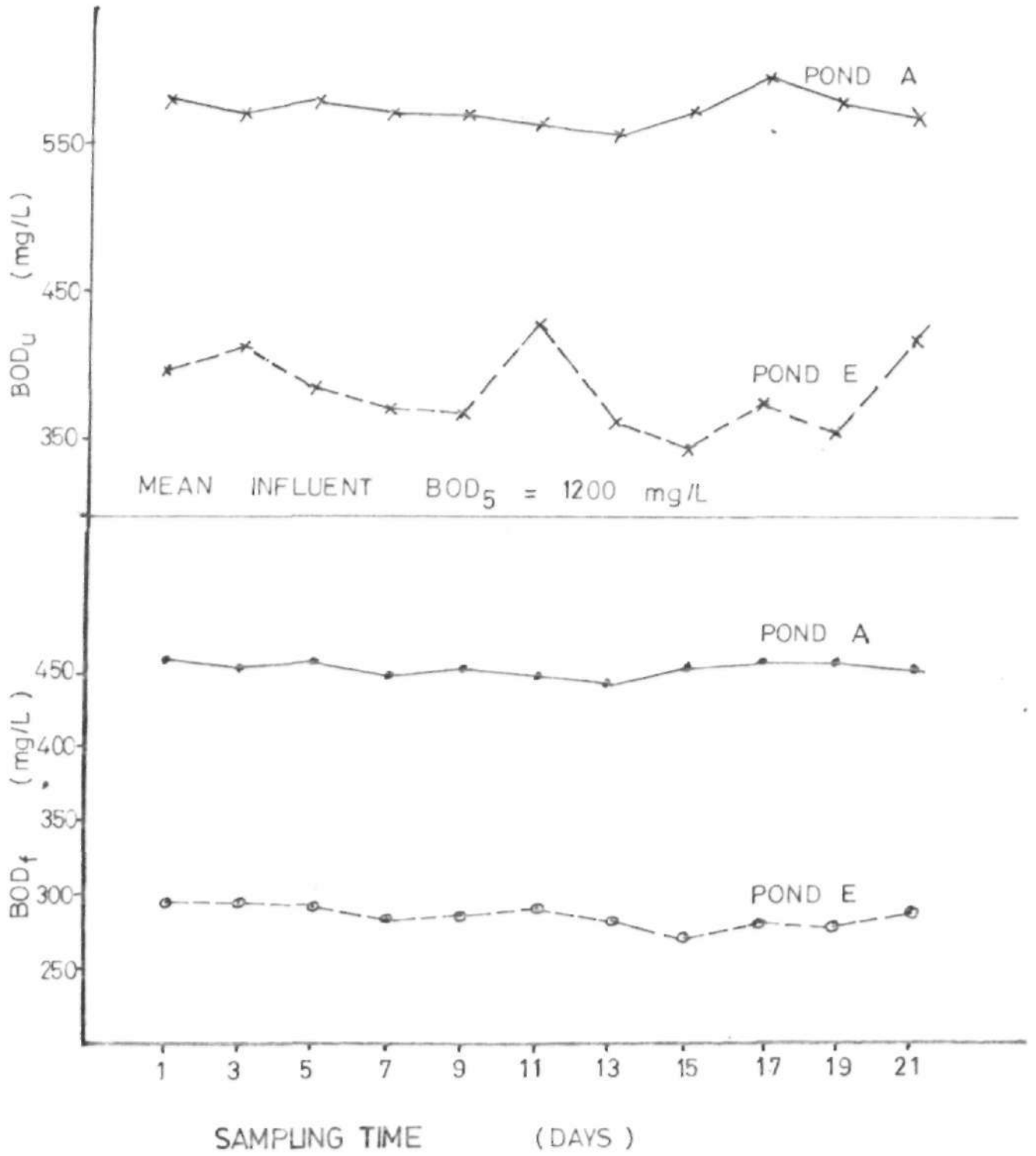


FIG 6-2-5 (a): VARIATIONS IN EFFLUENT BOD_5 WITH TIME (TEST 5).

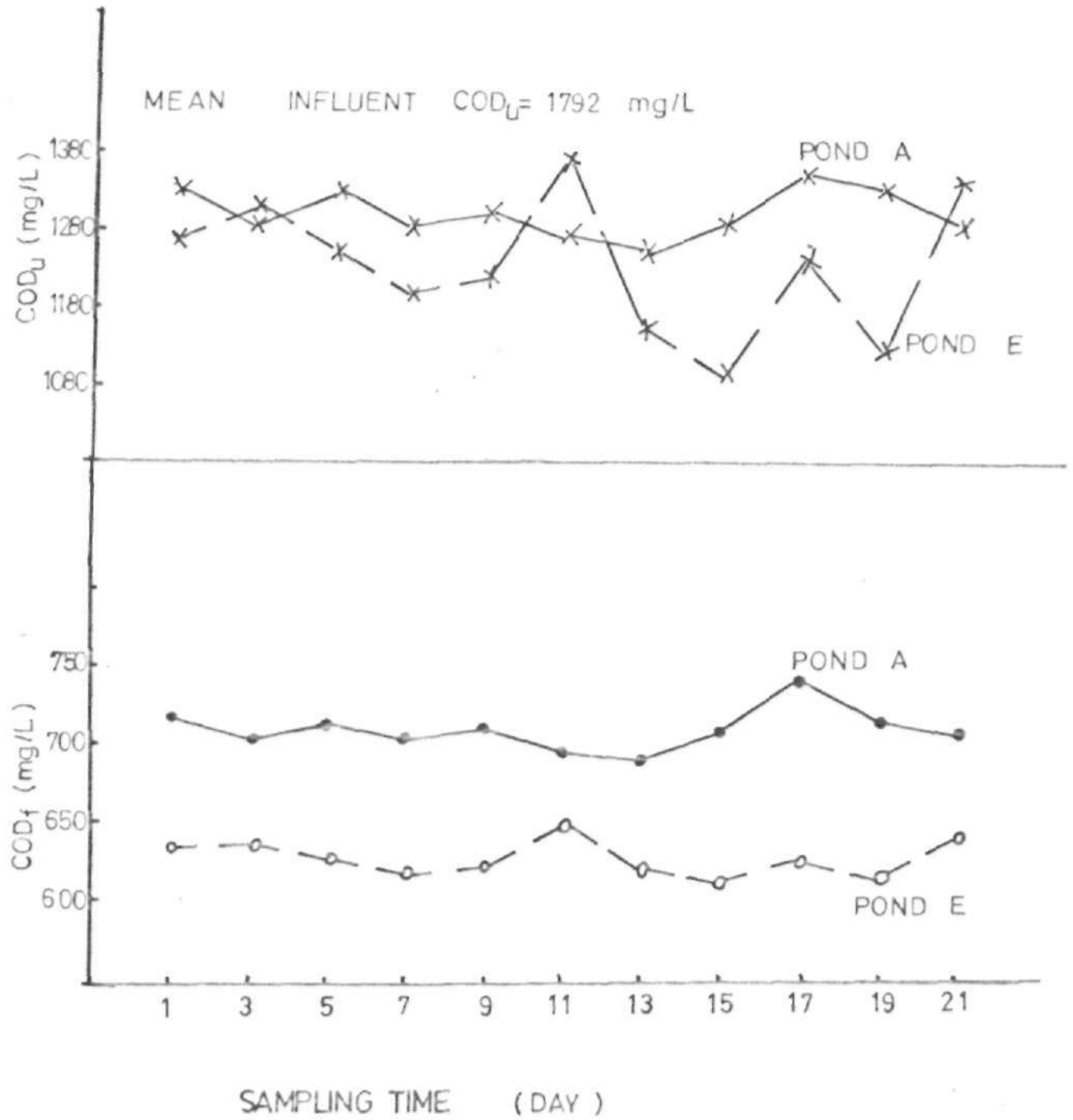


FIG 6-2-5(b): VARIATIONS IN EFFLUENT COD WITH TIME

(TEST 5).

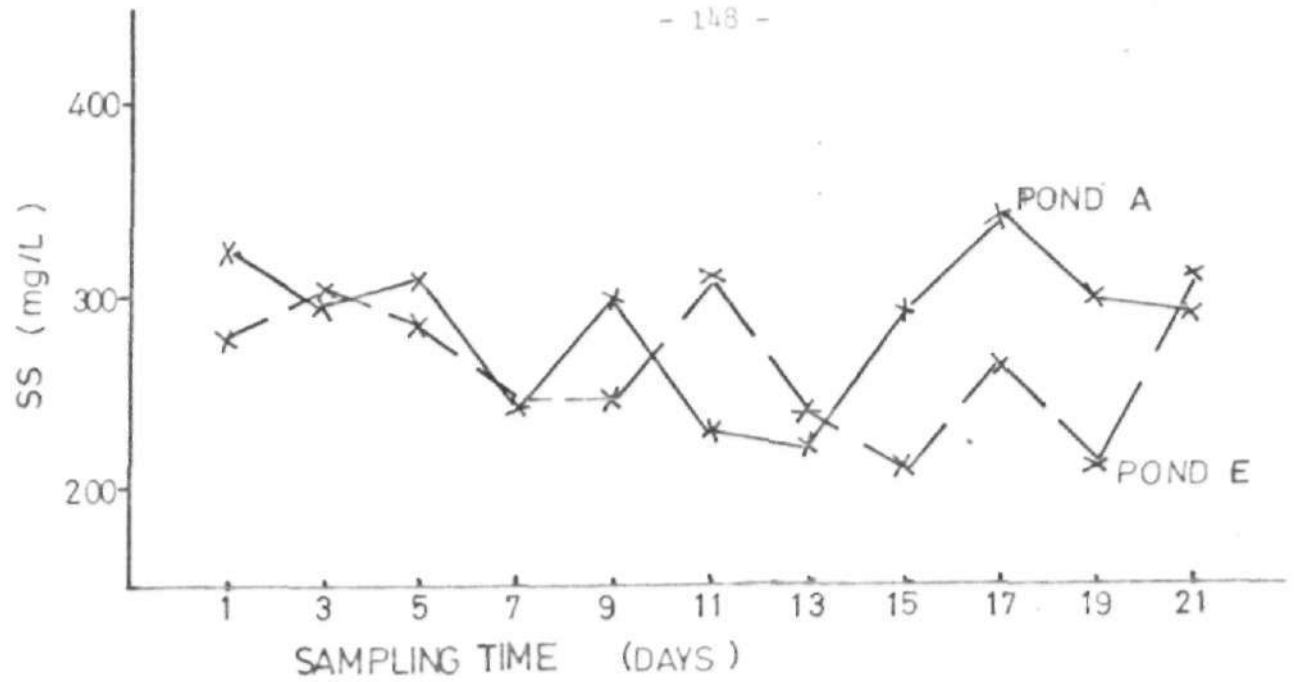


FIG 6-2-5(c): VARIATIONS IN EFFLUENT SUSPENDED SOLIDS (SS) WITH TIME (TEST 5)

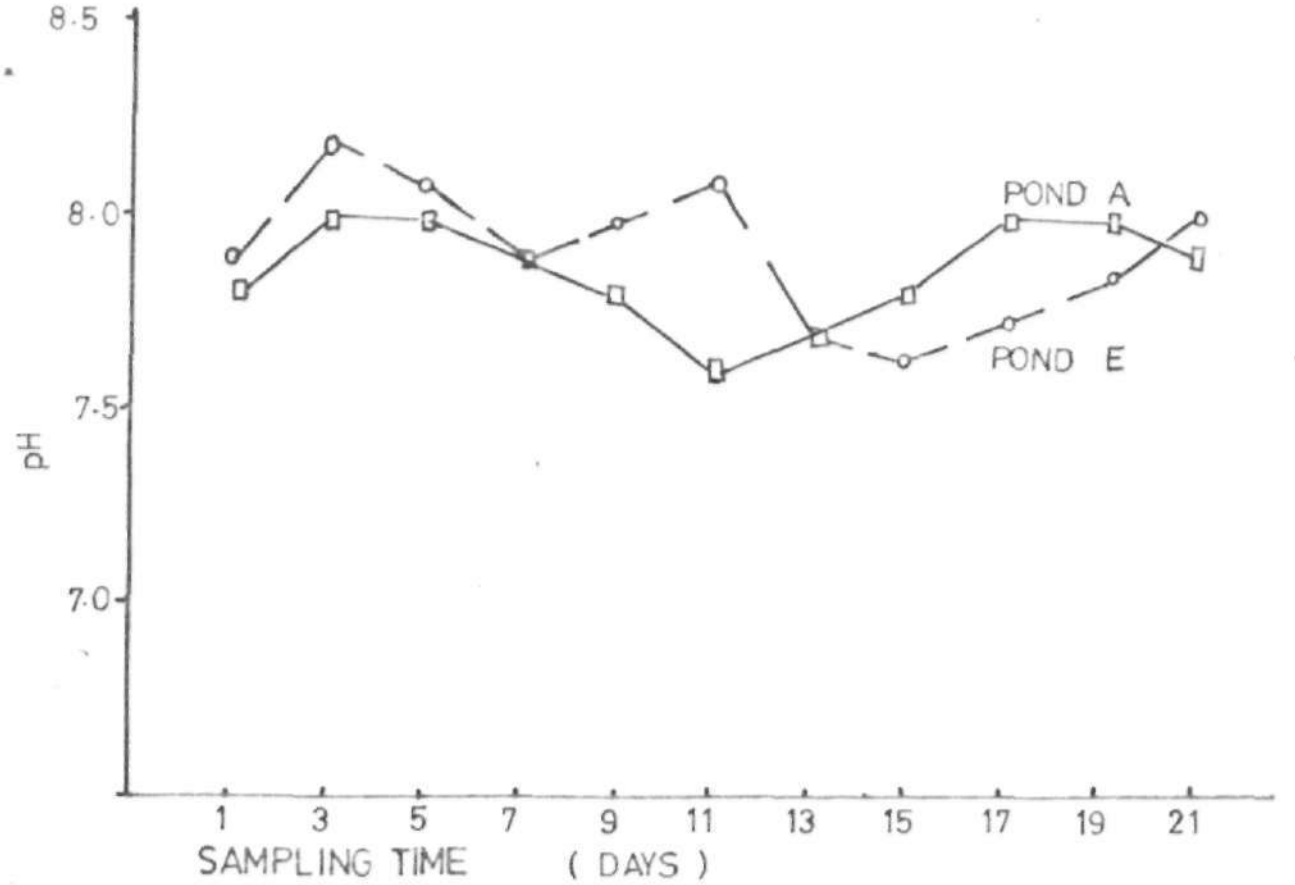


FIG 6-2-5(d): VARIATIONS IN EFFLUENT pH WITH TIME (TEST 5)

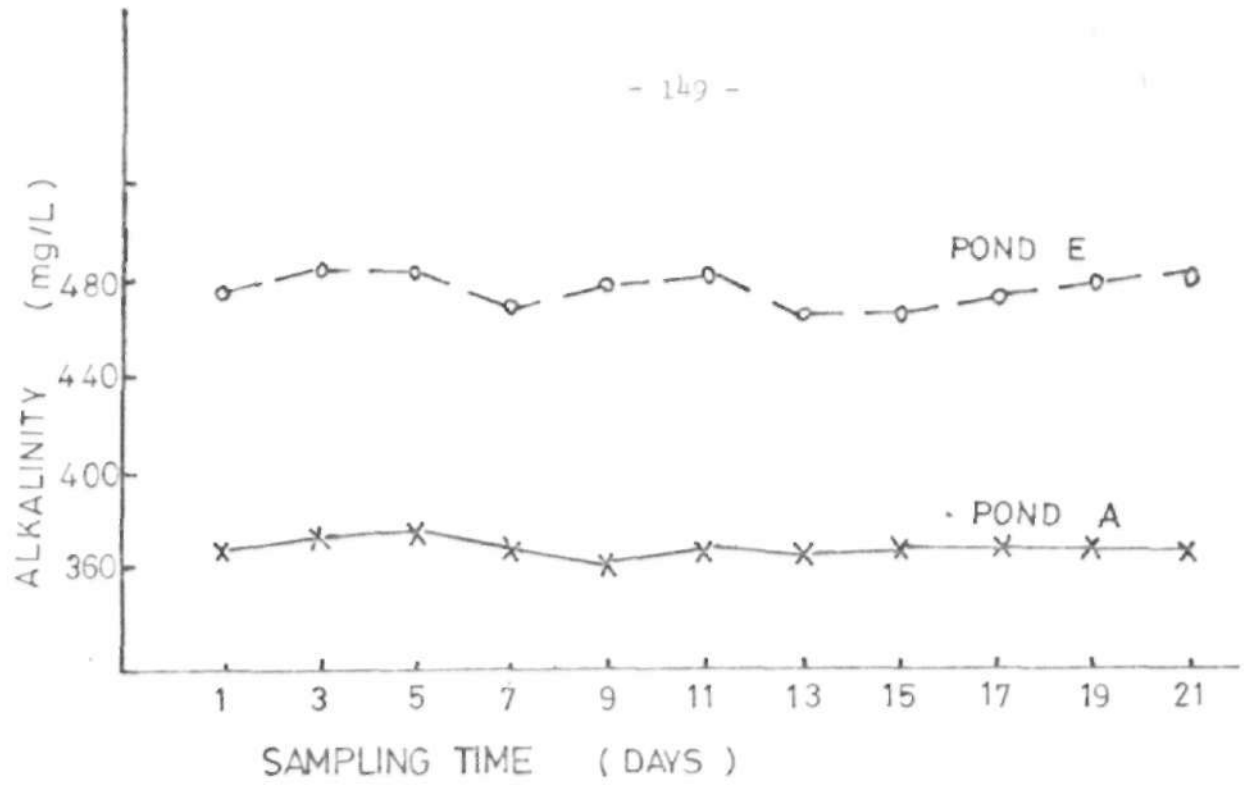


FIG 6-2-5 (e): VARIATIONS IN EFFLUENTS ALKALINITY WITH TIME (TEST 5)

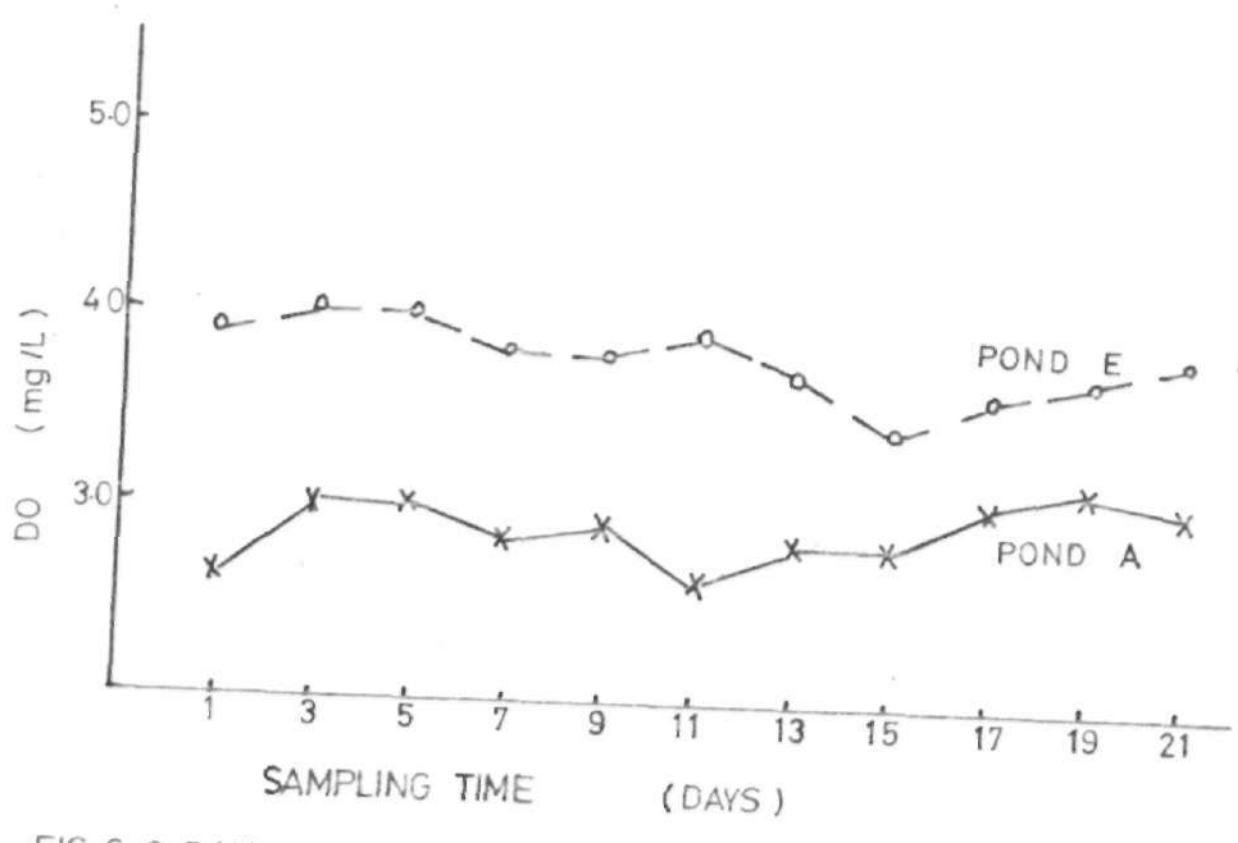


FIG 6-2-5 (f): VARIATIONS IN EFFLUENT DO WITH TIME (TEST 5).

TABLE 6-2-1 (a): SUMMARY OF INFLUENT AND EFFLUENT CHARACTERISTICS FOR ALL THE PONDS (33 kg BOD_5 /ha-day)

PARAMETERS	INFLUENTS		EFFLUENTS								AVERAGE PERCENTAGE REMOVAL				K VALUES (Base e)			
			PLAIN POND				15 BAFFLE POND				PLAIN POND		15 BAFFLE POND		PLAIN POND		15 BAFFLE POND	
			UNFILTERED		FILTERED		UNFILTERED		FILTERED									
	RANGE	AVE-RAGE	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	UNFILTERED	FILTERED	UNFILTERED	FILTERED	UNF.	FIL.	UNF.	FIL.
BOD_5 (mg/L)	184-210	200	11-18	15.00	10 - 14	12.00	8 - 14	10.00	5 - 8	6.0	92.00	94.00	95.00	97.00	0.422*	0.457	0.162	0.189
BOD_5 (Kg/ha-day)	31-35.0	33.0	2.0-3.0	2.40	1.7-2.3	2.0	1.3-2.3	1.7	0.8-1.3	1.0								
COD (mg/L)	275-313	300	16 - 25	21.00	12 - 18	15.00	17 - 25	20.00	11 - 16	13.00	93.00	95.00	93.00	96.00	-	-	-	-
COD (Kg/ha-day)	46.0 - 52.0	50.0	3.0-4.2	3.5	2.0-3.0	2.5	2.8-4.2	3.3	1.8-2.7	2.2								
SS (mg/l)	79-89	83.00	19-28	23.00	-	-	9-13	9.00	-	-	76.00	-	90.00	-	-	-	-	-
pH	7.3-8.6	8.00	7.8-8.2	8.0	-	-	7.8-8.4	8.15	-	-	-	-	--	-	-	-	-	-
ALKALINITY (mg/L as $CaCO_3$)	8.4-107	100.00	94-142	125.00	-	-	152-170	162.00	-	-	-	-	-	-	-	-	-	-
DO (mg/l)	4.7-8.8	6.68	2.6-3.2	2.9	-	-	2.7-3.3	3.04	-	-	-	-	-	-	-	-	-	-

TABLE 6-2-1 (b): SUMMARY OF INFLUENT AND EFFLUENT CHARACTERISTICS FOR ALL THE PONDS (50 Kg BOD_5 /ha-day)

PARAMETERS	INFLUENTS		EFFLUENTS								AVERAGE PERCENTAGE REMOVAL				K VALUES (Base e)			
			PLAIN POND				15 BAFFLE POND				PLAIN POND		15 BAFFLE POND		PLAIN POND		15 BAFFLE POND	
			UNFILTERED		FILTERED		UNFILTERED		FILTERED									
	RANGE	AVE-RAGE	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	UNFILTERED	FILTERED	UNFILTERED	FILTERED	UNF.	FIL.	UNF.	FIL.
BOD_5 (mg/L)	290-300	300	34-52	41	25-38	30	19 - 26	23.00	16-21	18	86.00	90.00	92.00	94.00	0.305	0.352	0.139	0.152
BOD_5 (Kg/ha-day)	48.0 - 53.0	50.00	5.7-8.7	7.0	4.0-6.0	5.0	3.0-4.0	4.0	3.0-4.0	3.0								
COD (mg/L)	433-480	4.5-4.0	98-141	119	78-94	85	80-115	96	36-52	43	74.00	81.00	79.00	90.00	-	-	-	-
COD (Kg/ha-day)	72.0 - 80.0	76.0	16-24	120	13.0-16	150	13.0-19.0	16.0	6.0-9.0	7.0								
SS (mg/ℓ)	85-135	108.00	21-28	24.00	-	-	8 - 12	10.00	-	-	77.00	-	91.00	-	-	--	-	-
pH	6.1-8.1	6.65	7.6-8.0	7.82	-	-	7.7-8.3	7.98	-	-	-	-	-	-	-	-	-	-
ALKALINITY (mg/L as $CaCO_3$)	90-160	116	120-150	137.00	-	-	180-200	184.0	-	-	-	-	-	-	-	-	-	-
DO (mg/ℓ)	0.7-7.8	3.09	2.0-4.3	2.86	-	-	2.2-4.8	3.32	-	-	-	-	-	-	-	-	-	-

TABLE 6-2-1 (e): SUMMARY OF INFLUENT AND EFFLUENT CHARACTERISTICS FOR ALL THE PONDS (100 kg BOD_5/ha-day)

PARAMETERS	INFLUENTS		E F F L U E N T S								AVERAGE PERCENTAGE REMOVAL				K VALUES (Base e)			
			PLAIN POND				15 BAFFLE POND				PLAIN POND		15 BAFFLE POND		PLAIN POND		15 BAFFLE POND	
			UNFILTERED		FILTERED		UNFILTERED		FILTERED									
	RANGE	AVE-RAGE	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	UNFILTERED	FILTERED	UNFILTERED	FILTERED	UNF.	FIL.	UNF.	FIL.
BOD_5 (mg/L)	600-615	600	119-169	151.00	77-110	98.00	111-136	127.00	44-58	51.00	75.00	84.00	79.00	92.00	0.205	0.294	0.086	0.139
BOD_5 (Kg/ha-day)	98.5 - 102.7	100.0	20.0 - 28.2	25.00	13.0 - 18.4	16.4	18.5 - 22.7	21.7	7.4-9.7	8.5								
COD (mg/L)	880-918	900.00	198-278	251.00	151-214	192.0	216-254	237.0	164 - 124	181.00	72.00	79.00	74.00	80.00	-	-	-	-
COD (Kg/ha-day)	147-153	150.0	33.0 - 46.4	42.0	25.2 - 35.7	32.0	36.0 - 42.4	39.6	27.4 - 32.4	30.2								
SS (mg/l)	175-222	198.36	32-45	41.00	-	-	40-56	50.00	-	-	80.00	-	75.00	-	-	-	-	-
pH	6.7-8.8	7.85	6.6-7.2	7.00	-	-	6.9-7.5	7.11	-	-	-	-	-	-	-	-	-	-
ALKALINITY (mg/L as $CaCO_3$)	108-140	124.00	190-300	252.00	-	-	290-320	300.00	-	-	-	-	-	-	-	-	-	-
DO (mg/l)	1.4-8.2	6.35	1.5-2.2	1.95	-	-	1.8-2.6	2.08	-	-	-	-	-	-	-	-	-	-

TABLE 6-2-1 (d): SUMMARY OF INFLUENT AND EFFLUENT CHARACTERISTICS FOR ALL THE PONDS (150 kg BOD_5/ha-day)

PARAMETERS	INFLUENTS		EFFLUENTS								AVERAGE PERCENTAGE REMOVAL				K VALUES (Base e)			
			PLAIN POND				15 BAFFLE POND				PLAIN POND		15 BAFFLE POND		PLAIN POND		15 BAFFLE POND	
			UNFILTERED		FILTERED		UNFILTERED		FILTERED									
	RANGE	AVE-RAGE	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	UNFILTERED	FILTERED	UNFILTERED	FILTERED	UNF.	FIL.	UNF.	FIL.
BOD_5 (mg/L)	893-910	900	240-291	273.00	180-197	191	270-331	294	113-142	127	70.00	79.00	67.00	86.00	0.176	0.217	0.061	0.106
BOD_5 (Kg/ha-day)	149.0 - 152	150	40.0 - 48.6	46.0	30.0 - 33.0	32.0	45.0 - 55.3	49.0	19.0 - 23.7	21.0								
COD (mg/L)	1330 - 1358	1344.36	557-628	600	348-393	375	526-680	572.00	346-421	376.00	55.00	72.00	58.00	72.00	-	-	-	-
COD (Kg/ha-day)	222.0 - 227	225.0	93.0-105	100.00	58.0-65.6	62.0	87.8 - 113.6	96.0	57.8 - 70.3	63.0								
SS (mg/l)	360-440	395	75-120	92	-	-	88-131	113.00	-	-	77.00	-	71.00	-	-	-	-	-
pH	5.1-8.6	7.51	7.1-7.9	7.35	-	-	7.3-8.0	7.65	-	-	-	-	-	-	-	-	-	-
ALKALINITY (mg/L as $CaCO_3$)	145-210	180	260-360	288	-	-	330-460	374.00	-	-	-	-	-	-	-	-	-	-
DO (mg/l)	0.9-8.4	4.74	1.8-3.3	2.45	-	-	2.5-46	3.69	-	-	-	-	-	-	-	-	-	-

TABLE 6-2-1 (e): SUMMARY OF INFLUENT AND EFFLUENT CHARACTERISTICS FOR ALL THE PONDS (200 kg BOD_5/ha-day)

PARAMETERS	INFLUENTS		EFFLUENTS								AVERAGE PERCENTAGE REMOVAL				K VALUES (Base e)			
			PLAIN POND				15 BAFFLE POND				PLAIN POND		15 BAFFLE POND		PLAIN POND		15 BAFFLE POND	
	RANGE	AVERAGE	UNFILTERED		FILTERED		UNFILTERED		FILTERED		UNFILTERED	FILTERED	UNFILTERED	FILTERED				
			RANGE	AVE.	RANGE	AVE.	RANGE	AVE.	RANGE	AVE.					UNF.	FIL.	UNF.	FIL.
BOD_5 (mg/L)	1195 - 1202	1200	558-590	574	440-455	450	349-433	385.00	279 - 299	289.00	52.00	63.00	68.00	76.00	0.106	0.129	0.063	0.081
BOD_5 (Kg/ha-day)	199.6 - 201.7	200.4	91.2-100	96.0	73.5-76.0	75.2	58.3 - 72.3	64.3	46.6 - 50	48.3								
COD (mg/L)	1784 - 1803	1792	1256 - 1350	1303.00	690-743	710.00	1089 - 1374	1235.00	666-650	627.27	28.00	60.00	31.00	65.00	-	-	-	-
COD (Kg/ha-day)	298-301	299.0	209.7 - 227.5	216.0	115.0 - 124	118.6	182.0 - 222.5	206.3	101.0 - 108.6	104.7								
SS (mg/l)	460-515	489.64	220-340	286.00	-	-	210-310	264.00	-	-	42.00	-	46.00	-	-	-	-	-
pH	7.7-8.5	8.16	7.6-8.0	7.86	-	-	7.7-8.2	7.90	-	-	-	-	-	-	-	-	-	-
ALKALINITY (mg/L as $CaCO_3$)	210-280	232.18	364-380	371.00	-	-	465-489	477.00	-	-	-	-	-	-	-	-	-	-
DO (mg/l)	0.3-9.2	5.70	2.6-3.1	2.87	-	-	3.4-4.0	3.78	-	-	-	-	-	-	-	-	-	-

TABLE 6-2-2: AVERAGE VALUES OF PERFORMANCE PARAMETERS

	SS	COD_u	COD_f	$COD_u - COD_f$	BOD_u	BOD_f	$BOD_u - BOD_f$	$COD_u - BOD_u$	$COD_f - BOD_f$
	22.82	21.00	15.00	6.00	15.10	12.18	2.92	5.90	2.82
	24.36	118.60	84.60	34.00	40.91	30.10	10.81	77.69	54.50
POND A	40.55	250.64	191.64	59.00	151.18	98.09	53.09	99.46	93.55
	92.00	600.00	375.00	225.00	273.00	191.18	81.18	327.00	183.82
	286.27	1293.00	710.36	582.64	574.00	450.27	123.73	719.00	260.09
	8.50	20.19	12.45	7.74	10.36	6.00	4.36	9.83	6.45
	10.18	96.00	43.00	53.00	22.73	18.27	4.46	73.27	24.73
POND E	49.45	237.18	181.00	56.18	127.18	51.00	76.18	110.00	130.00
	113.36	567.27	376.00	191.27	294.00	127.00	167.64	273.27	249.00
	263.82	1235.00	627.27	607.73	385.27	288.64	96.63	849.73	338.63

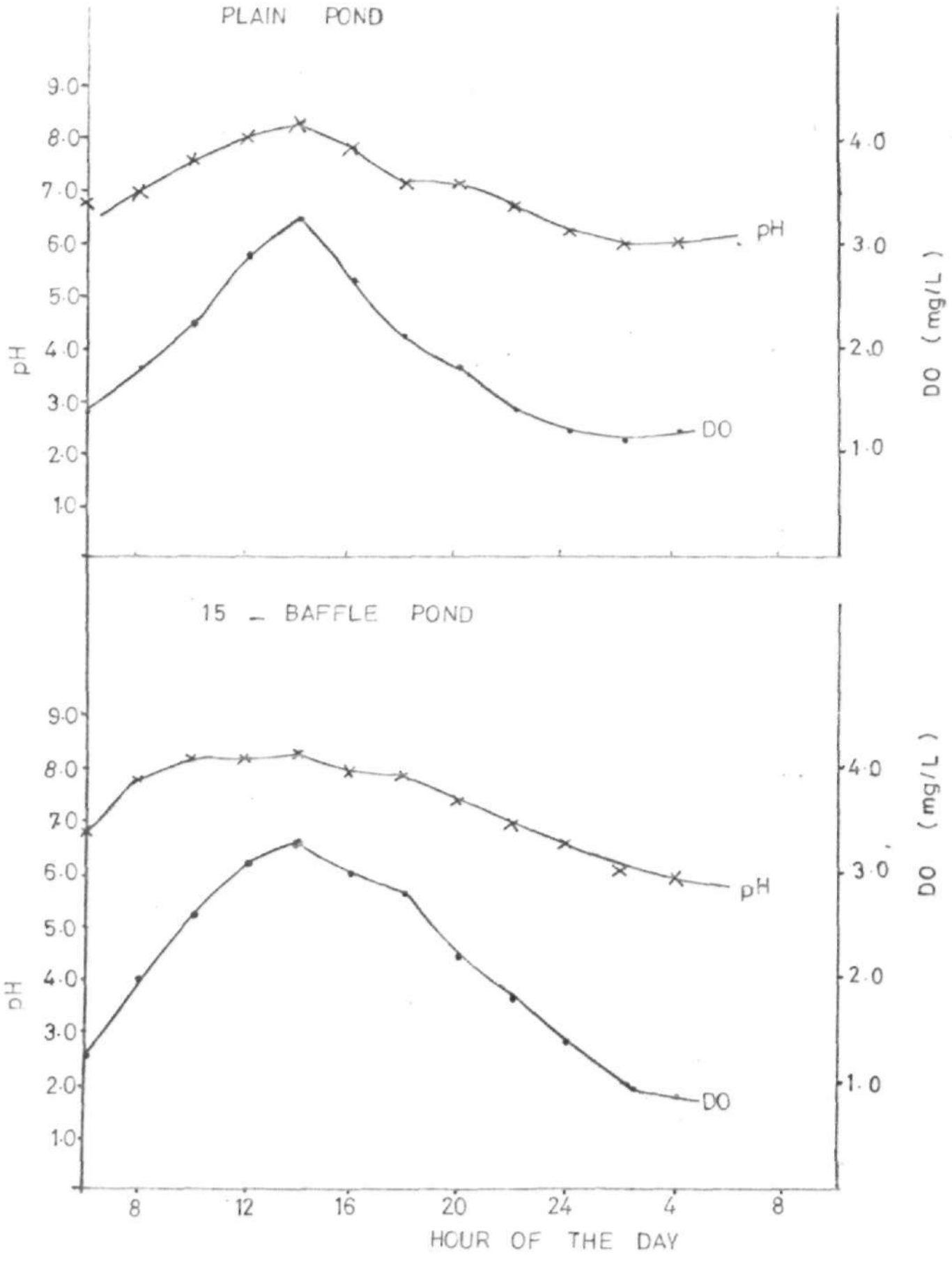


FIG 6-2-6 (a): pH AND DO DIURNAL VARIATIONS
TEST 1 (33 kg BOD_5/ha - day)

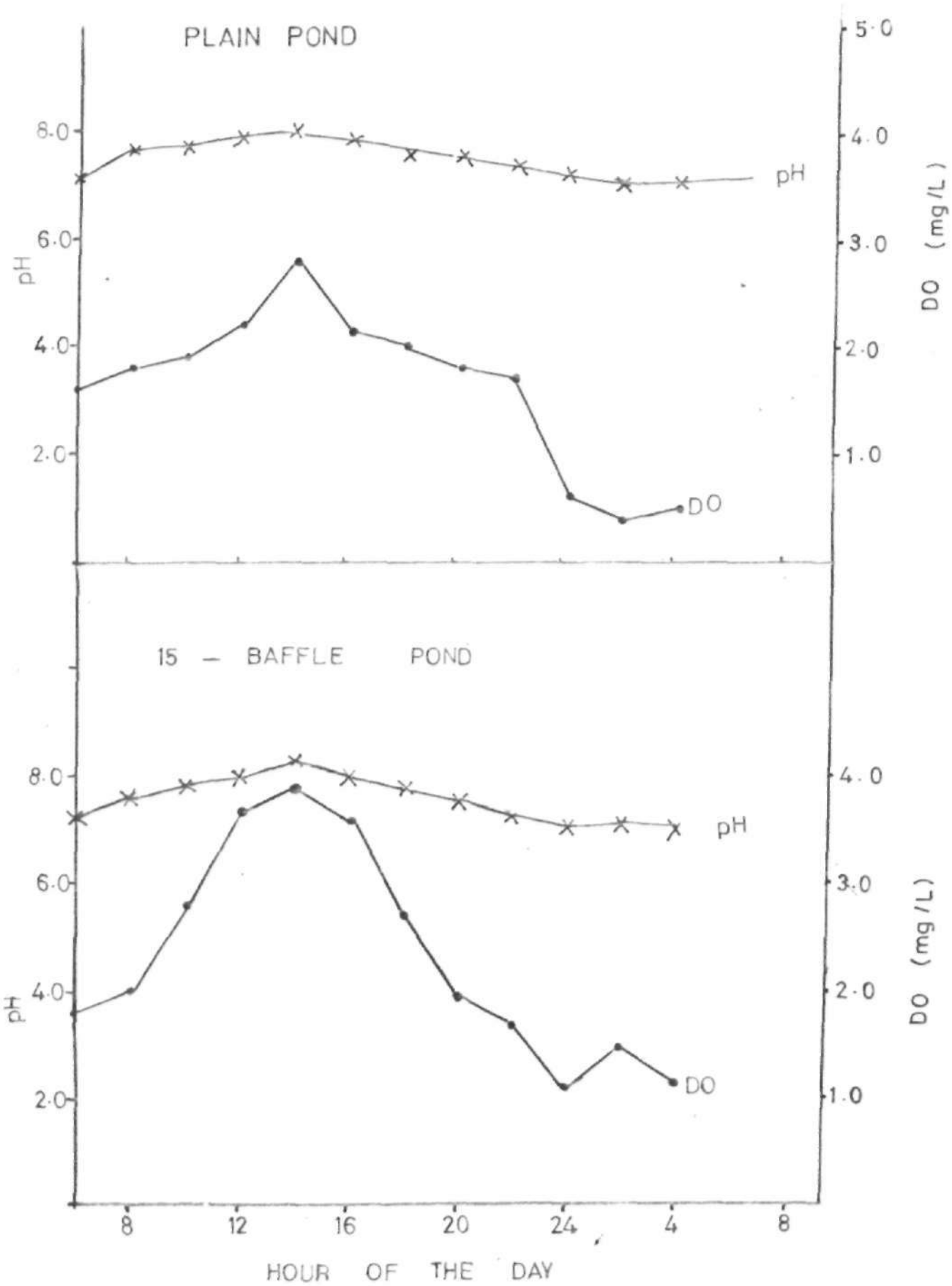


FIG 6-2-6(b): pH AND DO DIURNAL VARIATIONS
TEST 2 (50 kg BOD_5/ha-day)

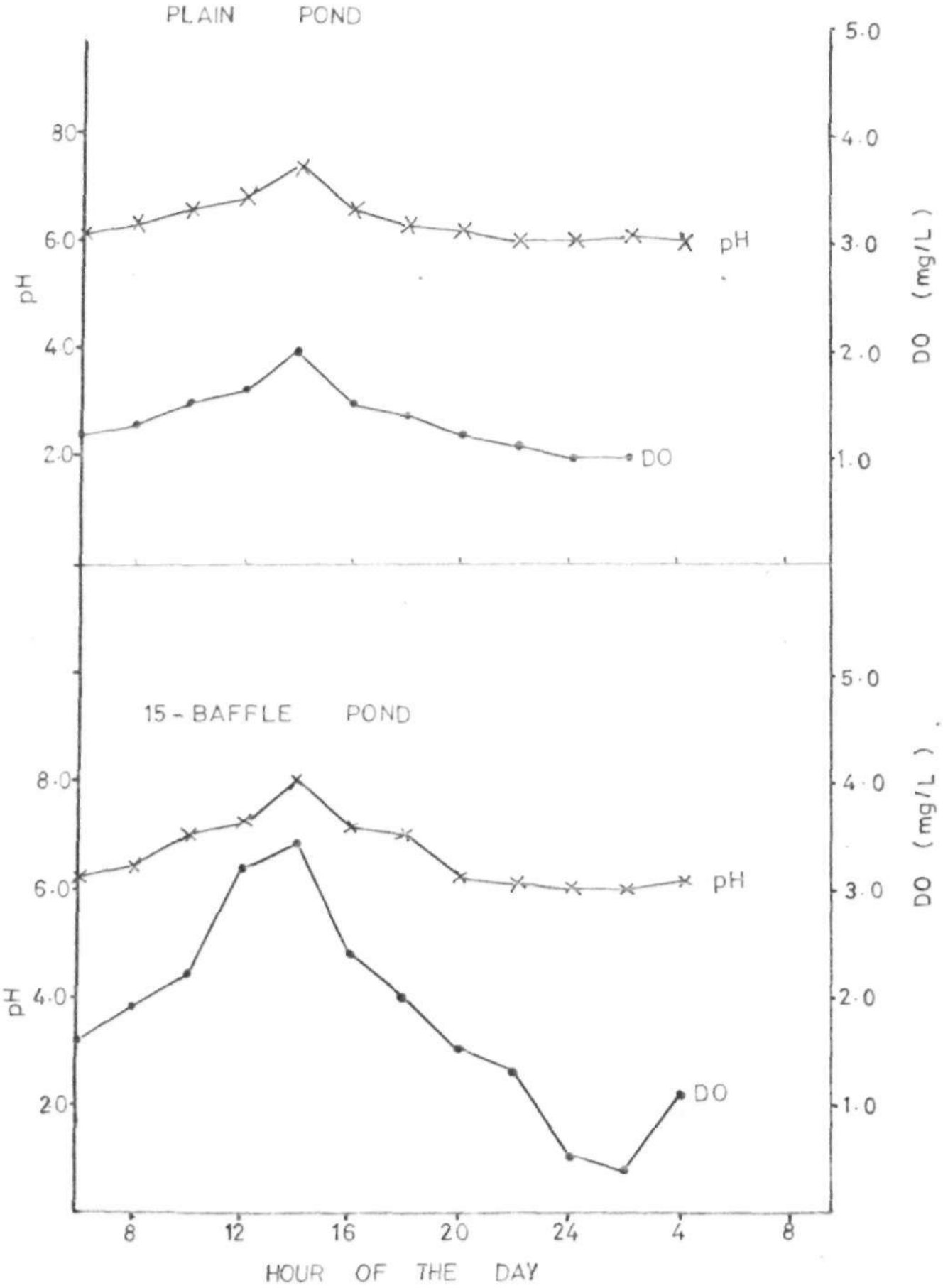


FIG 6-2-6 (c): pH AND DO DIURNAL VARIATIONS
(TEST 3)(100 kg BOD_5/ha-day)

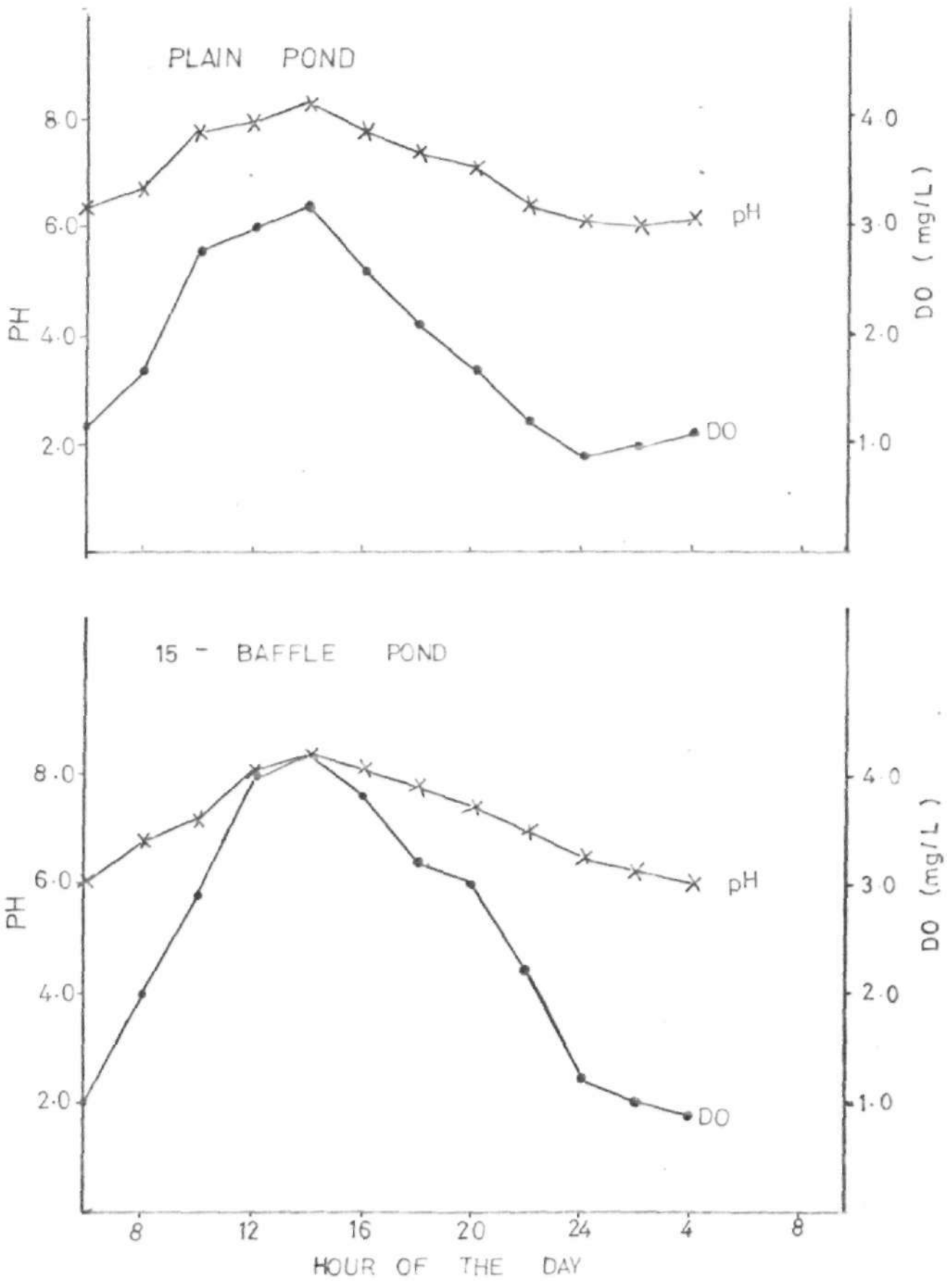


FIG 6-2-6(e): pH AND DO DIURNAL VARIATIONS

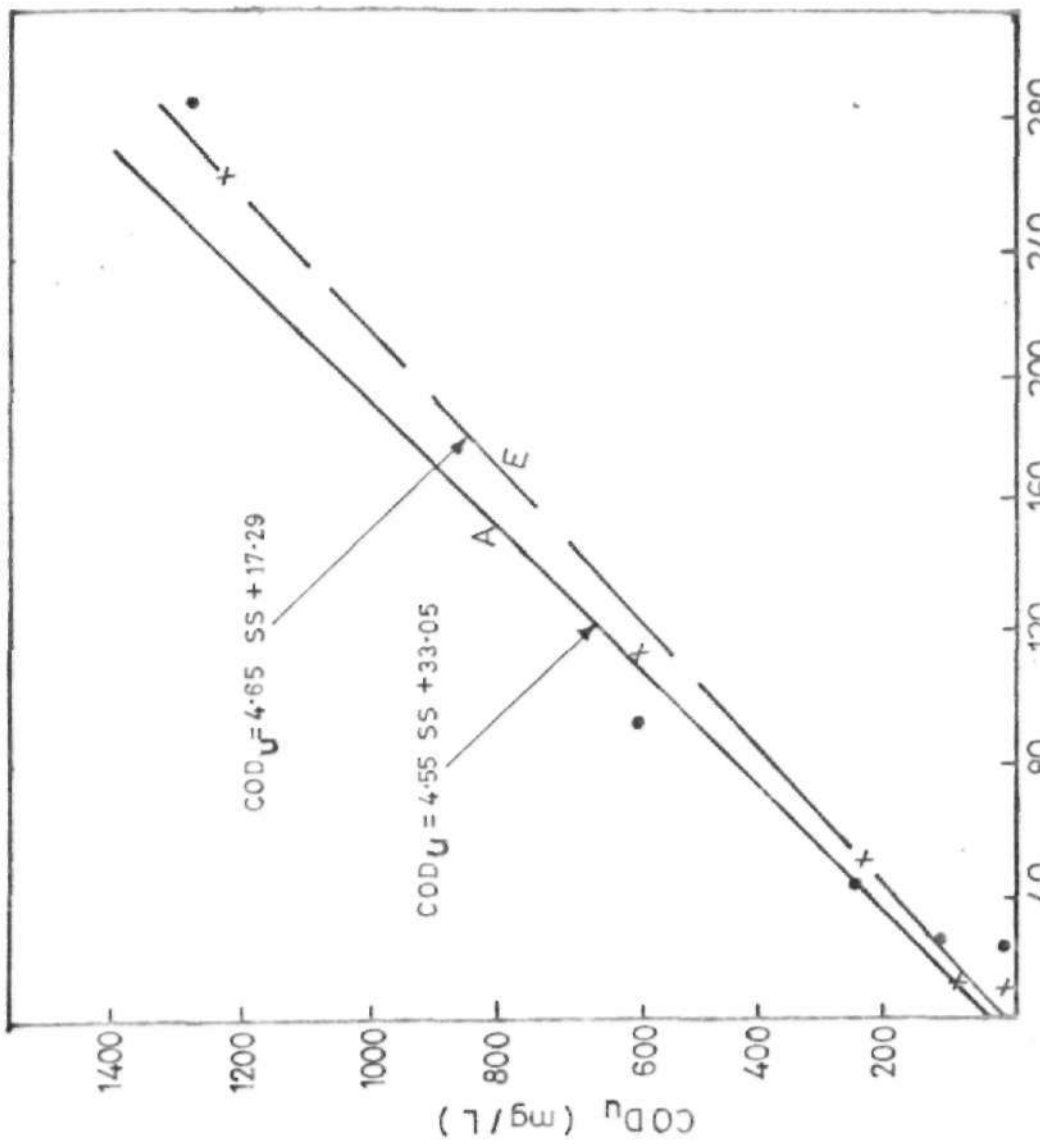


Fig. 6 - 2 - 7 COD_u Vs SS

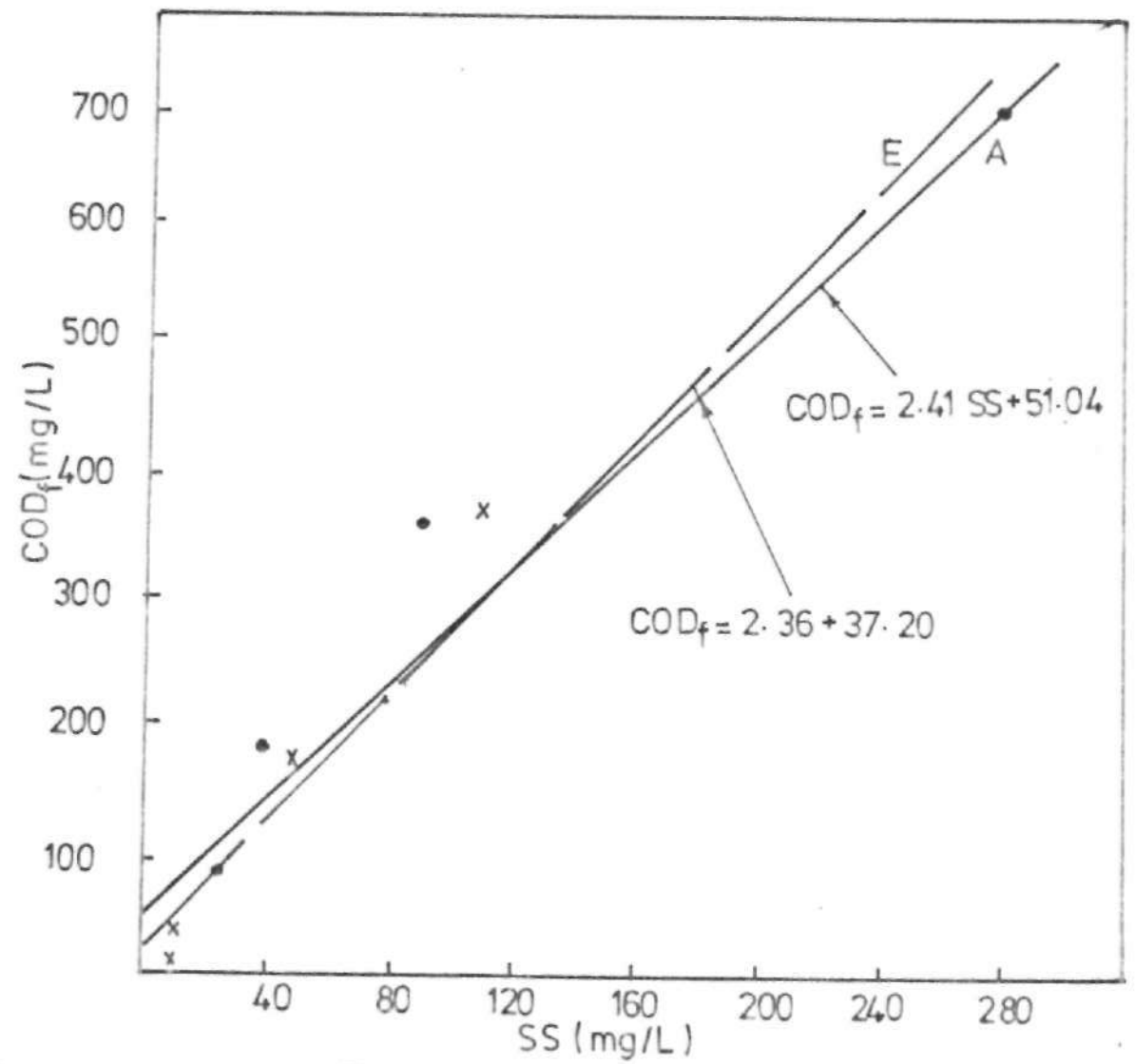


Fig. 6-2-8 COD_f Vs SS

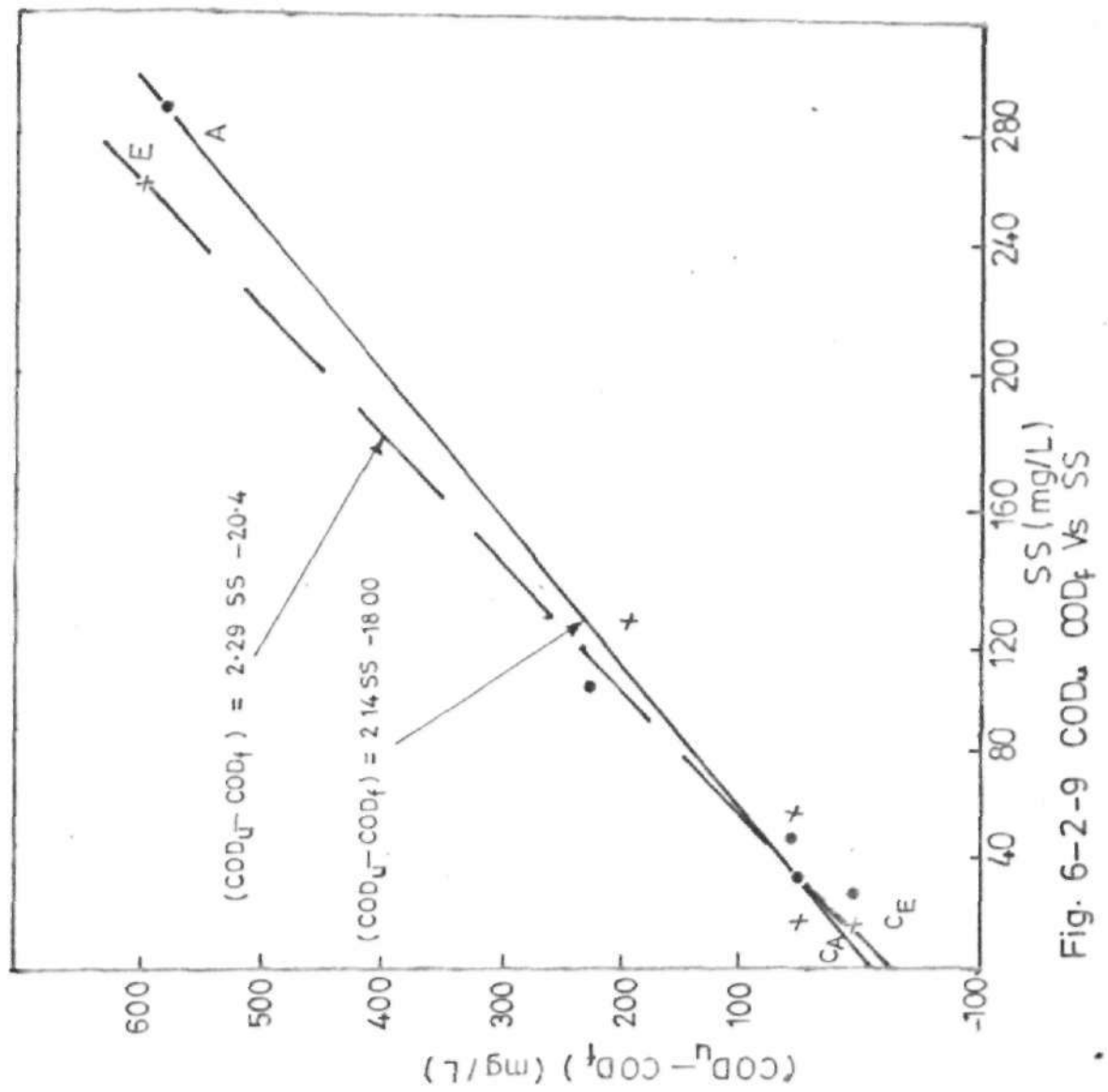


Fig. 6-2-9 COD_u COD_f Vs SS

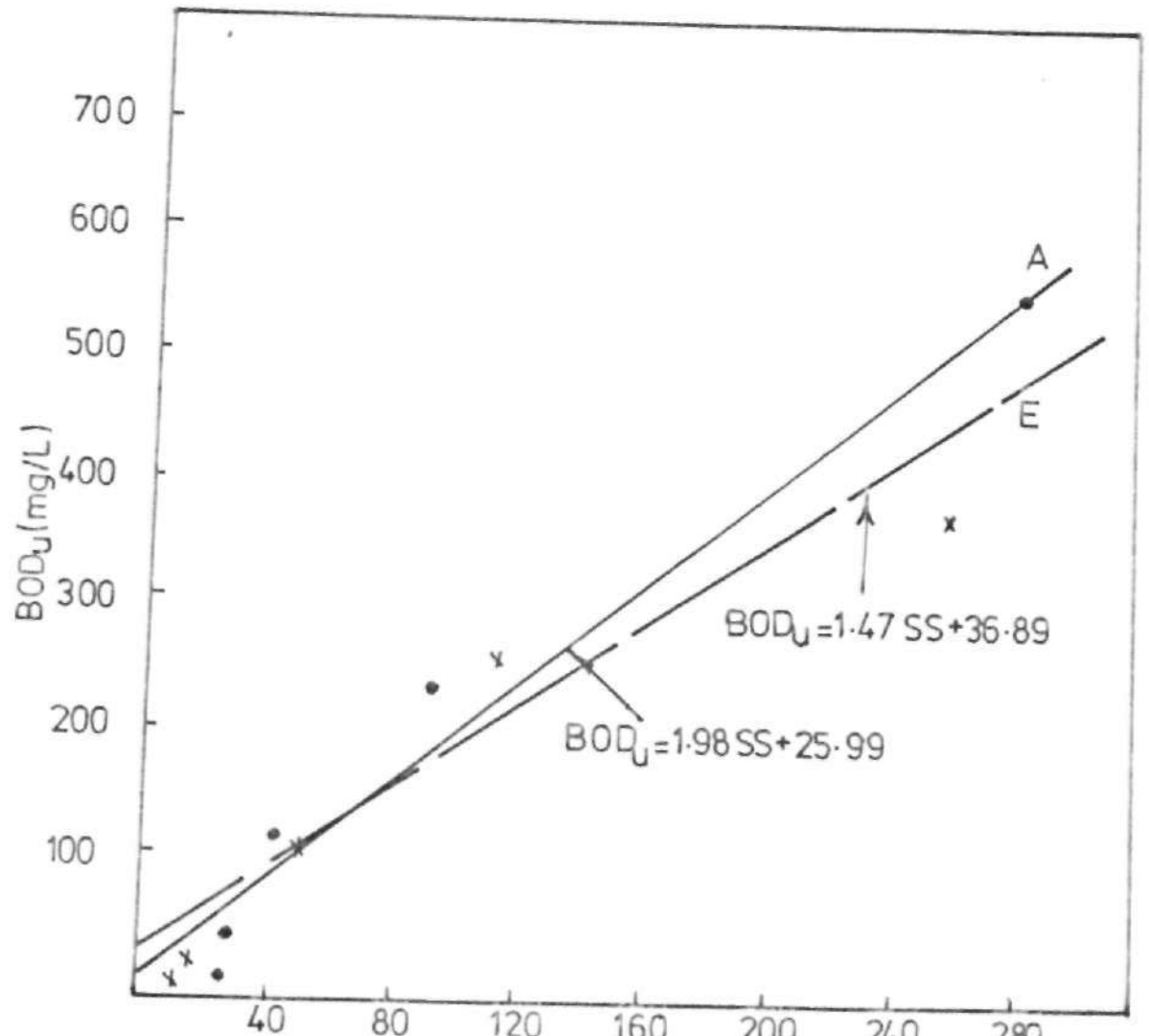


Fig. 6-2-10 BOD$_U$ Vs SS

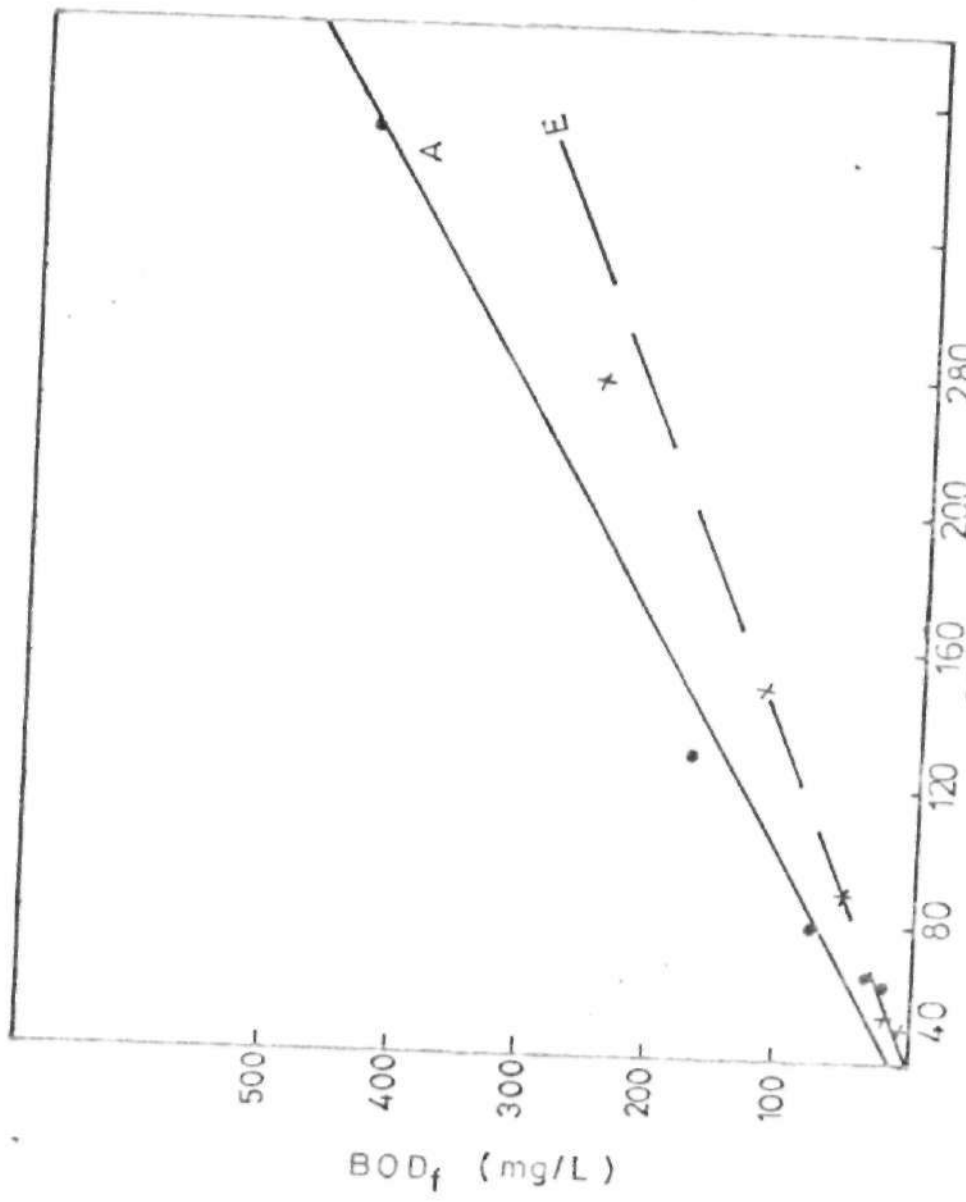


FIG 6-2-11 BOD$_f$ V_S SS

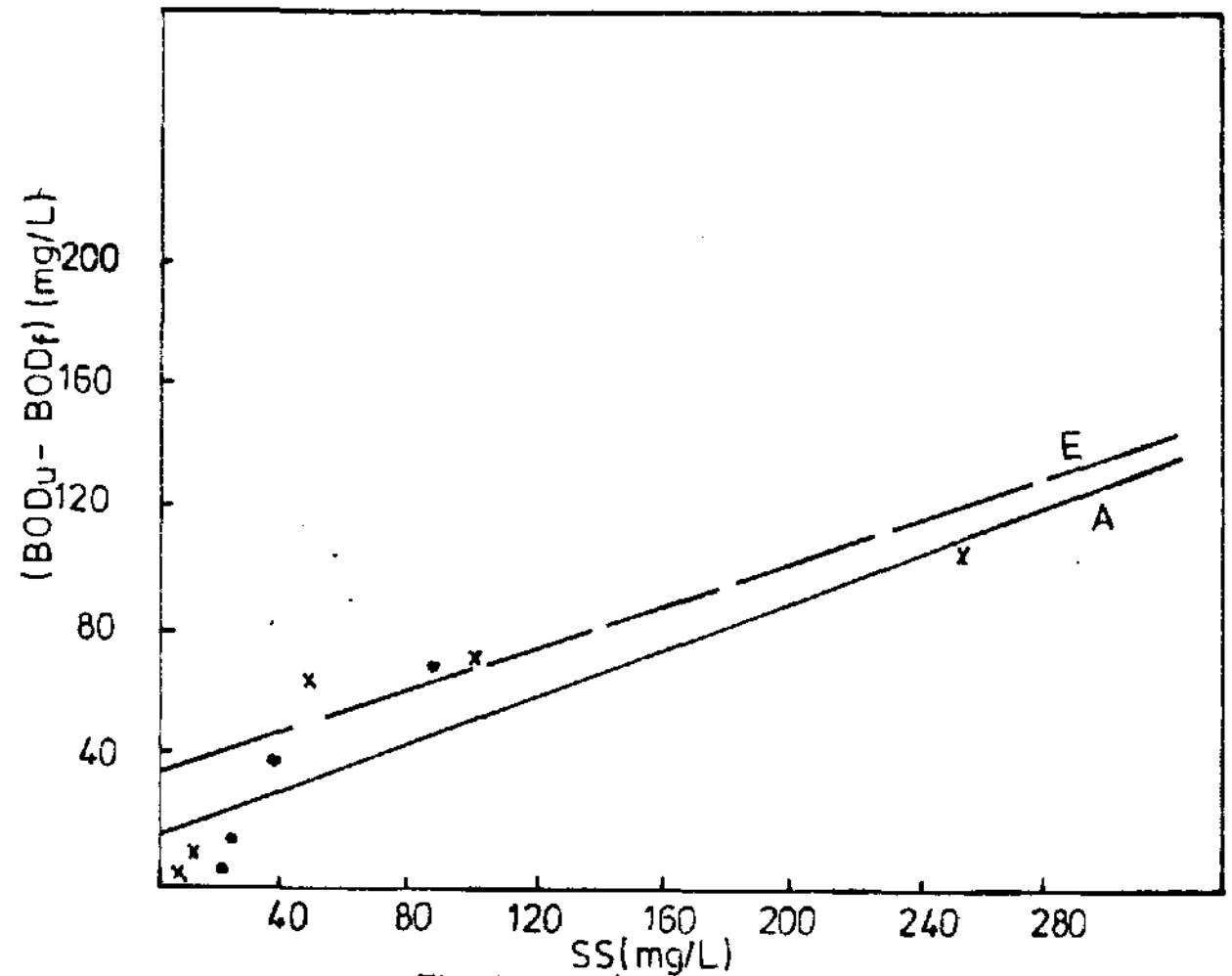


Fig. 6-2-12: (BOD$_u$ - BOD$_f$) Vs SS

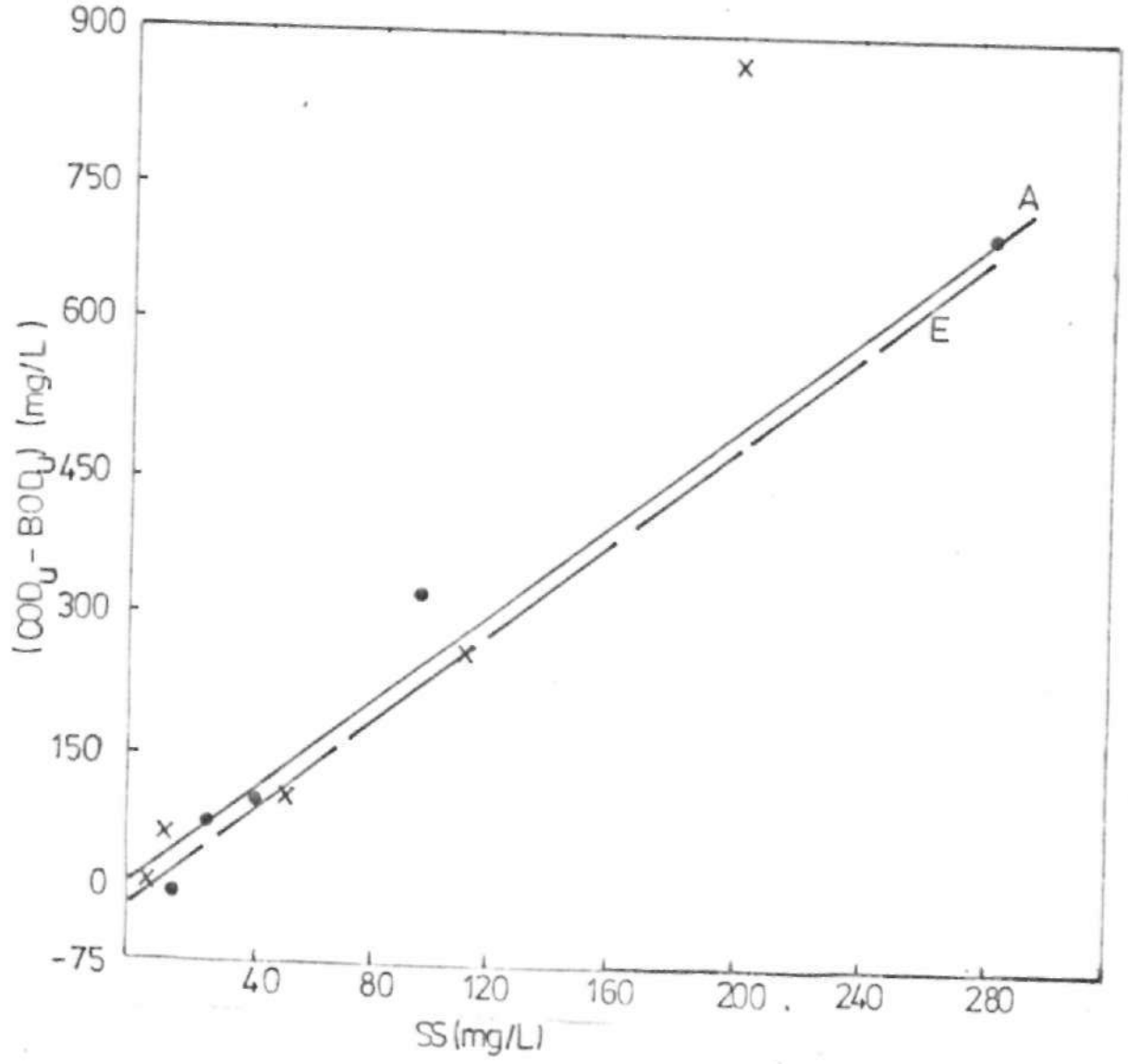


Fig. 6-2-13: $(COD_U - BOD_U)$ Vs SS

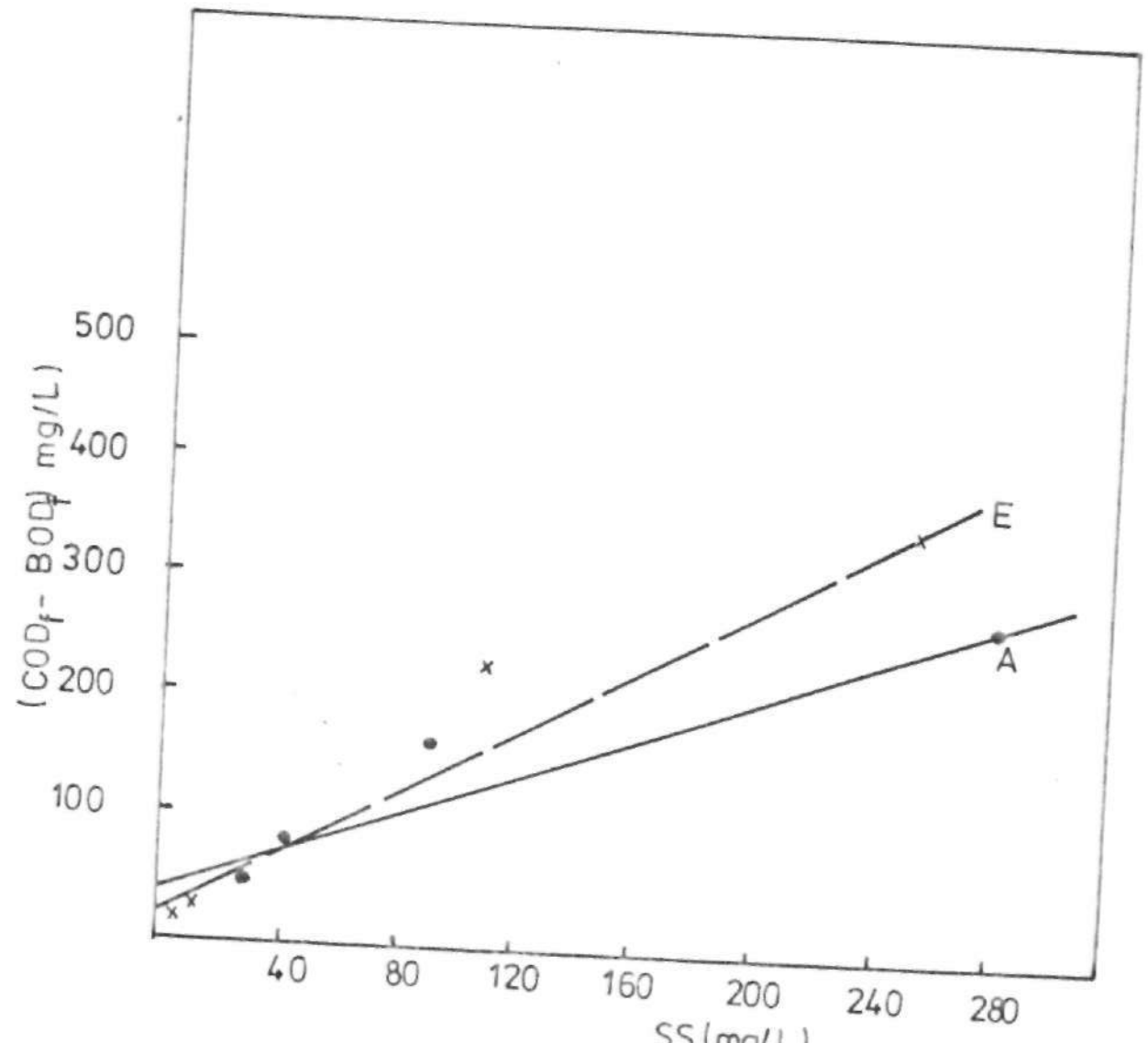


Fig. 6-2-14: $(COD_f - BOD_f)$ Vs SS

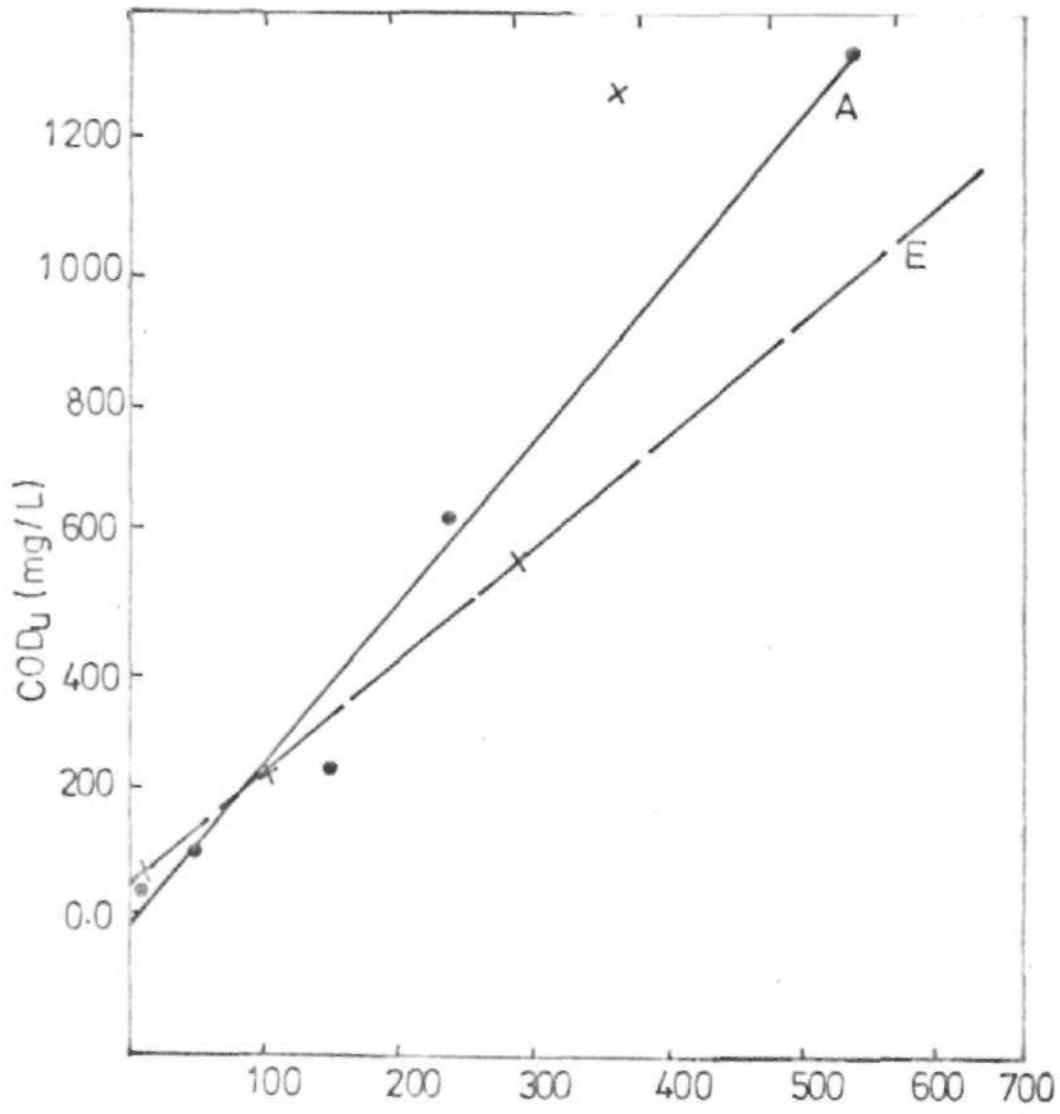


Fig. 6-2-15: COD_u Vs BOD_u

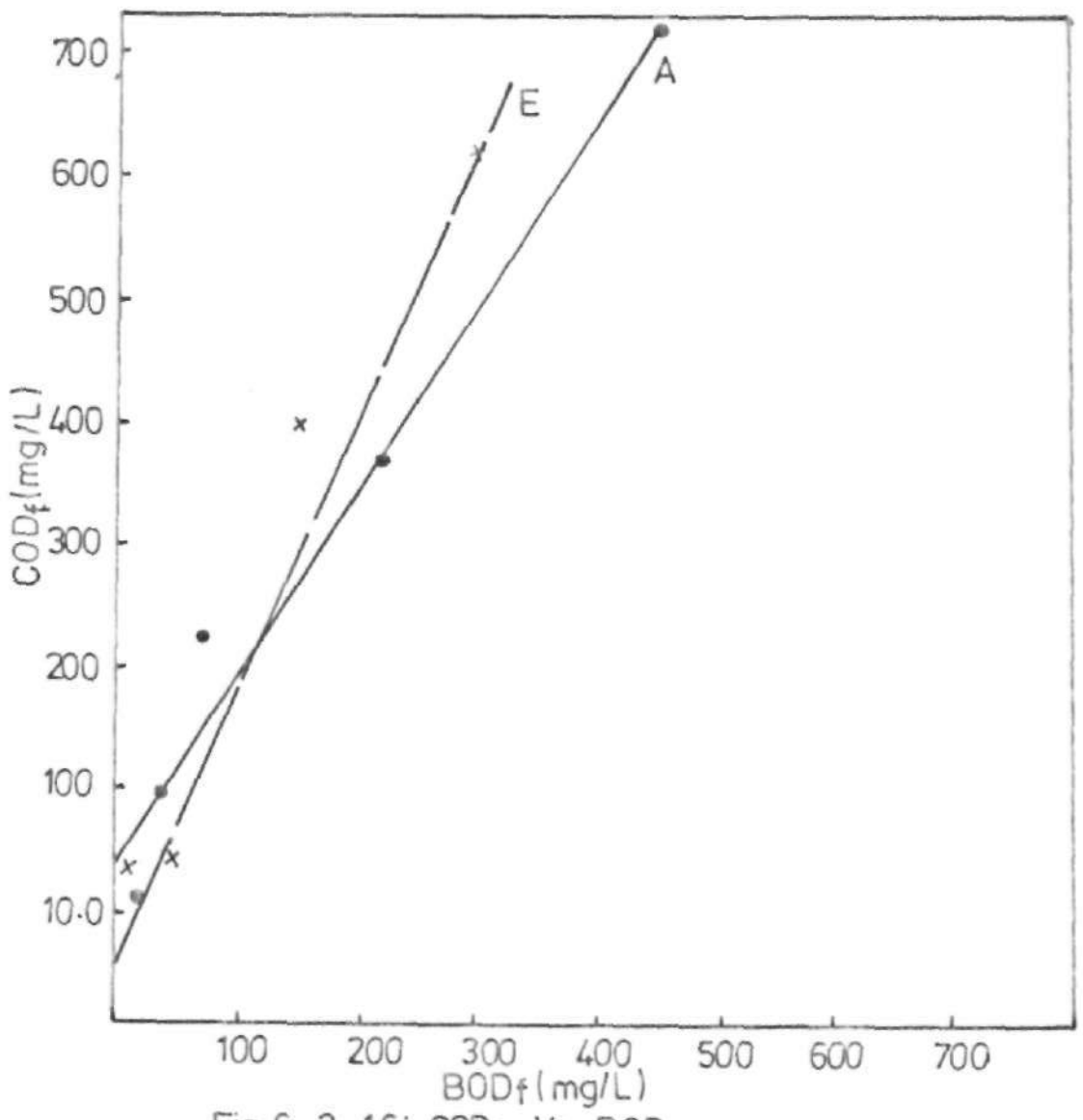


Fig. 6-2-16: COD$_f$ Vs BOD$_f$

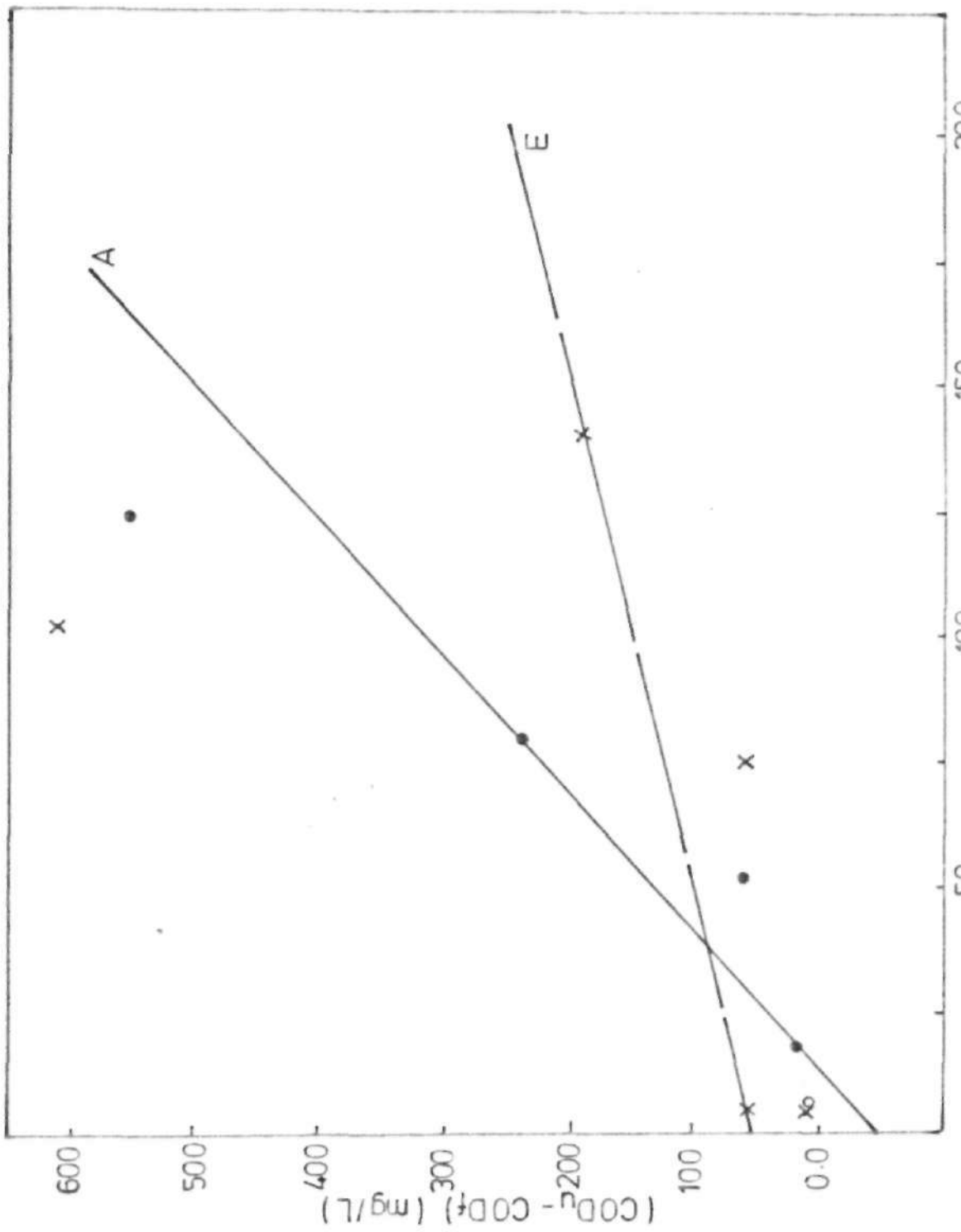


Fig. 6-2-17: $(COD_u - COD_f)$ Vs $(BOD_u - BOD_f)$

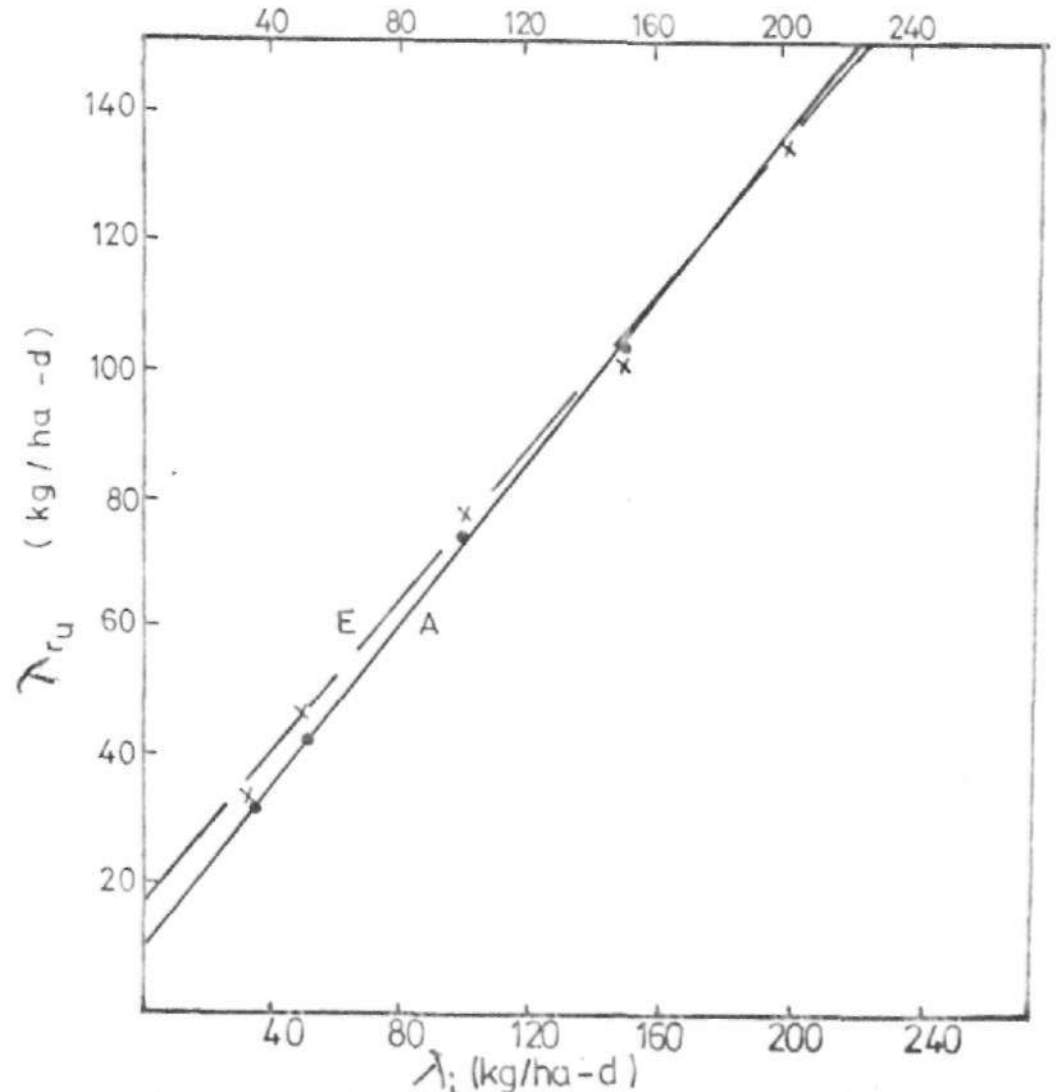


Fig 6-2-18: BOD SURFACE REMOVAL, λ_{ru} Vs LOADING, λ_i

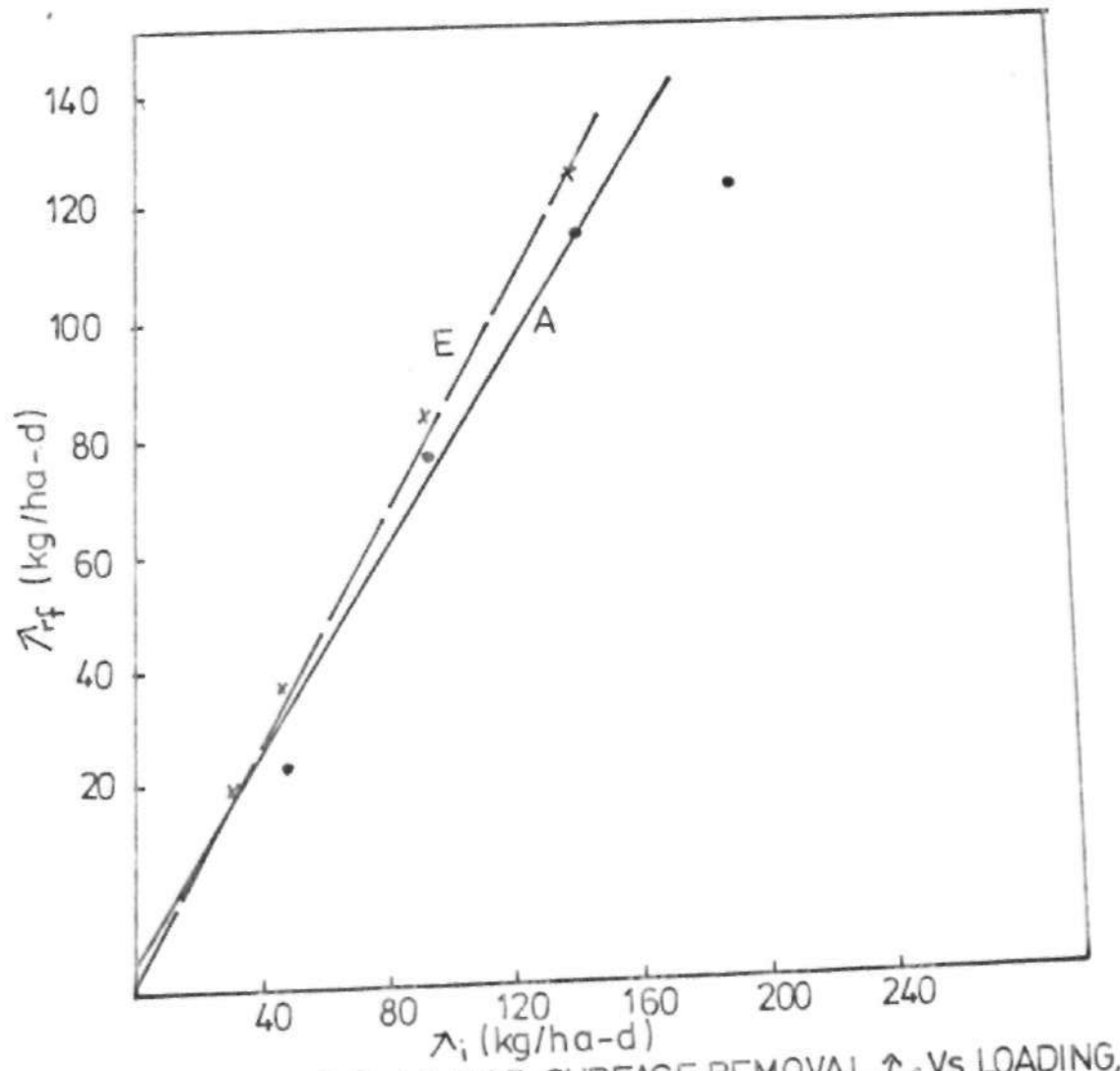


Fig. 6-2-19 BOD SURFACE REMOVAL, $\nearrow_{rf}$ Vs LOADING, $\nearrow_i$

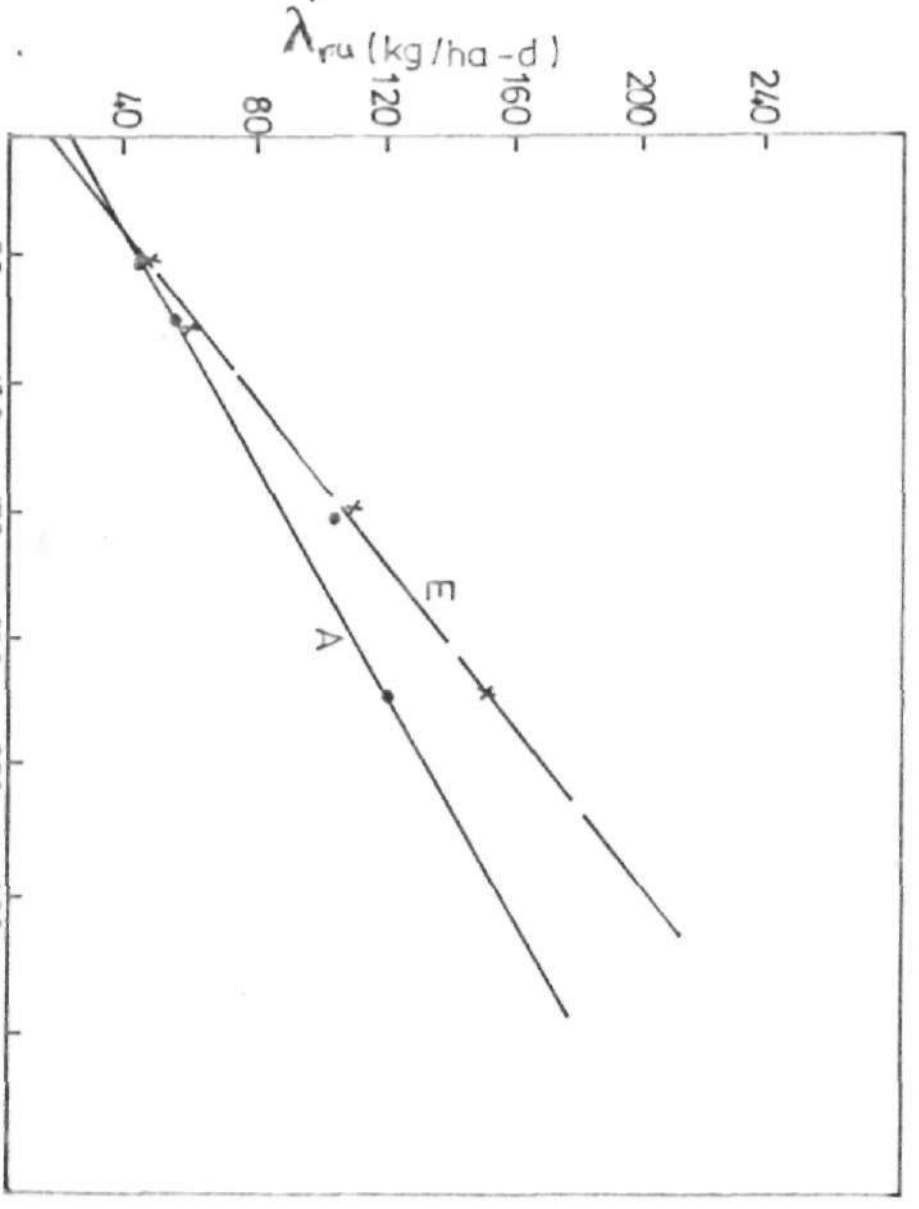


Fig. 6-2-20 COD SURFACE LOADING λ'_i Vs REMOVAL

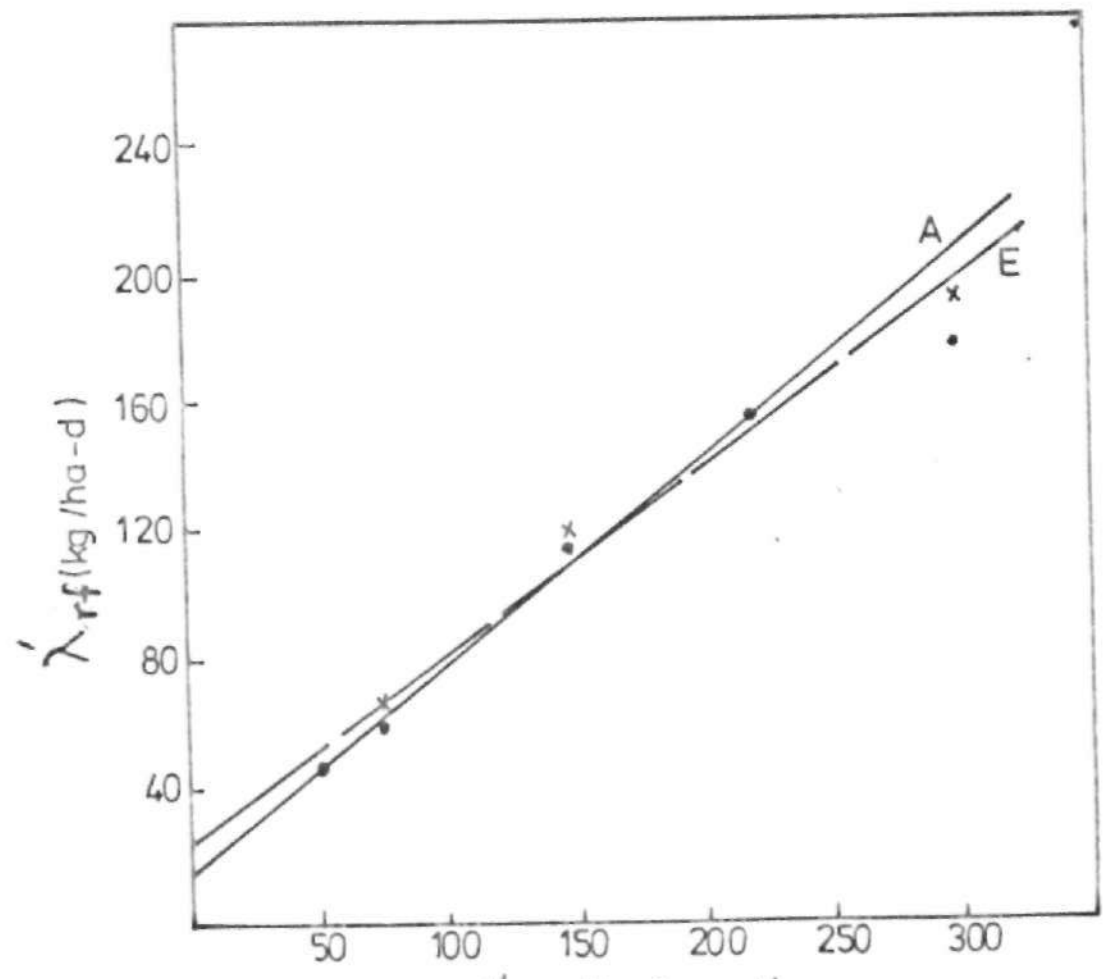


Fig. 6-2-21 COD SURFACE LOADING λ'_i Vs REMOVAL λ'_{rf}

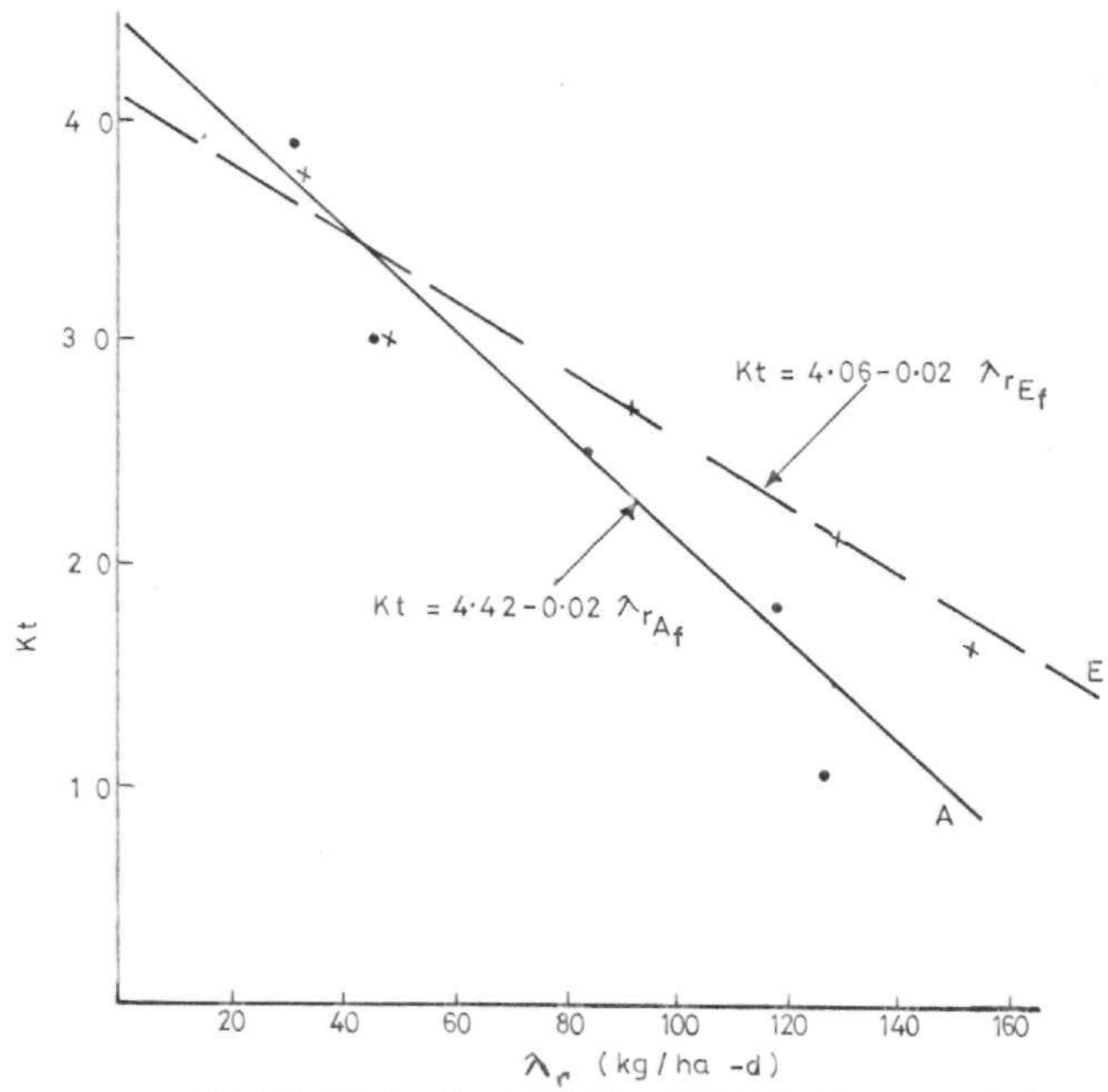


FIG 6-2-22 Kt Vs ORGANIC SURFACE REMOVAL, λ_r (FILTERED)

TABLE 6-3-1: SUMMARY OF MICROBIOLOGICAL INVESTIGATIONS

BOD_5 (mg/L)	PERCENTAGE BACTERIAL DIE - OFF						ALGAL CONCENTRATION (10^6/100 mL)*	
	POND A			POND E			POND A	POND E
	TC	FC	FS	TC	FC	FS		
200	99.00	99.10	91.10	94.60	99.80	99.90	20.00	23.00
300	89.80	98.70	98.30	91.70	99.80	99.90	21.00	26.00
600	89.80	98.60	98.50	92.10	99.70	99.90	23.00	32.00
900	89.20	98.30	98.30	89.40	99.20	99.50	26.00	41.00
1200	84.00	98.40	98.20	86.30	80.80	81.00	34.00	54.00

*Cells

CHAPTER 7

7.0 DISCUSSION OF RESULTS

7.1.0 Mixing Characteristics

As it was pointed out in Chapter 3, the hydraulic characteristics and the performance of stabilization ponds can be predicted from the tracer recovery (flow through) curves. Previous works have shown that stabilization ponds behave as dispersed flow reactor which is characterized by the combination of influence of detention time, dead spaces, lateral dispersion, short circuiting and standard detention time efficiency. (Thirumurthi, 1967, 1969 and Kilani and Ogunrombi, 1984). The more the deviation from an ideal plug flow, the lower the efficiency of the systems.

7.1.1 Effects of Baffles on Hydraulic Characteristics

The tracer recovery curves for the experimental ponds operated at various theoretical detention times are shown in Figures 6-1-1 through 6-1-4 and the summary of the mixing characteristics of all the ponds is shown in Table 6-1-1.

For 7, 11 and 15 day detention times 100 percent tracer concentration revoveries were obtained in all the tanks. But with 20 day detention time, tank A (plain tank) and tank B with 6 baffles gave tracer recoveries as low as 63.0 percent and a maximum of 94.0 percent in tank E with 15 baffles. Tanks C and D gave 69.0 percent and 75.0 percent recoveries respectively. The observed recoveries at 20 days may probably be due to the influence of settling velocity on the tracer material. Although both the settling and longitudinal velocities were not measured, it would appear that the effect of settling velocity

was significant when detention time was 20 days. The above statement may be justified by the observed film of tracer material at the bottom of the ponds at this detention time.

A curve which is high and narrow and attains its peak close to the theoretical detention time indicates a high mixing efficiency. In this study tanks D and E appear to meet these criteria at 7, 11 and 15 day detention times respectively while at 20 days none of the tanks met the criteria.

In the present study it has been observed that the actual detention time, t_g is generally smaller than the nominal (theoretical) detention time, t_n and t_g increases as the number of baffles increases (Figure 6-1-5). Thus the percentage deviation of t_g from t_n, was observed to decrease with increasing number of baffles (Figure 6-1-7). This is in agreement with the views of the previous investigators (Kilani and Ogunrombi, 1984). The lower the value of Ψ, the better the performance of the ponds in terms of mixing characteristics. In this study the lowest value of Ψ was obtained when 15 baffled pond was used. In Figure 6-1-8 a steady increase in t_g to a maximum value was observed when t_n increased to 15 days and thereafter a decreasing trend in the value of t_g was observed when t_n was increased to 20 days. Since the volume of the ponds were kept constant while changing the flow rates, the observed results tend to suggest the existence of a critical range of nominal

detention time at which stabilization ponds are most efficient. Operation of ponds outside these critical values may not necessarily improve efficiency or may even worsen the performance. In the present laboratory study, the flow rate and detention time were observed to be critical at 8.33 L/d and 15 days respectively.

In an attempt to describe the effectiveness of a reactor in terms of detention time the use of standard detention time efficiency t_η had been suggested (Fiedler and Fitch, 1959). This is a ratio of the effective detention time, t_s to the nominal detention time, t_n. High values of t_η had been associated with good pond performance. In the present study, the lowest value of t_η was given by t_n of 20 days while the highest value was obtained at t_n = 15 days. It is also shown that t_η increases with increasing number of baffles (Figure 6-1-9). The highest value was obtained for 15 baffle pond when operating at t_n of 15 days.

Other parameters have been used to explain the mixing characteristics in ponds such as the index of dead space, α_d, the index of short circuiting α_s and the dispersion index number, d. When short circuiting is apparent the actual detention time, t_g may be greater or less than t_n. It has also been suggested that when t_g is greater than t_n or when α_d is greater than unity, there exist dead spaces into which the tracer diffuses at an early stage of flow and diffuses out later thus causing the flow curve to be unduly

elongated with a resultant shift in t_g (Rebhum and Arganian, 1965; Kilani and Ogunrombi, 1984)

With this study, α_d was observed to increase with increasing number of baffles (Figure 6-1-10). This implies that the percentage dead volume δ decreases with increasing number of baffles (Figure 6-1-11) from Table 6-1-1, α_d is greater than unity when 15 baffles pond was operating at 15 day nominal detention time. The index of short circuiting, α_s was observed to decrease with increase in the number of baffles (Figure 6-1-12). This is in agreement with previous investigators (Kilani and Ogunrombi, 1984).

As suggested by Thirumurthi, α_s are not very good indicator of degree of mixing in the ponds (Thirumurthi, 1969). The dispersion index, d has been suggested to be the most quantitative and qualitative (Thirumurthi, 1967, 1969; Mangelson and Watters, 1972 and Kilani and Ogunrombi, 1984). It has also been suggested that d has the strongest and statistical probability of describing the hydraulic performance of ponds. Its derivation involves important parameters such as the time and concentration in the tracer study (Appendix B). The larger the number of points on the C-curve (flow-through curve) which are included in the calculations of an index, the smaller is the relative error in the measurement (Thirumurthi, 1969). As d tends to zero, the reactor tends to plug flow system and hence more efficient.

In this study the values of d were observed to decrease with increasing number of baffles at any detention time (Figures 6-1-13 and 6-1-14). At any hydraulic loading, there is no much difference in mixing as measured in terms of d in an unbaffled tank. The difference in mixing efficiency becomes progressively pronounced with increase number of baffles. Since d = D/uL, the longer the path length, the smaller the value of d. The objective of introducing baffles is to increase the length of travel, L_t (Figure D-1-1). For any detention time, tank E with 15 baffles gave the lowest value of d of 0.030. This lowest value occuring during the 15 day detention time instead of in the expected 20 day detention time (Figure 6-1-14).

Long detention time improves pond efficiency. However, in selecting the detention time one has to consider a very suitable flow rate. So the volume has to be adjusted if one is hooked up to a particular flow rate that may affect pond efficiency. 20 day detention time was observed to be poor for this particular study hence it was eliminated.

15 day detention time was then chosen in view of the longer detention time compared with other detention times of 7 days and 11 days.

As the number of baffles increases cost of materials and maintenance may increases. The relationship between the improved efficiency obtained for baffled ponds and increased cost of materials and maintenance has to be further investigated. For tank D to have

the same efficiency with tank E, the volume has to be increased. It was previously stated that for the same amount of efficiency, plug flow has smaller volume than other flow regimes, equation 3-2-45;, and tank E is closer to plug flow than tank D.

7.2.0 Treatment Performance

From the tracer tests analysis, it became obvious that Pond E with 15 baffles performed best in terms of mixing characteristics. Consequently the pond was used for the treatment performance study with pond A without baffle as control. Also from the previous analysis, 15 day nominal detention time was considered ideal and was used throughout the treatment performance study. The 15 day detention period was achieved by keeping both the volume of the ponds about 125 Litres and the flow rates about 8.33 L/day constant. While the detention time was maintained constant, the organic loadings were varied with changes in the BOD of the synthetic milk waste. The purpose of changing the organic loading rates was to study the effects of increased loadings of ponds' performance.

7.2.1 Organic Loading and Removal Rates

In order to minimize the fluctuation in the organic characteristics of the influent feeds, sufficient amount of synthethic milk waste was prepared at each feed to last 2 days. There were five experimental runs of which the average areal BOD_5 loadings of the synthetic waste fed into the ponds

at each run were: 33, 50, 100, 150 and 200 kg BOD_5/ha-day. The corresponding COD of the feeds were: 50, 76, 150, 225 and 300 kg/ha-day with suspended solids of 14, 18, 33, 67 and 82 kg/ha-day respectively.

The BOD and COD of the ponds' effluents were determined for both unfiltered and filtered samples. The former values would represent the total BOD and COD while the latter represent the soluble BOD and COD of the effluents. For the control pond, the percentage total BOD removals were 92, 86, 75, 70 and 52 percent corresponding to increase in organic loadings. The total COD removal were 93, 74, 72, 55 and 28 percent while the SS removal were 76, 77, 80, 77 and 42 percent respectively. For the baffled pond the total BOD removal were 95, 92, 79, 67 and 68 percent; the total COD removal were 93, 79, 74, 58 and 31 percent in this baffled pond while the SS removal were 90, 91, 75, 71 and 46 respectively. The observed deterioration in the effluent quality as the organic loads are progressively increased is attributed to the significant increases in algal population reaching the effluents as the organic loadings were increased. Also high organic loads might result in more benthal deposits with intense anaerobic activities which might influence the resuspension of more solids to the effluents. However, there was some improvement in the quality of the effluent after filtration process which removed some of the algal cells from the effluent.

7.2.2 Effect of Baffle on Treatment Performance

The rssults of this investigation confirm the view of previous workers in showing that as a result of using baffles there is an improvement in removal efficiency of organic and solid matters in the ponds. For example at an organic loading of 200kg BOD/ha-day an improvement in efficiency of about 14% was noted in BOD removal. The mechanism for organic removal in the baffled pond may be explained by the prevailing biological environments and the mixing characteristics within the ponds. The submerged surface of the baffles would encourage the growth of attached biomas or microorganisms which are responsible for organic biodegradation. Also because of longer length of travel in baffled pond, the organic matter is expected to remain much longer in it for the bacteria degradation than for the plain pond.

It has been established that plug flow reactors with d = 0 are more efficient than a completely mixed flow reactor with d equals to infinity. Facultative ponds operate between these two extremes. From the tracer tests, baffled ponds have been shown to have smaller dispersion index than the plain pond when operated under the same hydraulic loadings. In this study the value of d for the plain pond was found to be 0.151 while that of the 15 baffled pond was 0.030. These results indicate that the flow pattern in the baffled pond would approach that of a plug flow than an unbaffled pond.

Generally baffled pond may be regarded as a system of ponds

in series whose overall performance would approach that of a plug flow reactor and hence tends to be more effective than a single plain pond of the same size. In the present study it was found that the actual detention time of the baffled pond is about 2.32 times that of the plain pond. This implies that the effective volume of the baffled pond is also about 2.32 times that of the plain pond. It would therefore appear that in order to achieve the same efficiency as the baffled pond one would need a plain pond which is at least twice the size of the baffled pond. Hence where land is a constraint, the use of baffles would be an advantage.

7.2.3 Correlation of Organic Loading and Removal Rates

It had been pointed out that the major problem of stabilization ponds is SS removal. So rather than look at SS removal efficiency, the impact of SS on the other two chemical features namely BOD and COD were looked into. There is no direct correlation between the influent SS and the effluent SS due to the nature of the waste (Synthetic milk waste), algae and other microbial cells inside the ponds.

Figures 6-2-14 and 6-2-15 present the variations of organic loading rates λ_i with removal rates λ_r for both the plain and baffled ponds based on unfiltered (total) and filtered (soluble) BOD; while in figures 6-2-16 and 6-2-17 are presented similar variations in terms of COD.

With the assumption that these parameters follow a bivariate normal distribution, the relationships between these parameters were analysed by means of regression analysis. With a programmable desk calculator the correlation coefficients, r and the coefficient of determination, r^2 were found. Under the hypothesis that n = 0, the correlation coefficient is related to the t-distribution for small sample size (less than 500) by the expression:-

$$t = r\{(n-2)/(1-r^2)\}^{\frac{1}{2}} \quad .. \quad .. \quad .. \quad 7\text{-}2\text{-}1$$

which follows the t-distribution with (n-2) degrees of freedom. The null hypothesis is tested by computing the

value of t (t_{obs}) as given by equation (7-2-1) and comparing that value with the value available in standard tables of the t-distribution with (n-2) degrees of freedom at any stated level of significance. For the degree of freedom under 95 percent level of certainty in this study, t is given as 3.182. If the calculated t (t_{obs}) is greater or equal to t (t = 3.182) from the table, then the null hypothesis can be accepted. However, if the value is less than t then the relationship suggested by the regression equation is weak and probably not real hence the null hypothesis should be rejected.

The analysis of the effluent BOD showed the following relationships betwen λ_i and λ_r

For the unfiltered (total) BOD the relationships are:

$\lambda_{rAu} = 0.64\ \lambda_i + 10.00$ ..... 7-2-2(a)

$r^2 = 0.92$

$t_{obs} = 5.86,\ 5.86 > 3.182$

$\ell n\ \lambda_{rAu} = 7.26\ \lambda_i + 3.39$ ..... 7-2-2(b)

$r^2 = 0.84$

$t_{obs} = 4.07$

$\ell n\ \lambda_{rAu} = 0.67 \ell n\ \lambda_i + 1.2$ ..... 7-2-2(c)

$r^2 = 0.96$

$t_{obs} = 6.90$

$$\lambda_{rEu} = 0.67\lambda_i + 14.2 \quad \ldots\ldots \quad 7\text{-}2\text{-}3(a)$$

$r^2 = 0.99$

$t_{obs} = 12.16$

$$\ell n\ \lambda_{rEu} = (8.073 \times 10^{-3})\lambda_i + 3.38 \quad \ldots\ldots \quad 7\text{-}2\text{-}3(b)$$

$r^2 = 0.94$

$t_{obs} = 6.9$

$$\ell n\ \lambda_{rEu} = 0.77\ \ell n\lambda_i + 0.82 \quad \ldots\ldots \quad 7\text{-}2\text{-}3(c)$$

$r^2 = 1.0$

$t_{obs} = \infty$

And for the filtered (soluble) BOD the relationships are:

$$\lambda_{rAf} = 0.77\lambda_i + 6.02 \quad \ldots\ldots \quad 7\text{-}2\text{-}4(a)$$

$r^2 = 0.95$

$t_{obs} = 8.53$

$$\ell n\ \lambda_{rAf} = (8.421 \times 10^{-3})\lambda_i + 3.34 \quad \ldots\ldots \quad 7\text{-}2\text{-}4(b)$$

$r^2 = 0.88$

$t_{obs} = 4.77$.

$$\ell n\ \lambda_{rAf} = 0.821 \ell n\ \lambda_i + 0.58 \quad \ldots\ldots \quad 7\text{-}2\text{-}4(c)$$

$r^2 = 0.98$

$t_{obs} = 12.16$

$$\lambda_{rEf} = 0.84\ \lambda_i + 5.12 \quad \ldots\ldots \quad 7\text{-}2\text{-}5(a)$$

$r^2 = 0.99$

$t_{obs} = 12.16$

$$\ell n\ \lambda_{rEf} = (8.906 \times 10^{-3})\ \lambda_i + 3.41 \qquad \ldots\ldots\ 7\text{-}2\text{-}5(b)$$

$r^2 = 0.91$

$t_{obs} = 5.27$

$$\ell n\ \lambda_{rEf} = 0.86\ \ell n\ \lambda_i + 0.534 \qquad \ldots\ldots\ 7\text{-}2\text{-}5(c)$$

$r^2 = 0.99,\ t_{obs} = 12.16$

The effluent showed the following relationships for the two ponds total and soluble COD

For total COD,

$$\lambda'_{rAu} = 0.44\lambda'_i + 25.00 \qquad \ldots\ldots\ 7\text{-}2\text{-}6\ (a)$$

$r^2 = 0.41$

$t_{obs} = 1.44;\ 1.44$ is less than 3.182

$$\ell n\ \lambda'_{rAu} = (2.86 \times 10^{-3})\lambda'_i + 3.88 \qquad \ldots\ldots\ 7\text{-}2\text{-}6\ (b)$$

$r^2 = 0.47$

$t_{obs} = 1.65$

$$\ell n\ \lambda'_{rAu} = 0.47\ \ell n\ \lambda'_i + 2.06 \qquad \ldots\ldots\ 7\text{-}2\text{-}6\ (c)$$

$r^2 = 0.66$

$t_{obs} = 2.39$

$$\lambda'_{rEu} = 0.47\ \lambda'_i + 25 \qquad \ldots\ldots\ 7\text{-}2\text{-}7\ (a)$$

$r^2 = 0.49$

$t_{obs} = 1.70$

$$\ell n\ \lambda'_{rEu} = (3.145\times10^{-3})\lambda'_i + 3.90 \quad \ldots\ldots \text{7-2-7 (b)}$$

$r^2 = 0.53$

$t_{obs} = 1.85$

$$\ell n\ \lambda'_{rEu} = 0.51\ \ell n\lambda'_i + 1.94 \quad \ldots\ldots \text{7-2-7 (c)}$$

$r^2 = 0.72$

$t_{obs} = 2.80$

And for soluble COD

$$\lambda'_{rAf} = 0.53\ \lambda_i + 22 \quad \ldots\ldots \text{7-2-8 (a)}$$

$r^2 = 0.97$

$t_{obs} = 12.16$

$$\ell n\ \lambda'_{rAf} = (5.441\times10^{-3})\ \lambda'_i + 3.75 \quad \ldots\ldots\ldots \text{7-2-8 (b)}$$

$r^2 = 0.91$

$t_{obs} = 5.27$

$$\ell n\ \lambda'_{rAf} = 0.779\ \ell n\ \lambda'_i + 0.84 \quad \ldots\ldots\ldots \text{7-2-8 (c)}$$

$r^2 = 0.98$

$t_{obs} = 12.16$

$$\lambda'_{rEf} = 0.63\ \lambda'_i + 20 \quad \ldots\ldots\ldots \text{7-2-9 (a)}$$

$r^2 = 0.99$

$t_{obs} = 12.16$

$$\ell n\ \lambda'_{rEf} = (5.546\times10^{-3})\ \lambda'_i + 3.79 \quad \ldots\ldots\ldots \text{7-2-9 (b)}$$

$r^2 = 0.91$

$t_{obs} = 5.27$

$$\ell n\ \lambda'_{rEf} = 0.798\ \ell n\ \lambda'_{i} + 0.804 \qquad \ldots\ldots 7\text{-}2\text{-}9(e)$$

$$r^2 = 0.99$$

$$t_{obs} = 12.16$$

where:

λ and λ' refer to the areal BOD and COD loadings respectively (kg/ha-day); the subscripts, i and r are for the influent and removal rates respectively for either BOD or COD;

u and f refer to the unfiltered (total) and filtered (soluble) respectively for the organic matters; while A and E are referring to the pond A (plain pond) and pond E (15 Baffle Pond) respectively.

The regression equations which are based on organic loading rates varying from 33kg BOD/ha-day to 200kg BOD/ha-day are comparable with those developed by previous investigators (Mara, 1974; Arthur, 1981 and Kilani, 1985).

$$\lambda_r = 0.73\ \lambda_i + 25 \quad \text{(Arthur)}$$

$$\lambda_r = 0.79\ \lambda_i + 2 \quad \text{(Mara)}$$

$$\lambda'_r = 0.917\ \lambda'_i - 12.56 \quad \text{(Kilani)}$$

for the unfiltered (total) COD, and

$$\lambda'_r = 0.971\ \lambda'_i - 11.36$$

for filtered (soluble) COD.

Usually facultative ponds would give higher BOD removal than COD removal. This was the trend as illustrated by equations 7-2-2(a) with 7-2-6(a), 7-2-3(a) with 7-2-7(a), and 7-2-5(a)

with 7-2-9(a). Although the percentage total BOD removal, equations 7-2-2(a) and 7-2-3(a) are lower than those predicted by previous investigators (McGarry and Pescod, 1970; Mara, 1974 and Arthur, 1981)

$\lambda_r = 0.725\ \lambda_i + 10.75$ (McGarry and Pescod) .... 7-2-13(a)

$\lambda_r = 0.79\ \lambda_i + 2.00$ (Mara) .... 7-2-13(b)

$\lambda_r = 0.73\ \lambda_i + 25.00$ (Arthur) ..... 7-2-13(c)

However, those of the soluble BOD, equation 7-2-4(a) and 7-2-5(a) are quite above 75% (about 80% in Pond A and over 85% in Pond E). If the influence of the algae is minimized as in the soluble portion of the waste, the efficiency obtained are quite above those reported by Arthur, Mara, and McGarry and Pescod.

Equations 7-2-2(a) through 7-2-9(c) gave very strong correlation between the removal rates and the loading rates except with those of the total COD. With COD loading and removal correlation the coefficient of determination r^2 was as low as 0.41 with a corresponding $t_{obs.}$ much less than 3.182. The highest r^2 was given by $\log_e - \log_e$ regression of Pond E effluent total COD. These values of r^2 and t_{obs} are 0.72 and 2.80 respectively.

It therefore seems that the relationship given by equations 7-2-6(a) through 7-2-7(c) can not be relied upon. Even though the symmetry between the rates, λ'_r and the loading rates, λ'_i given by the correlation coefficient, r may be accepted, there is very little prediction one can make with the relationships.

Only the linear and $\log_e$ - $\log_e$ relationships seem to be the most representative of the trend of the events in the ponds (equations 7-2-6(c) and 7-2-7(c).

The trend of increasing removal rate with increasing organic loading rate probably suggests that the contributory effect of algal content of the ponds and the resuspension of the pond septage greatly outweighed that due to pure synthetic milk waste alone. It can be concluded therefore that the total COD is not entirely from the synthetic milk wastes but probably from the solids especially the algae inside the ponds.

7.2.4 Relationship Between Effluent BOD, COD and SS

Effluent BOD and SS are common criteria for judging the performance of facultative ponds. One of the criticisms of the performance of facultative ponds is the high contribution of suspended solids in their effluents. As it was observed in this study, algae concentration increased with increased organic loads and hence contribute the bulk of the SS in the ponds' effluents. It seems therefore desirable to quantify the contribution of SS to both the BOD and COD in the effluents. It is also desirable to correlate the effluent BOD with COD. All these factors will obviously influence the degree of upgrading effluents from facultative ponds where necessary.

The regressions of the organic loads in the effluent against SS and both BOD and COD (Figures 6-2-7 through 6-2-17) give the following equations:-

For unfiltered (Total) COD, COD_u

Pond A: $COD_u = 4.55\ SS + 33.05$ ..... 7-2-14(a)

$r^2 = 0.96$

$t_{obs} = 8.53$

Pond E: $COD_u = 4.65\ SS + 17.29$ ..... 7-2-14(b)

$r^2 = 1.0$

For Filtered (Soluble) COD, COD_f

Pond A: $COD_f = 2.41\ SS + 51.04$ ..... 7-2-15(a)

$r^2 = 0.92$

$t_{obs} = 5.94$

Pond E: $COD_f = 2.36\ SS + 37.2$ ..... 7-2-15(b)

$r^2 = 0.96$

$t_{obs} = 8.53$

For Non-filtrable COD

Pond A: $(COD_u - COD_f) = 2.14\ SS - 18.0$ ..... 7-2-16(a)

$r^2 = 0.98$

$t_{obs} = 12.16$

Pond E: $(COD_u - COD_f) = 2.29\ SS - 20.43$ ..... 7-2-16(b)

$r^2 = 0.98$

$t_{obs} = 12.16$

For Unfiltered (Total) BOD, BOD_u

Pond A: $BOD_u = 1.98\ SS + 25.99$ ..... 7-2-17(a)

$r^2 = 0.94$

$t_{obs} = 6.91$

Pond E: $BOD_u = 1.47\ SS + 36.89$ .... 7-2-17(b)

$r^2 = 0.88$

$t_{obs} = 4.77$

For Filtered (Soluble) BOD, BOD_f

Pond A: $BOD_f = 1.58\ SS + 9.07$ ..... 7-2-18(a)

$r^2 = 0.98$

$t_{obs} = 12.16$

Pond E: $BOD_f = 1.09\ SS + 0.94$ ..... 7-2-18(b)

$r^2 = 0.88$

$t_{obs} = 4.77$

For the Non-Filtrable BOD

Pond A: $(BOD_u - BOD_f) = 0.40\ SS + 16.79$ ..... 7-2-19(a)

$r^2 = 0.81$

$t_{obs} = 3.58$

Pond E: $(BOD_u - BOD_f) = 0.38\ SS + 36.04$ ..... 7-2-19(b)

$r^2 = 0.34$

$t_{obs} = 1.27$

For Total Non-biodegradable oxygen demand

Pond A: $(COD_u - BOD_u) = 2.56\ SS + 7.06$ ..... 7-2-20(a)

$r^2 = 0.91$

$t_{obs} = 8.49$

Pond E: $(COD_u - BOD_u) = 3.18\ SS - 19.60$ ..... 7-2-20(b)

$r^2 = 0.98$

$t_{obs} = 12.16$

For Soluble non-biodegradable oxygen demand

Pond A: $(COD_f - BOD_f) = 0.83\ SS + 41.91$ ..... 7-2-21(a)

$r^2 = 0.81$

$t_{obs} = 3.58$

Pond E: $(COD_f - BOD_f) = 1.27\ SS + 36.82$ ..... 7-2-21(b)

$r^2 = 0.88$

$t_{obs} = 4.77$

For Unfiltered (Total) BOD, BOD_u and COD, COD_u regression:

Pond A: $COD_u = 2.26\ BOD_u - 20.83$ ..... 7-2-22(a)

$r^2 = 1.0$

$t_{obs} = \infty$

Pond E: $COD_u = 2.83\ BOD_u + 43.91$ ..... 7-2-22(b)

$r^2 = 0.96$

$t_{obs} = 8.49$

For Filtered (Soluble) BOD, BOD_f and COD, COD_f regression:

Pond A: $COD_f = 1.55\ BOD_f + 33.13$ ..... 7-2-23(a)

$r^2 = 0.98$

$t_{obs} = 12.16$

Pond E: $COD_f = 4.43\ BOD_f - 59.24$ ..... 7-2-23(b)

$r^2 = 0.96$

$t_{obs} = 8.49$

For Non-Soluble Organics

Pond A: $(COD_u - COD_f) = 4.43\ (BOD_u - BOD_f) - 59.24$ .... 7-2-24(a)

$r^2 = 0.87$

$t_{obs} = 4.38$

Pond E: $(COD_u - COD_f) = 1.66\ (BOD_u - BOD_f) + 67.19$ ..... 7-2-24(b)

Effluent SS could also be influenced by two factors, namely dead algal cells reaching the effluent and the anaerobic condition at the bottom of the ponds resulting in resuspension of more solids some of which may find their way to the effluent. It is obvious from the results obtained from this study that both effluent SS and COD especially total COD varied erratically and were the most difficult parameters to predict.

From equations 7-2-14 through 7-2-16, the differences in COD in the two ponds as a function of SS are not appreciable. However, in terms of BOD, as much as 68% of the total BOD is due to SS in pond E whereas only 51% of the total BOD is due to SS in Pond A (equation 7-2-17). These observations are in order since most of the algae would register as SS and one has more of these algae in Pond E than in Pond A. This 51% for Pond A is within the range of 50 - 62% reported by Bradley on a full scale pilot plant in Malaysia (Bradley, 1983). In Pond E, the 68% is somehow also satisfactory.

Some of the algae are filtrable. In the filtrable portion 92% of the soluble BOD is due to SS in Pond E as against 63% in Pond A (Equation 7-2-18). The reason for this difference is perhaps due to the algal contents of the two ponds. The same trend was observed in the unfiltrable portion of the waste. There are more BOD in this component in pond E than in Pond A (equation 7-2-19).

Care must be taken when interpreting experimental data. In this study for instance, the intercepts either positive or negative indicating that a difference could be observed in the parameters of the two axis are not applicable at very low concentrations. For instance the negative values of non-filtrable COD when SS is zero in Equation 7-2-16 are not reasonable. For the equations to be valid at low concentrations their graphs would be expected to pass through the origin (Figures 6-2-7 through 6-2-13).

Algae seems to contribute more to non-biodegradable matters than to biodegradable ones (Equations 7-2-20 and 7-2-21). It seems therefore that next to the problem of SS is COD and then BOD due to algae in stabilization ponds. This is in line with previous studies (Kilani, 1985).

More materials especially non-biodegradable ones are as SS in Pond A than Pond E. This trend of higher non-biodegradable content contributed by the algae may be responsible for the relative stronger effluent in Pond A (Equations 7-2-20 through 7-2-23). Generally, equations 7-2-19 and 7-2-24 are estimates of the amount of BOD and COD contributed by SS in the effluents respectively and only valid for the filtration technique adopted in this study.

7.2.5 BOD Removal Rate Constant, K

The BOD removal rate coefficient, K depends on the biological environmental factors while the values of Kt (product of BOD removal rate and detention time)are influenced by the ponds configuration, biological and hydraulic properties in the pond. In the present study, the values of both K and Kt were estimated at various organic loadings between 33 and 200kg BOD/ha-day and using the dispersed flow model (Equation 4-1-23) while keeping the detention time constant at 15 days and at an average temperature of 24°C (Room Temperature).

Tables 6-2-1 shows the calculated values of K for both baffled and plain ponds. Also Figure 6-2-18, Equations 7-2-25 and 7-2-26 show the relationships between Kt and BOD removal rate, λ_r. From these results, it is observed that Kt increases with decrease in organic removal. This is quite obvious since removal increases with areal loading. Theoretically when the organic loading increases the reaction rate is always retarded.

This follows the same trend as those found by previous investigators (Thirumurthi, 1969 and Arthur, 1981).

Taking the filtered BOD for Ponds A and E for instance, based on least squares method the relationship between Kt and areal organic removal, λ_r can be approximated as:

For Pond A (Plain Pond)

$$Kt = 4.42 - 0.02\lambda_r \quad \text{..... 7-2-25}$$

$$r^2 = 0.92$$

For Pond E (Baffled Pond)

$$Kt = 4.06 - 0.02\,\lambda_r \quad \text{..... 7-2-26}$$

$$r^2 = 0.93$$

7.3.0 Changes in Dissolved Oxygen, Alkalinity and pH

Table 6-2-1 shows the changes in DO, Alkalinity and the pH of the ponds' influents and effluents. Also, the average diurnal variations in these parameters except alkalinity are shown in Figures 6-2-6(a) through 6-2-6(e). From the results it is observed that there were some slight variations in these parameters of the influent feeds. As it was explained before, preparing the feed every other day probably helped to minimize the wide fluctuations in these parameters.

As expected, there was a considerable variation in the values of the parameters for the effluent samples. The observed fluctuation are probably influenced by the variation in the organic loadings and microbial activities.

The most significant changes in these parameters are the diurnal variation . It is evident that diurnal changes depend on algal photosynthetic activity which also is a light dependent process. During the day time, photosynthetic algae utilize for growth CO_2 and the inorganic materials generated by bacterial action and in the process oxygen is liberated as an end product (Equation 2-1-4). This oxygen plus that obtained from surface aeration make up the total oxygen available for the bacterial breakdown of organic matter. Thus the DO and pH increase after sunrise and stay high during the middle of the day. At sunset photosynthesis ceases, alkalinity and CO_2 accumulate and the residual DO is utilized by both bacteria and algae (endogeneous respiration). Consequently the DO and pH decrease during the sunset and remain low in the early hour of the morning before sunrise while the alkalinity increases.

For the effluents the pH and DO in Pond E were always found to be higher than those in Pond A; (Figures 6-2-1(d) through 6-2-5(d)). An increase in DO usually accompanied high pH probably because algae extract dissolved carbon dioxide from water causing a shift in the carbonate - bicarbonate equilibrium and increasing the pH. The lowest mean pH - DO occured in both Ponds A and F at an organic loading of 100kg BOD/ha-day (Figures 6-2-3(d)). Because of the nature of the waste the decomposition of the waste increases with the concentration of the milk. This follows the trend of previous studies (Aluko, 1976 and Kilani, 1985).

Influence of mixing played an important role over the DO and Alkalinity. In Pond A, the DO appeared to be decreasing with increasing organic loading. Due to good mixing in Pond E both DO and alkalinity increaeed with the organic loading. The DO range in Pond E was between 2.08 and 3.78mg/L whereas a low range of as low as 1.95 to 2.9 was recorded in Pond A. Under aerobic conditions metabolism of organic matter results in an increase in both pH and alkalinity as the amino groups in the proteins are released as ammonia.

The observation in Pond A may be due to poor mixing and when one recalls that there is more dead volume in Pond A than in Pond E, the dead volume would behave like anaerobic system especially with increased organic loading. The percentage dead volume in Pond A ranged from 43 to as high as 79 percent compared to that in Pond E which never exceeded about 41 percent (Table 6-1-1) and (Figure 6-1-11). Under anaerobic metabolism the production of organic acid would depress the pH.

The diurnal cycle and oxygenation of the ponds were noticed to be influenced much more probably by variation in algal population than species except when the algal population was heterogenous as in the lower organic loadings. As some species gave way to more shock resistant species like Euglena the system would become more homogenous and hence population would be the deciding factors.

Diurnal fluctuations were less pronounced in Pond A in all the loadings. At lower organic loading the influence of algae is less. In Pond A the algal concentration is relatively lower than that in Pond E; this influence of algae may be responsible for this observation in diurnal variation. Also the decrease may depend probably upon alkalinity or buffer capacity of the waste.

7.4.0 Biological Features

7.4.1 Bacterial Population

The synthetic milk waste was presumed free of any bacteria. If there are any at all, they should be from the tap water used for diluting the milk and the water inside the Ponds. In view of this the bacterial population initially was considered very low. However, during the start up of each run, the ponds were seeded with sewage containing a viable population of bacteria; it is desirable to investigate their die-off rates in the ponds. The reductions were measured in terms of Total Coliform (TC); Faecal Coliform (FC) and Faecal Streptococci (FS). Table 6-3-1 shows the percentage die-off corresponding to the variations in organic loadings.

From these results it is observed that for all the ponds at a fixed detention time of 15 days, the die-off rates decreased with increase in organic loadings. This is in agreement with the findings of a previous study (Arthur, 1981). However, it is observed that the baffled pond (Pond E) performed better than the plain pond (Pond A).

Also it is observed that the percentage bacterial removal follows the same trend as Kt. Regressions analysis of Kt with the efficiency of bacterial removal, η gave the following equations:

For Pond A

TC : $Kt = 0.11\eta - 6.49$ 7-4-1

$r^2 = 1.00$

FC : $Kt = 0.25\eta - 17.98$ 7-4-2

$r^2 = 0.98$

FS : $Kt = 0.24\eta - 16.61$ 7-4-3

$r^2 = 0.96$

For Pond E

TC : $Kt = 0.11\eta - 6.96$ 7-4-4

$r^2 = 0.96$

FC : $Kt = 0.25\eta - 18.66$ 7-4-5

$r^2 = 0.90$

FS : $Kt = 0.37\eta - 28.40$ 7-4-6

$r^2 = 0.74$

The negative intercepts in the above regression equations have no physical interpretation. The rate of bacterial removal, K increases with the efficiency, η.

7.4.2 Algal Population

Table 6-3-1 shows the estimated algal population in the ponds in terms of microscopic counts using equation 7-4-7.

$$N = 100 \frac{n}{a} Ad^4 \quad \text{..... 7-4-7}$$

where:

N = number of cells (algae) per 100 mL

n = number of cells

a = the number of squares occupied by n number of cells.

A = total number of squares = 10^3, and

d* = dilution factor

It is observed that algal production is influenced by the organic loadings. Increasing the organic loadings from about 33 kg BOD/ha-day to about 200kg BOD/ha-day resulted in an increased algal production. Also it is observed that under the same loading and environmental conditions the baffled pond produced higher concentration of algae than the plain pond. But inspite of higher concentration of algae the baffled pond still performed better than the plain pond in terms of organic and SS removal. It is believed that baffles acted as barriers preventing these waste to reaching the effluents.

The identification of the algae showed that the following species were found in the ponds: the blue green algae, - Anacystics, Oscillatoria and Arthrostira; flagellate algae - Euglena and Chlamydomona; the filamentous green algae - Spirogyra; colloid green algae - Gloecystis and Chlorella and then diatoms - Diatom Vulgare. The Euglena were predominant followed by Anacystics. The Chlorella were found at the initial stage of the ponds' operation and usually at the influent of the ponds.

Recent analyses of stabilization pond effluents have indicated that effluent algae carry-over can impose a significant oxygen demand on receiving waters (King et al., 1970). Theoretical calculations of the oxygen demand created by algae destruction suggests that 1.58 mg of oxygen are required to oxidize 1 mg of algae (dry weight) to CO_2 and H_2O. Bare, et al. found that 1.11 mg of oxygen, O_2 was required per mg of algae obtained from laboratory studies and 0.81 mg O_2 per mg of algae obtained from field studies (Bare, et al., 1975).

Varma and Digiano reported that 0.67 mg O_2 was required for each mg of algae destroyed (Varma and Digiano, 1968). And a value of 1.19 mg O_2 per mg of algae was reported by Friedman et al (Friedman, et al., 1975).

Despite the cited differences in O_2 demand per unit weight of algae one can conclude that a significant O_2 demand is imposed on receiving water by algae discharged from stabilization ponds' effluents. Only the removal or destruction of algae prior to discharge can sufficiently improve effluent quality to an acceptable level. Some of the current control methods of algae are chemical coagulation and flocculation followed by sedimentation or floatation (Ives, 1956 and Tenney et al., 1969).

The small size of algae 3-15 micron and low specific gravity further complicated physical removal processes. In this study destabilization of algae was accomplished by the use of Chloroform. Lime, allum, feric sulphate, magnessium ions and many synthetic organic polyelectrolytes had been used to

destabilize algae in suspension as well (McGarry and . Pescod 1970; Caldwell et al., 1973; Folkman 1973; and Bare et. al., 1975 and Anyam, 1984).

As might be expected, the effect of each coagulant is dependent on pH, algal population and a few other parameters. For instance it was detected in this study that chloroform always react with plastic hence the use of chloroform especially in laboratory pond made of plastic may not be encouraged.

CHAPTER 8

8.0 SUMMARY, CONCLUSION AND RECOMMENDATIONS

8.1.0 Summary

The study was in two parts. In the first part the hydraulic behaviour of five different rectangular ponds were studied under four different hydraulic loadings of 6.2, 8.3, 11.2 and 17.8 L/day using tracer. It was found that the pond with the greatest number of baffles under 8.3 L/day gave the optimum efficiency. The second part of the study was based upon the conclusion of the first part in selecting the ponds for the model pond treatment performance. The pond with the lowest efficiency (that is the plain pond, Pond A) and that with the highest efficiency (that is the pond with 15 baffles, Pond E) under the same hydraulic loadings were examined under five different areal organic loadings of 33.3, 50, 100, 150 and 200 kg BOD/ha-day.

Pond E with 15 baffles performances under various areal organic loadings were much superior to that of Pond A without baffles. In order for Pond A to have the same efficiency with Pond E, there must be about 2.32 of such plain pond working in series. The results from both tests compare favourably with previous studies.

8.2.0 Conclusion

Tracer test is useful in evaluating the hydraulic property of a reactor. The more the deviation of a real pond from the ideal, the lower the efficiency. As the dispersion number, d tends to zero, the system tends towards plug flow regime.

The system is more realistic under a steady state condition with operation period (nominal detention time) at least equal to the mean (actual) detention time.

Better mixing is enhanced by baffles. The degree of mixing increases with number of baffles.

In addition to O_2 supplement afforded by the algae and bacteria, they can also be used as pollutional indicator.

While animal such as Daphnia are important in clarification step, both algae and bacteria are important in purification step.

The regression analyses show that algae and SS have great effect on stabilization pond efficiency.

Both biodegradable and non-biodegradable O_2 demand due to filtrable and/or non-filtrable solids (SS) are positively and strongly related.

Baffled ponds provide higher efficiency than unbaffled ones of the same size and shape for BOD, COD and bacteria die-off.

Efficiency is a function of length of travel or actual detention time, t_g.

Length of travel or the detention time (actual) is strongly related to the dispersion number, d. The higher the length or time the lower the dispersion number, d and then the higher the efficiency.

Reaction rate decreases with efficiency. A more accurate prediction of a stabilization pond performance can be made with this model than all the methods so far being used in stabilization pond design.

Application of chemical and microbiological principles should enhance the improvement of stabilization pond performance.

8.3.0 Recommendations

Very often during the course of an attempt to investigate a problem, several other problems always emerge. Since no single study can find answers to all such problems the following areas closely related to this present study should be investigated further:

- the influence of hydraulic loading on dispersion and tracer recovery;
- optimum number of baffles in a rectangular pond;
- optimum spacing to length of the tank ratio;
- recirculation effect on the pond performances;
- identification of bacterial and algal species as a function of organic surface loading;
- quantitative oxygen contribution to stabilization ponds by algae ;
- algal population control in stabilization ponds;
- verification of germicidal or phacocidal effects of light on both bacterial and algae;
- the use of effluent and algal harvesting for protein substitute, and

- application of real domestic sewage and field investigation of the model.

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APPENDIX A

COMPUTER PROGRAM FOR SOLVING WEHNER AND WILHEM EQUATION

TANK A:

```
1Ø    LET D  =  Ø.151
15    L PRINT "D  =  Ø.151"
16    L PRINT
2Ø    LET A  =  1.Ø
3Ø    LET AA  =  (A+2)-1
4Ø    LET BB  =  6Ø * D
5Ø    LET  K  =  AA/BB
6Ø    LET  EE  =  EXP (1-A)/(2 * D)
7Ø    LET  A1  =  1+A
8Ø    LET  CC1  =  (4*A*EE)/(A1+2)
85    LET CC  =  (1 - CC1) * 1ØØ
9Ø    L PRINT A, K, CC
91    L PRINT
1ØØ   LET  A  =  A+Ø.1
11Ø   IF (A = 4.3) THEN GO TO 2ØØ
12Ø   GO TO 3Ø
2ØØ   STOP.
```

TANK E

```
1Ø    LET D  =  Ø.Ø3
15    L PRINT "D = Ø.Ø3"
16    L PRINT
2Ø    LET  A = 1.Ø
```

```
3Ø    LET  AA = (A+2)-1
4Ø    LET  BB = 6Ø * D
5Ø    LET  K = AA/BB
6Ø    LET EE = EXP (1-A)/(2 * D)
7Ø    LET A1  =  1+A
8Ø    LET CC1 = (4 * A * EE)/(A1+2)
85    LET CC = (1-CC1) * 1ØØ
9Ø    L PRINT  A, K, CC
91    L PRINT
1ØØ   LET A = A+Ø.1
11Ø   IF (A = 4.3) THEN GO TO 2ØØ
12Ø   GO TO 3Ø
2ØØ   STOP
```

APPENDIX B

TRACER TEST AND ESTIMATION OF RELATED PARAMETERS

The two most convenient types of tracer input that can be used are the step and impulse signals. Unit impulse function is a derivative of unit step function (Grady and Lim 1980).

$$\delta(t-T) = \frac{d}{dt}\,\delta(t-T) \qquad \ldots\ldots\ldots \text{B-1}$$

This impulse function or the Dirac delta is defined by:

$$\delta(t-T) = 0, \quad t \neq T \qquad \ldots\ldots\ldots \text{B-2}$$

$$\int_{-\infty}^{\infty} \delta(t-T)dt = 1 \qquad \ldots\ldots \text{B-3}$$

This represents a mathematical idealization of a sudden "jolt" of input to the reactor, and the jolt occurs instantaneously in zero time.

There are many ways of estimating the degree of deviation of a flow from an ideal one, but most rely on experimental measurement of the residence time distribution (RTD). This is the time taken by a molecule to pass through a reactor. Fluid element taking different routes requires different lengths of time.

Suppose the residence time of fluid elements in a reactor is t, then the function, F(t) is the fraction of the elements in the effluent stream having residence times less than t. This implies that, $F(0) = F(\infty) = 1$, indicating that none of the fluid can pass through the vessel in zero time and all will eventually come out. This is the cummulative distribution curve function or F-curve. Another function is the point distribution function or E-curve. This is the RTD function of the fluid. It is related to the F-curve by:

$$E(t) = F(t)/dt \qquad \ldots\ldots\ldots \text{B-4}$$

Hence E(t) is the function of the effluent that has residence times between t and t + dt. The area under the E-curve between 0 and ∞ is unity.

$$E(t)dt = E(\infty) = 1 \qquad \text{... B-5}$$

while the fraction of the fluid element that has residence times less than t is given by:

$$F(t) = \int_0^{t_1} E(t)dt \qquad \text{...B-6}$$

The space time of a reactor may be determined experimentally. Assuming a constant density of the moving fluid in the vessel with isothermal conditions, the mean residence time, $\bar{t}$, can be expresses as:

$$\bar{t} = \int_0^1 td\,(F(t)/\int_0^1 dF(t) = \int_0^\infty tE(t)dt/\int_0^\infty E(t)dt = \int_0^\infty tE(T)dt \quad \text{..B-7}$$

Although equation B-7 gives the mean residence time, the space time and space velocity concept seems more descriptive.

The space time, $V/F = \tau = \bar{t}$ .. B-7(a)

where:

V = reactor volume, L

F = the feed rate L/hour

This is the time required to process one reactor volume of feed. It is equivalent to reaction time for a batch reactor. Sometimes the term mean residence time is also used. This is the average time the fluid spends in the reactor before it leaves the reactor. When the reaction fluid density is constant, then the mean residence time is equal to the space time. Only constant density situations are considered in this study.

Space velocity, F/V is the reciprocal of space time. This is the number of reactor volumes treated by the reactor in a unit time. In Biochemical Engineering it is commonly called the dillution rate.

Since,

$$E(t) = dF(t)/dt \qquad \ldots B\text{-}4$$

and the impulse input is equal to the derivative of the step input response, both responses can be related thus:

$$E(t) = dF(t)/dt = d(c(t)/c_o)_{step} = (C(t)/C_o)_{impulse} \qquad ..B\text{-}8$$

Or,

$$E(t) = C(t)_{impulse}/\int_o^\infty C(t)dt \qquad ....B\text{-}8$$

This indicates that the F(t) function can be obtained from the impulse input response by integration just as E(t) function can be obtained from step input by differentiation. As such either step input or impulse input may be used to obtain both E(t) and F(t). However, it is safer to avoid differentiation of experimental data whenever possible, hence the use of an impulse in this study as in most other studies. Although having obtained the E(t) function from an impulse input, one can integrate

to obtain F(t) depending upon the requirement.

The total reaction time of the tracer introduced can be estimated by operating the integral with Simpson's rule (Grady and Lim 1980).

$$t = \int_{C_o}^{C_f} \frac{dC}{-r} = \Sigma \left(\frac{\Delta i}{6} \left(\frac{1}{-r\,(C_1)} + \frac{4}{-r\,(C_2)} + \frac{1}{-r(C_n)} \right) i \right. \quad \text{....B-10}$$

$$t = \frac{\Delta}{6} (f_o + 4f_1 + 2F_2 + 4f_3 + 2f_4 + \ldots + 4f_{j-1} + f_j) \quad \text{..... B-11}$$

where:

$$f_o = \frac{1}{-r\,(C_1)}$$

And the total amount of tracer introduced into the tank is (Figure B-1):

$$\int_o^\infty C\,(t)dt_{impulse} = C(t)_{impulse}/E\,(t) \quad \text{.....B-12}$$

Using Simpson's rule,

$$\int_o^\infty C(t)_{impulse}\,dt = \frac{\Delta}{6} (C_o + 4C_1 + 2C_2 + 4C_3 + 2C_4 + .. 4C_{n-1} + C_n)$$

.....B-13

where:

$$C = \frac{1}{-r(C_i)}$$

An interval for Simpson's rule is twice the length of the subinterval used. Whatever may be the case, the RTD functions are not unique and a number of flow patterns can give the same one. This is a major set-back in the application of RTD in predicting the performance of actual reactors except in the lst order reaction where the conversion is indepent of the flow pattern.

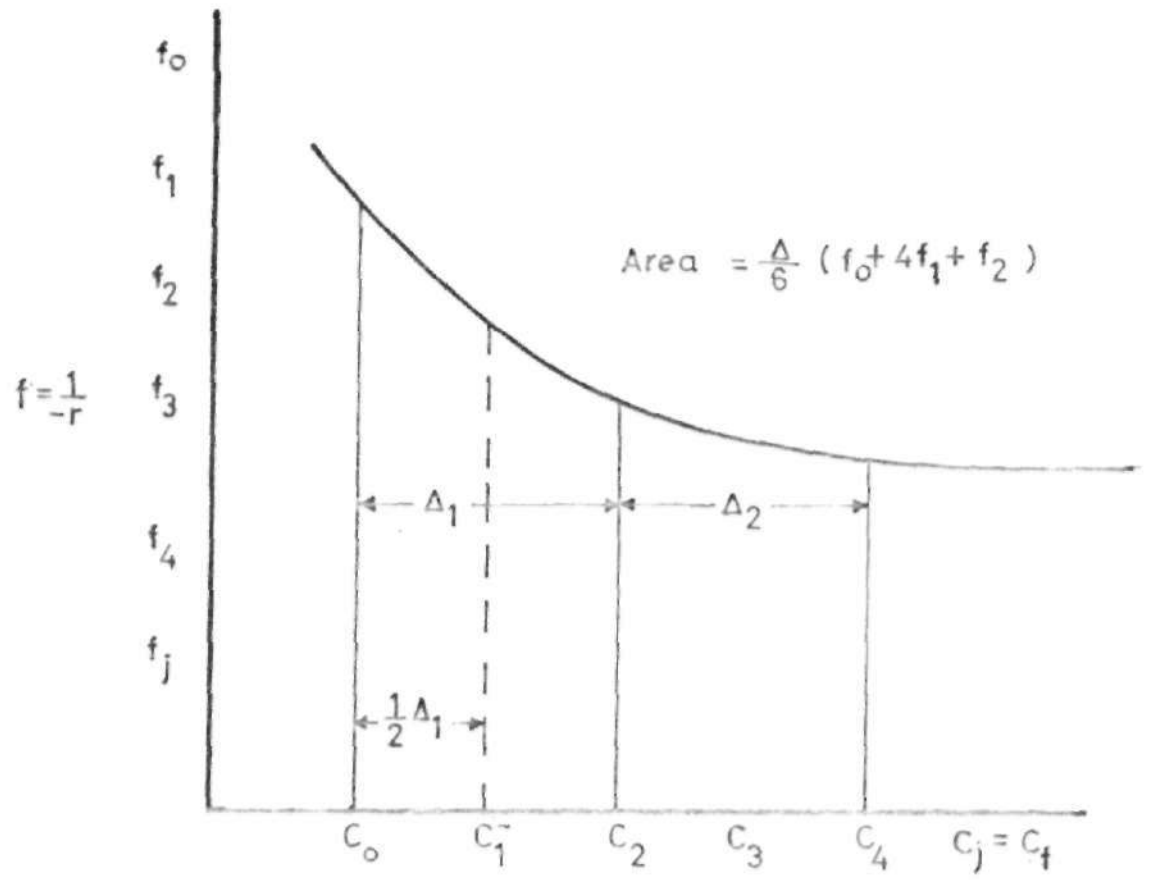


FIG B-1: SIMPSONS RULE FOR EVALUATING AN INTEGRAL

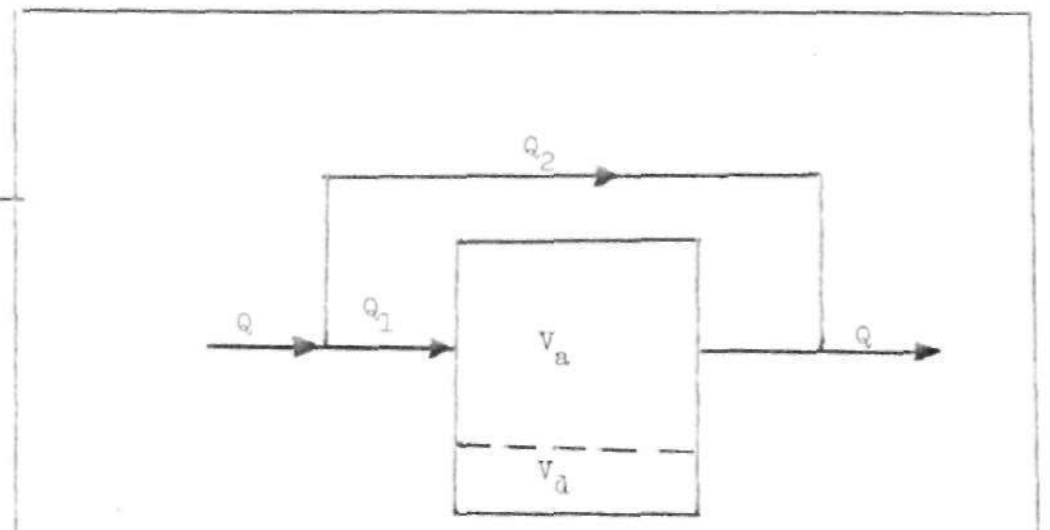


FIG. B-2: FLOW CHART OF SUBSTRATE SHORT CIRCUITTING

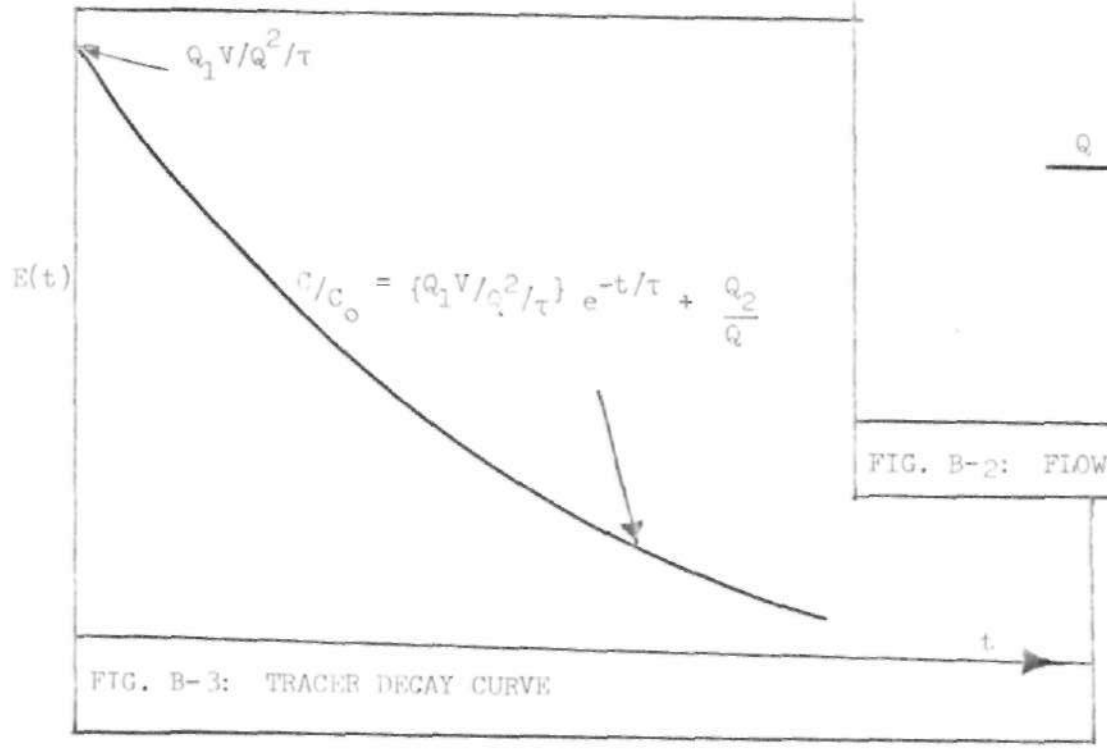


FIG. B-3: TRACER DECAY CURVE

If there are dead spaces in the tank, the effective tank volume is less than the true volume (Figures B-2 and B-3).

$$Ce/Ci = Q_1 \, V/Q^2/\tau_{obs} \quad e^{-} \, t/\tau_{obs} + Q_2/Q \, \delta'(t=0) \quad .....B\text{-}14$$

Without short circuiting the relation is:

$$Ce/Ci = V/Va \; e^{-tv/va/\tau_{obs}} \quad .....B\text{-}15$$

Ci = Initial theoretical tracer concentration assuming total volume utilization.

Ce = tracer concentration at time, t

V = nominal tank volume

Va = Tank Actual mixed volume

Q = total flow rate

Q_1 = fraction of Q entering active mixed zone

Q_2 = fraction of Q short-circuiting

τ_{obs} = mean hydraulic retention time observed in actively mixed zone.

δ' = Dirac delta function (equation (B-2)).

$V - V_a = V_d$ = The dead space. This is the region of the tank that will retain tracer sludge for periods of an order of magnitude greater than the mean retention time of the total system.

The flow through curve (C-curve) for a group of real tanks is shown in Figure B-4 with dimensionless coordinates C_e/C_i and t_g/t_n.

Where;

C_e = is tracer concentration at the effluent after time, t.

C_i = total weight of tracer added at the influent divided by the volume of the tank, i.e the concentration if the dose is dispersed instantaneously throughout the tank volume.

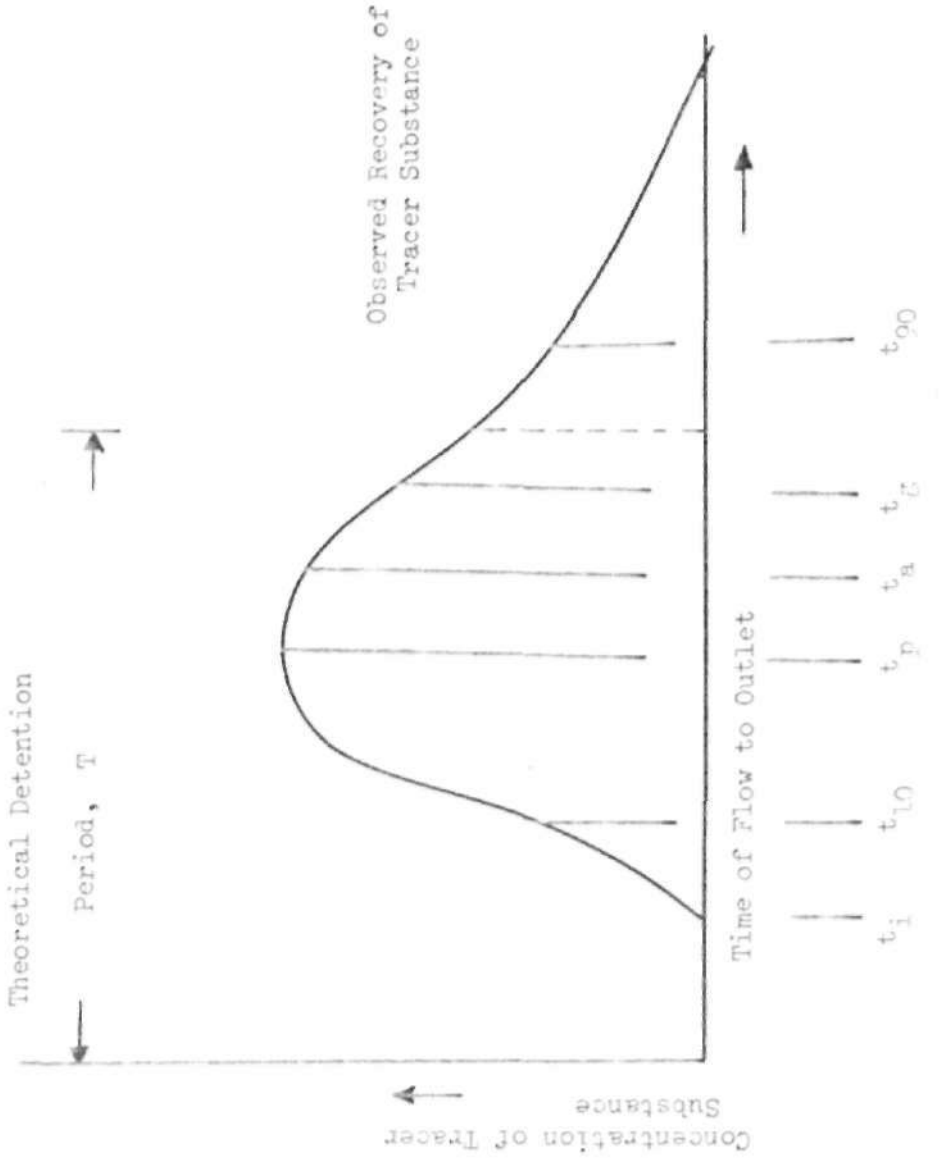


FIGURE B-4: DIMENSIONLESS C-CURVE

t_n = theoretical retention time.

This plot is independent of the amount of tracer used and the retention time. For a stable flow and without dead spaces the value of t/t_n to the center of gravity will be unity. Unstable condition would give unreproducible curve shapes whereas a stable flow pattern is indicated when the same curve can be reproduced with fair precision. Moreover, unstable condition will yield undependable short circuiting study results and erratic suspended solid removal performance.

t_i = time interval for the initial indication of the tracer in the effluent

t_p = time for effluent tracer to reach peak concentration

t_{10}, t_{50}, t_{90} = Time for 10%, 50% and 90% of the tracer to be observed at the effluent

$t_a = t_{50}$ ≡ median time or time at which 50% of the tracer is observed at the effluent.

$t_g = \frac{\Sigma ct}{\Sigma c}$ ≡ Time to reach the centroid of the curve or mean detention time (Actual).

$t_n = V/Q$ ≡ Theoretical (Nominal) detention time

$\frac{t_g - t_p}{t_g}$ ≡ Index of short circuiting.

The effective retention period will be less than the true retention period and the relative time to the center of gravity, of the dispersion curve, t_g/t_n will be less than unity if there is any interchange between dead spaces and the main body of the liquid, there will be flow through the dead spaces, but the resulting "tail" of the dispersion curves may be so long and the ordinates may be so small that the end of the tail will be lost and as apparent, value of t_g/t_n less than unity will result.

If there are dead spaces the recorded values of Ce/Ci will be greater than the effective values and all ordinates to the dispersion curve will be increased in proportion to the decrease in t_g/t_n such that the area will remain unity.

Since the relative time to the center of gravity is unity except for tanks with dead spaces, t_g/t_n is an indicator of amount of the dead space, but it is not a suitable index of short-circuiting with tanks without dead spaces. The relative time to the center of area, t_g/t_n is usually less than unity and the smaller the worse the short circuiting. Since half the particle of fluid pass in less time, t_a is the "probable flowing through time" of the fluid. The dispersion of the fluid is measured approximately by the time of initial appearance, t_i of the change in the effluent. The smaller the relative time t_i/t_n the more rapid the dispersion. Small values of ti/t_n and ta/t_n indicate serious short circuiting.

Yet another good parameter for evaluating mixing pattern in the tank is the Morril Index (MI). As the MI approaches 1.0 the flow through the tank approaches ideal plug flow.

For this particular study as developed in this Appendix, the most appropriate model is the axial dispersion model. Suppose the following data in Table B-1 were compiled from a tracer test, the sampling time t would be given in Column 1 with the corresponding tracer concentration C in the third column. In the second column the sampling time increment, Δt would be listed with corresponding tracer concentration increment Δc in column 4. Δc would change sign from positive to negative whenever the concentration falls.

In the fifth column would be the values of the Y-function $(C-\frac{1}{2}\Delta C)\ \Delta t$, while in the sixth and seventh columns the Z-function, $Y/(t-\frac{1}{2}\Delta t)$ and W-function, Y^2/Z calculated values would be listed respectively.

Under column eight and column thirteen would be calculated values of λ (i.e.Wt) and β (i.e $\Sigma Wt(\Sigma Y)$ respectively. While in columns nine, ten, eleven and twelve would be the summations of Y, Z, W and λ respectively. The results of such operations were used for table 6-1-1.

In this table the parameters were evaluated.

t_s, that is the standard detention time is $\Sigma Y/\Sigma Z$.

Effective detention time t_e is the time during which both batch and continuous feed operation concentrations are equal.

This is given by:

$$t_e = t_s = \frac{\sum_{o}^{m} (C-\frac{1}{2}\Delta C)\ \Delta t}{\sum_{o}^{m} \frac{(C-\frac{1}{2}\Delta C)\ \Delta t}{t - \frac{1}{2}\Delta t}} \qquad \text{.....B-16}$$

Or, upon simplifying,

$$t_e = \Sigma Y/\Sigma Z \qquad \text{.....B-17}$$

Due to simplifying assumptions in B-16 especially its validity for only second order reaction flocculation thereby imposing some further limitations on its validity, it is called standard detention time rather than effective detention time.

If one divides t_s by the nominal detention time, t_n one obtains the standard detention efficiency which is a good measure of the detention effectiveness of the reactor, i.e

$$t_\eta = t_s/t_n$$

The actual (mean) detention time, t_g is approximately:

TABLE B-1: ESTIMATION OF TRACER TEST PARAMETERS

t (day)	Δt (day)	C Tracer Concentration (mg/L)	ΔC (mg/L)	Y $(C-\frac{1}{2}\Delta C)\Delta t$	Z $\frac{Y}{t-\frac{1}{2}\Delta t}$	W $\frac{X^2}{z}$	λ Wt	ΣY	ΣZ	ΣW	Σλ	β $\frac{\Sigma\lambda}{\Sigma Y}$
1	2	3	4	5	6	7	8	9	10	11	12	13
t_1	t_1	C_1	E_1	Y_1	Z_1	W_1	λ_1					
t_2	t_2-t_1	C_2	C_2-C_1	Y_2	Z_2	W_2	λ_2					
t_3	t_3-t_2	C_3	C_3-C_2	Y_3	z_3	W_3	λ_3	ΣY	Σz	Σw	Σλ	β
-	-	-	-	-	-	-	-					
t_n	t_n-t_{n-1}	C_n	C_n-C_{n-1}	Y_n	Z_n	W_n	λ_n					

$$\frac{\sum_{o}^{m} (C-\frac{1}{2}\Delta C)\,(t-\frac{1}{2}\Delta t)\Delta t}{\sum_{o}^{m} (C-\frac{1}{2}\Delta C)\,\Delta t} \qquad \text{..... B-18}$$

$$t_g = \Sigma tC/\Sigma C = \Sigma W/\Sigma Y \qquad \text{..... B-19}$$

t_g is also a useful measure of d and the active volume thereby permitting determination of unused stagnant volume in the tank.

δ, the fraction of deal space is given by:

$$\delta = (t_n - t_g)/t_n \qquad \text{.....B-20}$$

where

$t_n = V/Q$ = Nominal detention time.

For the index of short circuiting, α_s:

$$\alpha_s = t_i/t_n \ (=)\ t_g - t_p)/t_g \qquad \text{.....B-21}$$

where:

t_p = time of peak concentration, and (=) implies not necessarily numerically equal to.

Alternative relations for B-20 and B-21 are V_d/V and Q_2/Q respectively as given by B-14. But a more qualitative expression for δ is the percentage difference, ψ

$$\text{Percentage difference} = (t_n - t_g)/\tfrac{1}{2}(t_n + t_g) \qquad \text{.....B-22}$$

Out of all the parameters, the dispersion index d, seems to be the most quantitative and qualitative parameter. It has the strongest statistical probability of correctly describing the hydraulic performance.

$$d = \frac{1}{3}\left((3\sigma^2 + 1)^{\frac{1}{2}} - 1\right) \qquad \text{.....B-23}$$

$$\sigma^2 = \sigma_t^2 / t_g^2 \qquad \text{.....B-24}$$

$$\sigma_t^2 = \frac{t^2 C}{C} - \frac{tC}{C} \qquad \text{.....B-25}$$

$$\sigma_t^2 = \beta - t_g^2 \qquad \text{.....B-26}$$

where σ_t^2 is the variance of C-curve in "time squared" units;

σ^2 variance of C-curve, dimensionless; t, time, C the tracer concentration (mg/L) and $\beta = \Sigma\lambda / \Sigma Y$.

APPENDIX C EXPERIMENTAL DATA.

TABLE C-1-1: TRACER CONCENTRATION PERCENTAGE TRANSMITTANCE

CONCENTRATION (mg/L)	% Transmittance
00.0	100.00
00.50	99.00
01.50	98.00
05.00	95.00
10.00	91.00
15.00	86.00
20.00	83.00
25.00	78.00
30.00	73.00
35.00	70.00
40.00	65.00
45.00	61.00

TABLE C-1-2: TRACER TEST DATA (TANK A (7 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	W_t λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	15.00	15.00	7.50	15.00	3.75	3.75	
2	1.0	24.00	9.00	19.50	13.00	29.25	58.50	
3	1.0	20.00	-4.00	22.00	8.80	55.00	165.00	
4	1.0	17.00	-3.00	18.50	5.29	64.70	258.80	16.44
5	1.0	12.50	-4.50	14.75	3.28	66.33	331.65	
6	1.0	8.00	-4.50	10.25	1.86	56.49	338.94	
7	1.0	4.00	-4.00	6.00	0.92	39.13	273.91	
8	1.0	2.00	-2.00	3.00	0.40	22.50	180.00	
9	1.0	0.00	-2.00	1.00	0.12	8.33	74.97	

TABLE C-1-3: TRACER TEST DATA (TANK A (11 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(C-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	2.75	2.75	1.38	0.69	2.76	2.76	
2	1.0	6.00	3.25	4.38	2.92	6.57	13.14	
3	1.0	8.00	2.00	7.00	2.80	17.50	52.50	
4	1.0	6.50	-1.50	7.25	2.07	25.39	101.57	
5	1.0	5.00	-1.50	5.75	1.28	25.88	129.38	
6	1.0	4.50	-0.50	4.75	0.86	26.24	157.41	
7	1.0	4.00	-0.50	4.25	0.65	27.79	194.52	47.9
8	1.0	3.75	-0.25	3.88	0.52	28.95	231.61	
9	1.0	3.20	-0.50	3.45	0.41	29.03	261.27	
10	1.0	2.50	-0.70	2.85	0.30	27.08	270.75	
11	1.0	2.00	-0.50	2.25	0.21	24.11	265.18	
12	1.0	1.40	-0.60	1.70	0.12	24.08	289.00	
13	1.0	0.90	-0.50	1.15	0.09	14.69	191.03	
14	1.0	0.80	-0.10	0.85	0.06	12.04	168.58	
15	1.0	0.50	-0.30	0.65	0.04	9.43	141.38	

TABLE C-1-4: TRACER TEST DATA (TANK A (15 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$ / C	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	1.00	1.00	0.50	1.00	0.25	0.25	
2	1.0	1.8	0.8	1.4	0.67	2.93	5.85	
3	1.0	4.5	2.7	3.15	1.25	7.94	23.81	
4	1.0	6.5	2.00	5.5	1.57	19.27	77.07	
5	1.0	7.4	0.9	6.95	1.54	31.37	156.83	
6	1.0	8.0	0.6	7.7	1.40	42.35	254.1	
7	1.0	7.5	-0.5	7.75	1.19	50.47	353.31	
8	1.0	6.3	-1.2	6.9	0.92	51.75	414.0	
9	1.0	5.2	-1.1	5.75	0.68	48.62	437.59	99.7
10	1.0	4.4	-0.8	4.8	0.51	45.18	451.76	
12	2.0	3.0	-1.4	7.4	0.67	81.73	980.78	
14	2.0	2.1	-0.9	5.1	0.39	66.69	933.69	
16	2.0	1.7	-0.4	3.8	P.25	57.76	924.16	
18	2.0	1.1	-0.6	2.8	0.16	49.00	982.0	
20	2.0	0.8	-0.2	1.8	0.09	36.0	720.0	
22	2.0	0.5	-0.3	1.3	0.06	28.17	619.67	

TABLE C-1-5: TRACER TEST DATA (TANK A (20 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L) C	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\ t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	1.50	1.50	0.75	1.50	0.38	0.38	
2	1.0	4.00	2.50	2.75	1.83	4.13	8.26	
3	1.0	5.00	-1.00	5.50	2.20	13.75	41.25	
4	1.0	4.30	-0.70	4.65	1.33	16.26	65.04	25.03
6	1.0	3.00	-1.30	7.30	1.46	36.50	219.0	
8	1.0	1.00	-.200	4.00	0.57	28.07	224.56	
10	1.0	0.00	-1.00	1.00	0.11	9.09	90.91	

TABLE C-1-6: TRACER TEST DATA (TANK B (7 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L) C	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	12.00	12.00	6.00	12.00	3.00		
2	1.0	18.50	6.50	15.25	10.17	22.87		
3	1.0	23.50	5.00	21.00	8.40	52.50		
4	1.0	20.00	-3.50	21.75	6.21	76.18		23.10
5	1.0	17.00	-3.00	18.50	4.11	83.27		
6	1.0	12.50	-4.50	14.75	2.68	81.18		
7	1.0	8.50	-4.00	10.50	1.62	68.06		
8	1.0	5.50	-3.00	7.00	0.93	52.69		
9	1.0	3.00	-2.50	4.25	0.50	36.13		
10	1.0	0.00	-3.00	1.50	0.16	14.06		

TABLE C-1-7: TRACER TEST DATA (TANK B (11 DAYS)):

t (d)	Δt	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(C-\frac{1}{2}\Delta C)\,t$	Z Function $\frac{Y}{t-\Delta\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	2.00	2.00	1.00	2.00	0.50	0.50	
2	1.0	4.80	2.80	3.40	2.27	5.09	10.19	
3	1.0	6.00	1.20	5.40	2.16	13.50	40.50	
4	1.0	6.90	0.90	6.45	1.84	22.61	90.44	
5	1.0	8.00	1.10	7.45	1.66	33.44	167.18	
6	1.0	6.60	-1.40	7.30	1.33	40.07	240.41	
7	1.0	5.50	-1.10	6.05	0.93	39.36	275.50	
8	1.0	4.80	-0.70	5.15	0.69	38.44	307.51	69.00
9	1.0	4.20	-0.60	4.50	0.53	48.21	433.93	
10	1.0	3.80	-0.40	4.00	0.42	38.10	381.00	
11	1.0	3.20	-0.60	3.50	0.33	37.10	408.30	
12	1.0	2.70	-0.50	2.95	0.26	33.50	401.70	
13	1.0	2.50	-0.20	2.60	0.21	32.20	418.50	
14	1.0	2.10	-0.40	2.30	0.17	31.10	435.60	
15	1.0	1.70	-0.40	1.90	0.13	27.80	416.50	
16	1.0	1.00	-0.70	1.35	0.09	20.30	324.00	
17	1.0	0.50	-0.50	0.75	0.05	11.30	191.30	

TABLE C-1-8: TRACER TEST DATE (TANK B (15 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\Delta\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0							
2	1.0	1.50	1.50	0.75	0.50	1.13	2.25	
3	1.0	2.10	0.60	1.80	0.72	4.50	13.50	
4	1.0	3.10	1.00	2.60	0.74	9.14	36.54	
5	1.0	4.90	1.80	4.00	0.89	17.98	89.89	
6	1.0	6.30	1.40	5.60	1.02	30.75	184.47	
7	1.0	7.3	1.00	6.80	1.05	44.04	308.27	
8	1.0	8.0	0.70	7.65	1.02	57.38	459.0	
9	1.0	7.0	-1.00	7.50	0.88	63.92	575.28	
10	1.0	6.2	-0.8	6.60	0.69	63.13	631.30	
11	1.0	5.4	-0.8	5.8	0.55	61.16	672.8	
12	1.0	4.8	-0.6	51	0.44	59.11	709.36	124
14	2.0	3.5	-1.3	8.30	0.64	107.54	1506.97	
16	2.0	2.6	-0.9	6.10	0.41	90.76	1452.1	
18	2.0	1.6	-1.0	4.20	0.25	70.56	1270.08	
20	2.0	0.9	-0.7	2.50	0.13	48.08	961.54	
22	2.0	0.4	-0.5	1.30	0.06	28.17	619.67	

TABLE C-1-9: TRACER TEST DATA (TANK B (20 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L) C	ΔC (mg/L)	Y Function $(0-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.00							
1	1.0							
2	1.0	1.50	1.50	1.50	1.50	1.50	3.00	
4	1.0	4.00	2.50	5.50	1.83	16.53	66.12	
6	1.0	5.00	1.00	9.00	1.80	45.00	270.00	
8	1.0	3.00	-2.00	9.00	1.14	56.14	449.12	
10	1.0	2.50	-0.50	6.00	0.67	53.73	537.31	
12	1.0	1.50	-1.00	4.00	0.36	44.44	533.33	
14	1.0	0.50	-1.00	2.00	0.15	26.67	373.33	
16	1.0	0.25	-0.25	0.75	0.05	11.26	180.00	
18	1.0	0.15	-0.10	0.40	0.02	8.00	144.00	
20	1.0	0.00	-0.15	0.15	0.01	2.25	45.00	

TABLE C-1-10: TRACER TEST DATA (TANK C (7 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(o-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	7.50	7.50	3.75	7.50	1.88	1.88	
2	1.0	12.50	5.00	10.00	6.67	14.99	29.98	
3	1.0	18.00	5.50	15.25	6.10	38.13	114.39	
4	1.0	24.00	6.00	21.00	6.00	73.50	294.00	
5	1.0	24.00	0.00	24.00	5.33	108.07	540.35	
6	1.0	20.00	-4.00	22.00	4.00	121.00	726.00	
7	1.0	18.00	-2.00	19.00	2.92	123.63	865.41	46.40
8	1.0	16.00	-2.00	17.00	2.27	127.31	1018.48	
9	1.0	12.50	-3.50	14.25	1.68	120.87	1087.83	
10	1.0	10.00	-2.50	11.25	1.18	107.26	1072.60	
11	1.0	7.00	-3.00	8.50	0.81	89.20	981.2	
12	1.0	4.50	-2.50	5.75	0.50	66.13	793.56	
13	1.0	1.50	-3.00	3.00	0.24	37.50	487.5	
14	1.0	0.00	-1.50	0.07	0.06	9.38	131.32	

TABLE C-1-11: TRACER TEST DATA (TANK C (11 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	1.20	1.20	0.60	1.20	0.30	0.30	
2	1.0	2.60	1.40	1.90	1.27	2.84	5.69	
3	1.0	4.50	1.90	3.55	1.42	8.88	26.63	
4	1.0	5.50	1.00	5.00	1.43	17.48	69.93	
5	1.0	6.80	1.30	6.15	1.37	27.61	138.04	
6	1.0	8.00	1.20	7.40	1.35	40.56	243.38	
7	1.0	6.70	-1.30	7.35	1.13	47.81	334.65	
8	1.0	5.70	-1.00	6.20	0.83	46.31	370.51	
9	1.0	4.60	-1.10	5.15	0.61	43.48	391.32	89.90
10.	1.0	3.80	-0.80	4.20	0.44	40.09	400.91	
11	1.0	3.50	-0.30	3.65	0.35	38.06	418.71	
12	1.0	3.10	-0.40	3.30	0.29	37.55	450.62	
13	1.0	2.70	-0.40	2.90	0.23	36.57	475.35	
14	1.0	2.30	-0.40	2.50	0.19	32.89	460.53	
15	1.0	2.00	-0.30	2.15	0.15	30.82	462.25	
16	1.0	1.70	-0.30	1.85	0.12	28.52	456.33	
17	1.0	1.50	-0.20	1.60	0.10	25.60	435.20	
18	1.0	1.40	-0.10	1.45	0.08	26.28	473.06	

TABLE C-1-12: TRACER TEST DATA (TANK C (15 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0							
2	1.0	0.5	0.5	0.5	0.5	0.5	1.00	
4	1.0	2.0	1.5	2.5	0.83	7.53	30.12	
6	1.0	3.5	1.5	5.5	1.10	27.5	165.00	
8	1.0	6.0	2.5	9.5	1.36	66.36	530.88	
10	1.0	8.0	2.0	14.00	1.56	125.64	1256.4	
12	1.0	6.5	-1.5	14.50	1.32	159.28	1911.36	156
14	1.0	5.5	-1.0	12.00	0.92	156.52	2191.28	
16	1.0	3.8	-1.7	9.30	0.62	139.50	2232.0	
18	1.0	2.3	-1.5	6.10	0.36	103.36	1860.48	
20	1.0	1.2	-1.1	3.50	0.18	68.06	1361.2	
22	1.0	0.5	-0.7	1.7	0.08	36.13	794.86	

TABLE C-1-13: TRACER TEST DATA (TANK C (20 DAYS))

t	Δt	Tracer Conc. (mg/L) C	ΔC (mg/L)	Y Function $(C-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{Wt}{\Sigma Y}$ β
0	0.0							
1	1.0							
2	1.0	0.50	0.50	0.50	0.50	0.50	1.00	
4	1.0	3.00	2.50	3.50	1.17	10.47	41.88	
6	1.0	4.00	1.00	7.00	1.40	35.00	210.00	
8	1.0	5.50	1.50	9.50	1.36	66.36	530.88	
10	1.0	2.50	-0.30	8.00	0.89	71.91	719.10	
12	1.0	1.50	-1.00	4.00	0.36	44.44	533.33	91.3
14	1.0	1.00	-0.50	2.50	0.19	32.89	460.53	
16	1.0	0.50	-0.50	1.50	0.10	22.50	360.00	
18	1.0	0.25	-0.25	0.75	0.04	14.06	253.13	
20	1.0	0.15	-0.10	0.40	0.02	8.60	160.00	
22	1.0	0.10	-0.05	0.25	0.01	6.25	137.50	
24	1.0	0.00	-0.10	0.10	0.004	2.50	60.00	

TABLE C-1-14: TRACER TEST DATA (TANK D (7 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L) C	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	6.00	6.00	3.00	6.00	1.50	1.50	
2	1.0	11.00	5.00	8.50	5.67	12.74	25.48	
3	1.0	14.50	3.50	12.75	5.10	31.88	95.64	
4	1.0	17.00	2.50	15.75	4.50	55.13	220.52	
5	1.0	20.00	3.00	18.50	4.11	83.27	416.35	
6	1.0	22.00	2.00	21.00	3.82	115.45	692.70	
7	1.0	24.00	2.00	23.00	3.54	149.44	1046.08	
8	1.0	22.50	-1.50	23.25	3.10	174.38	1395.04	63.30
9	1.0	18.00	-4.50	20.25	2.38	172.30	1550.70	
10	1.0	15.00	-3.00	16.50	1.74	156.47	1564.70	
11	1.0	11.50	-3.50	13.25	1.26	139.34	1532.74	
12	1.0	8.00	-3.50	9.75	0.85	111.84	1342.08	
13	1.0	6.00	-2.00	7.00	0.56	87.50	1137.50	
14	1.0	3.50	-2.50	4.75	0.35	64.46	902.44	
15	1.0	1.50	-2.00	2.50	0.17	36.76	551.40	
16	1.0	0.00	-1.50	0.75	0.05	11.25	180.00	

TABLE C-1-15: TRACER TEST DATA (TANK D (11 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\ C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	0.70	0.70	0.35	0.70	0.18	0.18	
2	1.0	1.50	0.80	1.10	0.73	1.66	3.32	
3	1.0	2.40	0.90	1.95	0.78	4.88	14.63	
4	1.0	3.50	1.10	2.95	0.84	10.36	41.44	
5	1.0	4.40	0.90	3.95	0.88	17.73	88.65	
6	1.0	6.00	1.60	5.20	0.95	28.46	199.24	
7	1.0	7.00	1.00	6.50	1.00	42.25	295.75	
8	1.0	8.00	1.00	7.50	1.00	56.25	450.00	
9	1.0	6.50	-1.50	7.25	0.85	61.84	556.54	
10	1.0	5.20	-1.30	5.85	0.62	55.20	551.98	119.00
11	1.0	4.20	-1.00	4.70	0.45	49.09	539.98	
12	1.0	3.50	-0.70	3.85	0.33	44.92	539.00	
13	1.0	3.10	-0.40	3.30	0.26	41.88	544.50	
14	1.0	2.70	-0.40	2.90	0.21	40.05	560.67	
15	1.0	2.50	-0.20	2.60	0.18	37.56	563.33	
16	1.0	2.30	-0.20	2.40	0.15	38.40	614.40	
17	1.0	2.00	-0.30	2.15	0.13	35.56	604.48	
18	1.0	1.80	-0.20	1.90	0.11	32.82	590.73	
19	1.0	1.50	0.30	1.65	0.09	30.25	574.75	
20	1.0	1.30	-0.20	1.40	0.07	28.00	560.00	
21	1.0	1.20	-0.10	1.25	0.06	26.00	546.87	

TABLE C-1-16: TRACER TEST DATA (TANK D (15 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0							
2	1.0	0.3	0.3	0.3	0.3	0.3	0.6	
4	1.0	1.0	0.7	1.30	0.43	3.93	15.72	
6	1.0	2.0	1.0	3.00	0.60	15.00	90.00	
8	1.0	3.3	1.3	5.30	0.76	36.96	295.68	
10	1.0	5.0	1.7	8.30	0.92	74.88	748.80	
12	1.0	6.6	1.6	11.60	1.05	128.15	1537.80	
14	1.0	8.0	1.4	14.60	1.12	190.32	2664.48	
16	1.0	7.1	-0.9	13.30	0.89	198.75	3180.04	290.00
18	1.0	6.8	-0.3	13.90	0.82	236.30	4253.40	
20	1.0	5.4	-1.4	12.20	0.64	232.56	4651.20	
22	1.0	3.8	-1.6	9.20	0.44	192.36	4231.92	
24	1.0	2.9	-0.9	6.70	0.29	154.79	3714.96	
26	1.0	2.0	-0.9	4.90	0.20	120.05	3121.30	
28	1.0	1.3	-0.7	3.30	0.12	90.75	2541.00	
20	1.0	0.7	-0.6	2.00	0.07	57.14	1714.20	
32	1.0	0.1	-0.6	1.60	0.05	51.20	1638.40	

TABLE C-1-17: TRACER TEST DATA (TANK D (20 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0							
2	1.00							
4	1.0	1.50	1.50	1.50	0.50	4.50	18.00	
6	1.0	3.00	1.50	4.50	0.90	22.50	135.00	
8	1.0	4.50	1.50	7.50	1.07	52.57	420.56	
10	1.0	6.00	1.50	10.50	1.17	94.23	942.31	
12	1.0	3.00	-3.00	9.00	0.82	78.78	1185.37	
14	1.0	2.50	-0.50	5.50	0.42	72.02	1008.3	
16	1.0	1.50	-1.00	4.00	0.27	59.26	948.15	
18	1.0	1.00	-0.50	2.50	0.15	41.67	750.00	139
20	1.0	0.50	-0.50	1.50	0.08	28.13	562.50	
22	1.0	0.30	-0.20	0.80	0.04	16.00	352.00	
24	1.0	0.20	-0.10	0.50	0.02	12.50	300.00	

TABLE C-1-18: TRACER TEST DATA (TANK E (7 DAYS))

M t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	4.00	4.00	2.00	4.00	1.00	1.00	
2	1.0	7.00	3.00	5.50	3.67	8.24	16.48	
3	1.0	10.00	3.00	8.50	3.40	21.25	63.75	
4	1.0	14.50	4.50	12.25	3.50	42.88	171.52	
5	1.0	18.00	3.50	16.25	3.61	73.15	365.75	
6	1.0	20.50	2.50	19.25	3.50	105.88	635.28	
7	1.0	22.00	2.50	20.75	3.19	134.97	944.79	
8	1.0	24.00	2.00	23.00	3.07	172.31	1378.48	
9	1.0	23.00	-1.00	23.50	2.76	200.09	1800.81	97.90
10	1.0	20.00	-3.00	21.50	2.26	204.54	2045.40	
11.	1.0	18.00	-2.00	19.00	1.81	199.45	2193.95	
12	1.0	15.50	-2.50	16.75	1.46	192.17	2306.04	
13	1.0	14.00	-1.50	14.75	1.18	184.38	2396.94	
14	1.0	11.00	-3.00	12.50	0.93	168.01	2352.14	
15	1.0	9.50	-1.50	10.25	0.71	147.98	2219.70	
16	1.0	7.00	-2.50	8.25	0.53	128.42	2054.72	
17	1.0	4.00	-3.00	5.50	0.33	91.62	1558.39	
18	1.0	2.00	-2.00	3.00	0.17	52.94	952.92	
19	1.0	0.00	-2.00	1.00	0.05	20.00	380.00	

TABLE C-1-19: TRACER TEST DATA (TANK E (11 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0	0.30	0.30	0.15	0.30	0.08	0.08	
2	1.0	0.70	0.40	0.50	0.33	0.76	1.52	
3	1.0	1.20	0.50	0.90	0.38	2.38	7.13	
4	1.0	2.00	0.80	1.60	0.46	5.57	22.28	
5	1.0	2.90	0.90	2.45	0.54	11.12	55.58	
6	1.0	4.00	1.10	3.45	0.63	18.89	113.36	
7	1.0	4.90	0.90	4.45	0.68	29.12	203.86	
8	1.0	6.30	1.40	5.60	0.75	41.81	334.51	
9	1.0	8.00	1.70	7.15	0.84	60.86	547.74	
10	1.0	6.70	-1.30	7.35	0.77	70.16	701.59	
11	1.0	5.80	-0.90	6.25	0.60	65.10	716.15	148.00
12	1.0	5.00	-0.80	5.40	0.47	62.04	744.51	
13	1.0	4.50	-0.50	4.75	0.38	59.38	771.88	
14.	1.0	3.80	-0.70	4.15	0.31	55.56	777.79	
15	1.0	3.30	-0.50	3.55	0.24	52.51	787.66	
16	1.0	2.70	-0.60	3.00	0.19	47.37	757.89	
17	1.0.	2.30	-0.40	2.50	0.15	41.67	708.38	
18	1.0	2.50	-0.30	2.15	0.12	38.52	693.57	
19	1.0	1.70	-0.30	1.85	0.10	34.23	650.28	
20	1.0	1.40	-0.30	1.55	0.08	30.03	600.63	
21	1.0	1.30	-0.10	1.35	0.07	26.04	546.75	
22	1.0	1.10	-0.20	1.20	0.06	24.00	528.00	

TABLE C-1-20: TRACER TEST DATA (TANK E (15 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0	0.3						
1	1.0	0.0						
2	1.0	0.0						
4	1.0	0.2	0.2	0.20	0.07	0.57	2.28	
6	2.0	0.5	0.3	0.70	0.14	3.50	21.00	
8	2.0	1.2	0.7	1.70	0.24	12.04	96.32	
10	2.0	3.0	1.8	4.20	0.47	37.53	375.32	
12	2.0	4.8	1.8	7.80	0.71	85.69	1028.28	
14	2.0	6.0	1.2	10.80	0.83	140.53	1967.42	
16	2.0	8.0	2.0	14.00	0.93	210.75	3372.04	416.00
18	2.0	7.1	-0.90	15.10	0.89	256.19	4611.44	
20	2.0	6.3	-0.80	13.40	0.71	252.90	5058.33	
22	2.0	5.00	-1.30	11.30	0.54	236.46	5202.12	
24	2.0	4.10	-0.90	9.10	0.40	207.03	4968.60	
26	2.0	2.80	-1.30	6.90	0.28	170.04	4420.93	
28	2.0	2.20	-0.60	5.00	0.19	131.58	3684.21	
29	2.0	2.00	-0.20	4.20	0.14	126.00	3780.00	
30	2.0	1.90	-0.10	3.90	0.13	117.00	3744.00	
31	2.0	1.80	-0.10	3.70	0.11	124.45	4231.45	

TABLE C-1-21: TRACER TEST DATA (TANK E (20 DAYS))

t (d)	Δt (d)	Tracer Conc. (mg/L)	ΔC (mg/L)	Y Function $(O-\frac{1}{2}\Delta C)\Delta t$	Z Function $\frac{Y}{t-\frac{1}{2}\Delta t}$	W Function $\frac{Y^2}{Z}$	Wt λ	$\frac{\Sigma Wt}{\Sigma Y}$ β
0	0.0							
1	1.0							
2	1.0							
3	1.0							
4	1.0	1.50	1.50	1.50	0.50	4.50	18.00	
6	1.0	2.50	1.00	4.00	0.80	20.00	120.00	
8	1.0	3.50	1.00	6.00	0.86	41.86	334.88	
10	1.0	6.00	2.50	9.50	1.06	85.14	851.42	
12	1.0	7.50	1.50	13.50	1.23	148.17	1778.05	
14	1.0	4.00	-3.50	11.50	0.88	150.28	2103.98	169
16	1.0	2.40	-1.6	6.40	0.43	95.26	1524.09	
18	1.0	1.50	-0.90	3.90	0.23	66.13	1190.36	
20	1.0	1.00	-0.50	2.50	0.13	48.08	961.54	
22	1.0	0.80	-0.20	1.80	0.09	36.00	792.00	
24	1.0	0.75	-0.05	1.55	0.07	34.32	823.71	

TABLE C-2-1: SYNTHETIC MILK WASTE EQUIVALENT BOD-COD CONCENTRATION

17/8/84 - 22/8/84			20/8/84 - 25/8/84			23/8/84 - 28/8/84			MEAN		
CONC./50L	COD*	BOD*	CONC./50L	COD*	BOD*	CONC./50L	COD*	BOD*	CONC./50L	COD*	BOD*
11.90	137	88	12.2	141	98	12.00	142	96	12.00	140	94.00
21.40	321	212	22	320	217	22.0	319	213	22.00	320	214
34.00	447	298	35	450	305	33.00	447	297	34.00	448	300
43.80	650	450	44	649	438	43.60	651	432	44.00	650	440
54.00	884	607	57	896	603	54.00	890	590	55.00	890	600
66.00	1182	829	67.00	1180	791	65.00	1178	780	66.00	1180	800
77.00	1354	901	76.60	1348	907	77.30	1348	892	77.00	1350	900
87,00	1519	975	89.00	1521	1020	88.00	1520	1005	88.00	1520	1000
108.00	1789	1192	109.00	1804	1210	110.00	1807	1198	109.	1800	1200

* mg/L

TABLE C-2-2: TEST NO. 1 (300 mg/L BOD_5) - 6/10/84 to 27/10/84

	I N F L U E N T								E F F L U E N T S							
									T A N K A							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY
1	310	306	463	459	1.4	7.2	125	150	34	25	100	80	2.3	7.6	21	140
3	300	297	450	447	7.6	8.1	135	140	39	29	110	80	3.6	7.9	23	140
5	320	315	480	474	2.0	7.1	125	160	37	26	99	79	3.9	8.0	21	140
7	300	295	450	445	1.2	6.9	100	130	39	28	108	82	4.0	8.0	24	130
9	300	297	450	448	7.8	8.0	95	94	34	25	98	78	4.3	7.9	21	120
11	298	296	445	440	1.4	7.8	95	100	43	33	123	85	2.2	7.8	25	130
13	290	287	433	424	0.7	7.0	85	90	52	38	141	94	2.0	7.6	28	140
15	310	305	460	451	7.8	8.0	120	95	40	29	120	80	2.1	7.7	25	150
17	300	294	450	446	2.0	7.4	100	100	42	31	126	86	2.2	7.7	26	140
19	310	307	460	449	1.4	6.7	105	110	47	35	138	94	2.4	7.8	27	136
21	300	296	450	441	0.7	6.1	98	105	43	32	139	92	2.5	8.0	27	138
$\bar{x}$	300	299.55	453.73	447.64	3.09	6.65	107.55	115.82	41	30	119	85	2.86	7.82	24.36	136.73

../Cont.

TABLE C-2-2 (CONTINUED)

	I N F L U E N T								E F F L U E N T S — T A N K E							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY
1	310	306	463	459	1.4	7.2	125	150	25	20	105	47	2.5	7.8	12	190
3	300	297	450	447	7.6	8.1	135	140	22	18	90	40	3.9	8.1	9	180
5	320	315	480	474	2.0	7.1	125	160	21	18	81	38	4.2	8.2	8	180
7	300	295	450	445	1.2	6.9	100	130	21	17	80	36	4.0	8.1	9	180
9	300	297	450	448	7.8	8.0	95	94	19	16	80	39	4.8	8.3	8	180
11	298	296	445	440	1.4	7.8	95	100	22	17	95	40	3.7	8.0	9	180
13	290	287	433	424	0.7	7.0	85	90	23	18	100	44	2.2	7.7	11	200
15	310	305	460	451	7.8	8.0	120	95	25	20	115	52	2.6	7.9	12	180
17	300	294	450	446	2.0	7.4	100	100	22	17	100	41	2.5	8.0	10	180
19	310	307	460	449	1.4	6.7	105	110	26	21	110	47	3.1	7.9	11	185
21	300	296	450	441	0.7	6.1	98	105	24	19	100	49	3.0	7.8	11	190
$\bar{x}$	300	299.55	453.73	447.64	3.09	6.65	107.55	115.82	22.73	18	96	43	3.32	7.98	10	184.0

TABLE C-2-3: TEST NO. 2 (600 mg/L BOD_5) - 24/10/84 to 14/11/84

	I N F L U E N T								E F F L U E N T S — T A N K A							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA LINI
1	608	600	907	900	4.8	7.8	195	138	158	102	262	199	1.5	6.7	42	300
3	600	590	896	874	8.2	8.5	200	120	145	94	241	184	1.9	6.6	39	280
5	610	604	910	896	4.0	7.4	210	118	169	110	278	214	2.2	7.2	45	260
7	600	592	896	888	1.4	6.7	215	140	160	104	266	203	2.0	7.1	43	280
9	594	586	887	882	8.1	8.8	160	130	157	102	261	198	1.9	7.0	42	284
11	615	600	918	910	7.8	8.0	205	122	149	97	247	189	1.6	6.8	40	270
13	610	604	910	892	8.1	8.0	195	110	152	99	253	194	2.1	7.0	41	250
15	600	584	896	880	6.0	7.4	210	108	131	85	216	166	2.0	7.1	35	244
17	610	605	910	894	7.9	8.6	222	110	119	77	198	151	2.0	7.2	32	200
19	600	592	896	872	6.0	7.1	175	128	168	109	276	212	2.1	7.2	45	210
21	590	581	880	870	7.6	8.0	195	140	155	100	259	198	2.1	7.1	42	190
$\bar{x}$	600	594	900.55	887	6.35	7.85	198.36	124	151.18	98.09	25.64	191.64	1.95	7.0	40.55	251.64

TABLE C-2-3 CONTINUED

	I N F L U E N T								E F F L U E N T S							
									T A N K E							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY
1	608	600	907	900	4.8	7.8	195	138	136	58	254	194	2.0	7.1	56	320
3	600	590	896	874	8.2	8.5	200	120	131	55	246	191	1.8	7.0	53	296
5	610	604	910	896	4.0	7.4	210	118	111	44	216	164	2.2	7.2	40	298
7	600	592	896	888	1.4	6.7	215	140	135	57	249	191	2.6	7.5	53	300
9	594	586	887	882	8.1	8.8	160	130	128	51	237	180	2.1	7.0	50	310
11	615	600	918	910	7.8	8.0	205	122	135	54	245	188	2.0	7.2	53	310
13	610	604	910	892	8.1	8.0	195	110	118	46	225	167	1.8	6.9	43	292
15	600	584	896	880	6.0	7.4	210	108	126	47	233	176	1.9	6.9	48	290
17	610	605	910	894	7.9	8.6	222	110	121	46	228	173	2.0	7.0	48	295
19	600	592	896	872	6.0	7.1	175	128	131	53	242	185	2.4	7.4	52	300
21	590	581	880	870	7.6	8.0	195	140	127	50	234	176	2.1	7.0	48	290
$\bar{x}$	600	594	900.55	887	6.35	7.85	198.36	124	127.18	51.00	237.18	181.00	2.08	7.11	49.45	300.00

TABLE C-2-4: TEST NO. 3 (900mg/L BOD_5) - 12/11/84 to 4/12/84

	I N F L U E N T								E F F L U E N T S							
									T A N K A							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINIT
1	900	892	1343	1317	7.1	8.0	360	200	276	196	606	378	1.8	7.5	100	300
2	896	889	1330	1304	3.0	7.4	360	170	275	196	605	378	2.5	7.2	90	260
3	910	901	1358	1331	8.4	8.5	435	180	240	180	557	348	2.3	7.2	75	280
4	895	884	1336	1309	6.8	8.0	387	154	275	189	600	375	2.0	7.3	87	360
5	910	900	1358	1333	8.2	8.5	440	210	291	199	628	393	3.3	7.9	120	280
6	900	892	1343	1320	3.8	7.8	410	198	288	197	622	389	3.0	7.4	110	310
7	910	903	1358	1327	1.5	7.0	396	174	273	188	596	372	2.4	7.1	80	300
8	904	896	1349	1332	0.9	5.1	377	160	260	186	582	364	2.5	7.2	78	260
9	900	894	1343	1317	8.1	8.6	410	210	261	186	579	363	2.3	7.2	77	270
10	896	885	1337	1311	3.2	8.0	390	180	277	189	603	377	2.5	7.4	88	264
11	893	880	1333	1307	1.1	5.7	380	145	290	197	621	388	2.4	7.4	107	284
12	900	892.36	1344.36	1318	4.74	7.51	395	180	273.27	191	600	375	2.45	7.35	92	288

../Cont.

TABLE C-2-4 CONTINUED

	I N F L U E N T								E F F L U E N T S							
									T A N K E							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINTTY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINTY
1	900	892	1343	1317	7.1	8.0	360	200	280	122	541	361	2.9	7.3	110	350
2	896	889	1330	1304	3.0	7.4	360	170	270	114	526	350	3.5	7.6	88	400
3	910	901	1358	1331	8.4	8.5	435	180	275	113	526	346	3.1	7.4	88	350
4	895	884	1336	1309	6.8	8.0	387	154	273	118	532	350	2.5	7.5	97	460
5	910	900	1358	1333	8.2	8.5	440	210	311	134	609	406	4.6	7.8	126	330
6	900	892	1343	1320	3.8	7.8	410	198	310	134	603	402	4.1	7.6	126	390
7	910	903	1358	1327	1.5	7.0	396	174	283	127	548	363	3.7	7.7	118	360
8	904	896	1349	1322	0.9	5.1	377	160	298	129	564	376	4.3	7.8	120	380
9	900	894	1343	1317	8.1	8.6	410	210	331	142	680	421	4.1	8.0	131	382
10	896	885	1337	1311	3.2	8.0	390	180	295	127	561	374	3.7	7.8	120	368
11	893	880	1333	1307	1.1	5.7	380	145	308	130	600	390	3.9	7.6	123	340
12	900	892.36	1344.36	1318	4.74	7.51	395	180	294	127	571.82	376.27	3.69	7.65	113.36	373.64

TABLE C-2-5: TEST NO. 4 (1200 mg/L BOD_5) - 17/12/84 to 7/1/85

	I N F L U E N T								E F F L U E N T S — T A N K A							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY
1	1203	1196	1796	1785	9.2	8.5	500	280	579	455	1304	717	2.6	7.8	325	370
3	1196	1189	1785	1774	4.0	8.3	474	248	572	450	1288	708	3.0	8.0	298	376
5	1205	1198	1799	1788	9.0	8.8	515	259	578	455	1302	716	3.0	8.0	310	380
7	1196	1190	1785	1776	3.4	8.6	471	250	570	448	1284	704	2.8	7.9	240	370
9	1200	1193	1791	1780	8.9	8.1	492	220	573	450	1290	710	2.9	7.8	300	364
11	1195	1188	1784	1774	3.5	7.7	460	214	565	445	1273	698	2.6	7.6	232	370
13	1208	1201	1803	1792	8.8	8.0	515	220	558	440	1256	690	2.8	7.7	220	368
15	1200	1195	1791	1778	3.0	7.8	484	210	571	450	1286	707	2.8	7.8	294	372
17	1205	1198	1799	1790	8.8	8.2	510	220	599	455	1350	743	3.0	8.0	340	370
19	1201	1194	1793	1782	3.8	8.1	494	214	578	455	1301	715	3.1	8.0	300	370
21	1198	1193	1788	1777	0.3	7.7	470	219	571	450	1284	706	3.0	7.9	290	370
MEAN	1200.64	1194.09	1792	1781	5.70	8.16	489.64	232.18	574	450.27	1292.55	710.36	2.87	7.86	286.27	370.91

../Cont.

TABLE C-2-5 CONTINUED

	I N F L U E N T								E F F L U E N T S T A N K E							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINIT
1	1203	1196	1796	1785	9.2	8.5	500	280	399	293	1277	636	3.9	7.8	280	476
3	1196	1189	1785	1774	4.0	8.3	474	248	414	294	1315	638	4.0	8.2	310	489
5	1205	1198	1799	1788	9.0	8.8	515	259	386	289	1257	629	4.0	8.1	275	482
7	1196	1190	1785	1776	3.4	8.6	471	250	370	285	1194	620	3.8	7.9	245	469
9	1200	1193	1791	1780	8.9	8.1	492	220	372	287	1217	624	3.8	8.0	246	482
11	1195	1188	1784	1774	3.5	7.7	460	214	433	299	1374	650	3.9	8.1	310	482
13	1208	1201	1803	1792	8.8	8.0	515	220	363	283	1153	615	3.7	7.7	240	470
15	1200	1195	1791	1778	3.0	7.8	484	210	349	279	1089	606	3.4	7.6	210	465
17	1205	1198	1799	1790	8.8	8.2	510	220	376	288	1241	626	3.6	7.7	266	476
19	1201	1194	1793	1782	3.8	8.1	495	214	356	281	1119	610	3.7	7.8	210	477
21	1198	1193	1788	1777	0.3	7.7	470	219	420	297	1343	646	3.8	8.0	310	482
MEAN	1200.64	1194.09	1792	1781	5.70	8.16	489.64	232.18	385.27	288.64	1234.45	627.27	3.78	7.90	263.82	477.27

TABLE C-2-6: TEST NO. 5 (200 mg/L BOD_5) - 5/1/85 to 27/1/85

	I N F L U E N T								E F F L U E N T S							
									T A N K A							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY
1	200	197	299	290	8.4	8.4	86	105	15	12	22	16	2.6	7.9	22	117
3	194	188	290	281	8.0	8.0	84	100	18	14	23	17	2.8	7.9	26	119
5	210	206	313	303	8.6	8.4	89	106	13	11	18	13	2.8	8.0	20	120
7	202	197	301	292	6.0	7.9	82	98	15	12	23	16	2.8	7.8	23	94
9	210	205	313	301	8.1	8.6	86	107	14	11	19	14	2.8	7.9	21	110
11	204	199	305	295	4.7	8.0	88	102	15	12	21	15	2.9	8.0	22	124
13	205	197	306	297	8.8	8.5	80	106	17	13	23	16	3.0	8.1	25	142
15	198	189	296	287	7.2	7.9	81	97	12	11	17	12	3.1	8.1	20	140
17	208	203	310	300	7.9	8.3	80	103	18	14	25	18	2.8	8.0	28	128
19	196	188	293	284	6.1	7.9	79	96	11	10	16	12	3.1	8.2	19	142
21	184	172	275	266	1.8	7.3	80	84	17	14	22	16	3.2	8.1	25	140
$\bar{x}$	200	194.64	300	291	6.68	8.11	83.18	100.36	15.09	12.18	20.82	15.00	2.9	8.0	22.82	125.09

...(Cont.)

TABLE C-2-6 CONTINUED

	INFLUENT								EFFLUENTS TANK E							
	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINITY	BOD_u (mg/L)	BOD_f (mg/L)	COD_u (mg/L)	COD_f (mg/L)	DO (mg/L)	pH	SS (mg/L)	ALKA-LINT
1	200	197	299	290	8.4	8.4	86	105	9	5	19	12	3.0	8.1	8	160
3	194	188	290	281	5.9	8.0	84	100	12	7	23	14	2.7	7.8	10	162
5	210	206	313	303	8.6	8.4	89	106	9	5	18	11	2.9	7.9	7	154
7	202	197	301	292	6.0	7.9	82	98	11	6	20	12	3.1	8.0	9	156
9	210	205	313	301	8.1	8.6	86	107	11	7	20	12	3.0	8.1	9	160
11	204	199	305	295	4.7	8.0	88	102	8	5	18	11	3.1	8.2	5	164
13	205	197	306	297	8.8	8.5	80	106	14	8	25	16	3.0	8.1	13	162
15	198	189	296	287	7.2	7.9	81	97	8	5	17	11	3.3	8.4	5	168
17	208	203	310	300	7.9	8.3	80	103	10	6	19	11	3.2	8.4	9	170
19	196	188	293	284	6.1	7.9	79	96	9	5	18	12	3.0	8.3	6	166
21	184	172	275	266	1.8	7.3	80	84	13	7	24	15	3.1	8.3	12	168
$\bar{x}$	200	194.64	300	291	6.68	8.11	83.18	100.36	10.36	6.0	20.18	12.55	3.04	8.15	8.45	161.8

TABLE C-2-7: PONDS' MICROBIOLOGICAL FEATURES

ORGANIC LOADING (kg/ha/d) BOD_5	P O N D A									
	TOTAL COLIFORM ($\frac{10^6}{100\ mL}$)			FAECAL COLIFORM ($\frac{10^6}{100\ mL}$)			FAECAL STREPTOCOCCT ($\frac{10^6}{100\ mL}$)			ALGAE (10^6/100mL)*
	POND	EFFLUENT	% REMOVAL	POND	EFFLUENT	% REMOVAL	POND	EFFLUENT	% REMOVAL	POND
33.30	2.89	0.285	90.14	0.439	0.004	99.10	0.0098	0.000092	99.06	19.50
50.10	25.79	2.62	89.84	27.70	0.36	98.70	3.10	0.05	98.83	21.0
100.00	29.50	2.99	89.86	31.20	0.44	98.59	4.10	0.06	98.54	23.00
150.00	2.60	0.28	89.23	0.36	0.006	98.33	0.047	0.0008	98.30	26.00
200.00	0.275	0.0437	84.11	0.00594	0.00128	78.40	0.00079	0.000172	78.23	34.00

*Cells

TABLE C-2-7 (CONTINUED)

ORGANIC LOADING (kg/ha/d) BOD_5	P O N D E									
	TOTAL COLIFORM ($\frac{10^6}{100mL}$)			FAECAL COLIFORM ($\frac{10^6}{100mL}$)			FAECAL STREPTOCOCCI ($\frac{10^6}{100mL}$)			ALGAE (10^6/100mL)*
	POND	EFFLUENT	% REMOVAL	POND	EFFLUENT	% REMOVAL	POND	EFFLUENT	% REMOVAL	POND
33.30	2.363	0.1276	94.60	0.0992	0.00017	99.83	0.0059	0.000004	99.23	23
50.10	27.70	2.29	91.73	28.40	0.050	99.82	3.90	0.004	99.90	25.50
100.00	30.70	2.43	92.08	31.25	0.10	99.68	4.488	0.006	99.87	32.00
150.00	2.10	0.223	89.38	0.047	0.0004	99.15	0.0038	0.00002	99.47	41.50
200.00	0.2044	0.0281	86.25	0.00039	0.000075	80.77	0.00002	0.0000038	81.00	54.50

* Cells

APPENDIX D

STABILIZATION POND UNIFIED DESIGN PROCESS

From the pond test it was found that tank A without baffle having dispersion number, d of 0.151 has to be 2.3 operating in series to have the same efficiency as that of tank E with 15 baffles and dispersion number, d of 0.030.

Suppose t_1, t_2 .... t_n refer to the inflow detention times in a series of n equal ponds, then

$$C_1 = \frac{c_1}{(Kt_1 + 1)} \qquad \text{..... D-1}$$

$$C_2 = \frac{C_o}{(Kt_1 + 1)(Kt_2 + 1)}, \text{ and} \qquad \text{..... D-2}$$

$$C_n = \frac{C_o}{(Kt_1+1)(Kt_2+1)\ldots(Kt_n+1)} \qquad \text{..... D-3}$$

$$= \frac{C_o}{\prod_{i=1}^{n} (Kt_n+1)} \qquad \text{..... D-4}$$

$$C_n = \frac{C_o}{(Kt + 1)^n} \qquad \text{..... D-5}$$

D-1 reduces to D-2 when $t_1 = t_2 = t_n$.

Due to the nature of the dispersion number, d as it approaches zero, the system tends towards plug flow. This happens when L the length of travel in (D-13) becomes very large. Then the system is analogous to a river with plug or piston flow. Under this condition the concentration gradient C^1 in the river is just due to reaction rate K, hence

$$dc/dt = - KC^1 \qquad \text{..... D-6}$$

where:

$$C' = C_o \frac{Q_p}{Q_r + Q_p}$$

Q_p = Pollution flow rate

and Q_r = River flow rate

$$C' = C_o \frac{Q_p}{Q_r + Q_p} e^{-kt} \quad \text{.... D-7}$$

If the flow is mainly that of pollution, Q_p

then

$$C' = C_o e^{-kt} \quad \text{..... D-8}$$

If the concentration in ponds in series is analogous to a single but baffled pond, then a common expression as observed in the test can be used to describe either of them. This expression can then be related to that for a river since as d tends to zero the reactor behaves like a river.

As stated in D-5,

$$C_n = \frac{C_o}{(Kt + 1)^n} \quad \text{..... D-5}$$

At any specific time, L_b/u (t_g = Actual detention time of baffled pond having L_b as the length of travel)

$$\therefore \quad L_b/u / L_p/u = n \quad \text{.....D-9}$$

i.e

$$L_b/L_p = n$$

where:

$L_p/u = t$, Actual detention time in plain pond having L_p as the length of travel.

Substituting in D-5

$$C_n = \frac{C_o}{(1+kL_p/u)L_b/L_p} \qquad \text{..... D-10}$$

As n increases for any specific length of travel in a baffled pond, L in the plain pond decreases or conversely in actual sense L in the baffled pond increases. Hence

$$\lim_{L_p \to 0} C_n = \lim_{L_p \to 0} C_o(1 + K^{L_P}/u)^{-L_b/L_p} \qquad \text{..... D-11}$$

$$C_n = C_o \; e^{-kt} \qquad \text{.....D-12}$$

As n becomes very large and L_p tends to zero or L_b tends to infinity the same condition of plug or piston flow of a river occurs. This is all the advantage one is attempting to take by introducing baffles or ponds in series. Unlike the conventional expression where the nominal detention time is always used, the actual detention time as found in the tracer test was used.

Consequent of the finding from the model tests in order to describe the actual operation in the pond a new expression must be sought. This is better described by combining Wehner and Wilhem equation with algal-bacterial symbiosis. While Wehner and Wilhem model has been investigated by several authors to be the best representative of chemical reactor operation where steady state chemical reactions take place as in stabilization pond, the symbiosis between algae and bacteria in such ponds offers the maximum potential for water quality treatment (Levenspiel 1962, and Thirumurthi, 1969). Although the last concept has always been based on high rate aerobic pond designed to have vigorous AGP

(Algal-growth potential) and completely mixed pond, here the following assumptions are made: (i) the anaerobic zone is quite relatively very small compared with the aerobic zone as a result of the pond depth (25cm); (ii) due to the introduction of baffles, there is always proper mixing and (iii) the algae are homogeneously dispersed within the pond. Other assumptions for the entire process are first order kinetic reaction, and C and C_e are maximum expected influent and effluent BOD respectively.

The first step is to find the hydraulic property of the pond. From such test the magnitudes of t and d can be evaluated. The only unknown value, K can be estimated from Wehner and Wilhem equation using the computer programme given in Appendix A or through Thirumurthi chart (Fig. 3-2-2).

The dispersion number, d varies with flow rate, Q; size of the reactor and length of travel, L_t. (Fig. D-1).

$$d = D/uL \qquad \text{..... D-13}$$

The fluid velocity, u can be estimated from the flow rate Q and the tube cross-sectional area, a.

$$Q_1 = Q_2 \quad \text{theoretically}$$

Actually

$$Q_1 = 1.004Q_2 \text{ that is } 3.33 \text{ L/d as against } 3.30 \text{ L/d}$$

$$u = Q/a \qquad \text{..... D-14}$$

$$L_t = (N + 1)(W - X) \qquad \text{..... D-15}$$

where:

A = Pond surface area (m^2)

W = Width of the pond (m)

N = Number of baffles

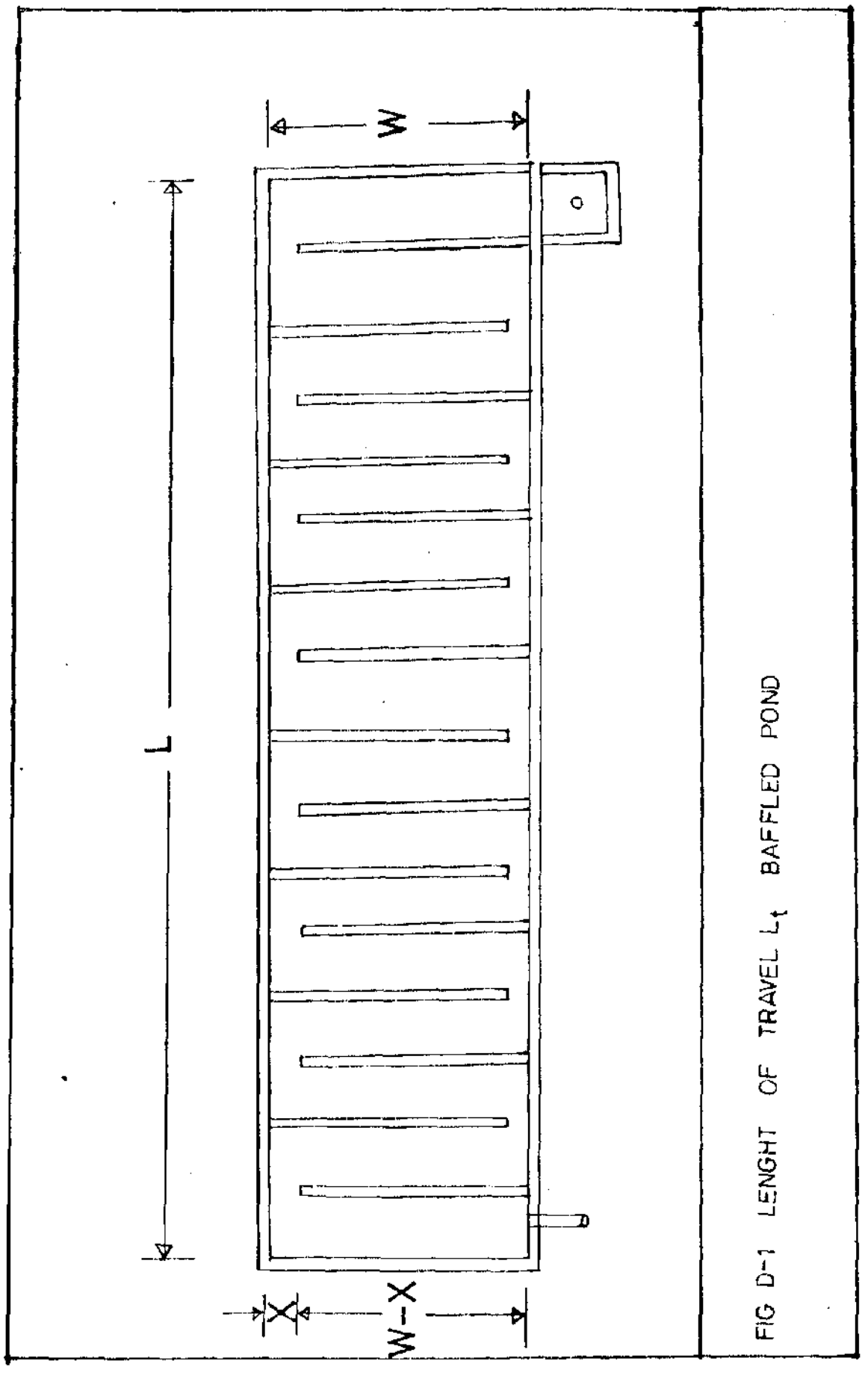


FIG D-1 LENGHT OF TRAVEL L_t BAFFLED POND

X = Space available for fluid flow between the baffle and the side of the pond (m).

Q_1, Q_2 = Influent and Effluent flow rate respectively.

For any single value of d, there is only one corresponding value of C_e/C satisfied by only one single value of Kt. So if the reactor is to be scaled up for the same efficiency and Kt it only implies that d is fixed.

For, d, Kt = Constant

ut = L D-16

Kt = K(L/u) D-17

i.e t = L/u = Constant D-18

If the length of travel, L is increased by increasing the number of baffles and/or size of the pond, the fluid velocity has to be increased proportionally too. Consider a reactor with initial fluid velocity u and length of travel, L scaled up to a velocity u^1 with a corresponding length of travel L^1.

$$d = D/uL \quad \text{.... D-13}$$

$$d^1 = D/u^1L^1 \quad \text{.... D-19}$$

For $d = d^1$, d^1 must be divided by a constant, C, ($C < 1.0$)

Recall:

$$uL < u^1L^1$$

For any value of d:

$$C = \frac{D}{u^1L^1} / D/uL = \frac{uL}{u^1L^1} \quad \text{.... D-20}$$

$$uL = C\,u^1L^1 \quad \text{.... D-21}$$

Generally,

$$C = (L/L^1) \quad \text{.... D-22}$$

For any given L^1 and holding d constant to achieve the efficiency obtained from previous control test with L, the right flow rate can be obtained bearing in mind (D-14) and (D-15). From (D-21) knowing C, L^1, u and L from the control test.

then

$$u^1 = uL/CL^1 = \frac{L^1}{L}^2 \frac{uL}{L^1} \qquad \ldots\ldots\ldots \text{D-23}$$

$$u^1 = \frac{L^1}{L} u \qquad \ldots\ldots\ldots \text{D-24}$$

whence, $Q' = \frac{L'}{L} Q = Q/C$

Having established the values of K, d, t_g, the number of baffles or pond size and the flow rate, then the following process design standard can be calculated in terms of algal activities.

Ultimate BOD, L_o

$$L_o = \frac{X}{1 - 10^{-kt}} \qquad \ldots\ldots\ldots \text{D-25}$$

$$O_2 = f(\text{BOD}) \qquad \ldots\ldots\ldots \text{D-26}$$

O_2 = the weight of photosynthetic O_2 that must be produced for BOD satisfaction.

$$\text{BOD removal} = \text{Efficiency} \times L_o \qquad \ldots\ldots\ldots \text{D-27}$$

(at 24°C)

$$\text{Total BOD removal, } C_a = \text{BOD removal} \times Q \qquad \ldots\ldots\ldots \text{D-28}$$

(mg/d)

The weight of algae required, C_c

Theoretically:

$$C_c = \frac{f}{p} (\text{Total BOD Removal}) \qquad \ldots\ldots\ldots \text{D-29}$$

$0.125 < f < 2.27$; $1.25 < P < 1.75$

Assume f = 1.1 and p = 1.5

C_c = 0.7 (Total BOD Removal)

$$= 0.7\ C_a \qquad \text{..... D-30}$$

Actual (i.e from the data)

$$C_c^1 = C_c \qquad \text{.... D-31}$$

C_a is based on the entire pond volume, hence, C_a in D-30 must be corrected to reflect the dilution.

Using Beer Lambert's Law, the light penetration, D^1 is

$$D^1 = \ell n \frac{I_o}{24} / h\ C_c \qquad \text{..... D-32}$$

The extinction coefficient, h = 0.0015 (Oswald, 1963).

The depth D^1 in D-32 relates only to algae depth. Hence

$$C_a = 1.43 C_c D^1/D \qquad \text{..... D-33}$$

Substitute D^1 in D-32 into D-33

$$C_a = \frac{T_c\ \ell n\ I_o}{\frac{24}{hD}} \qquad \text{..... D-34}$$

where, $T_c = -0.00328T^2 + 0.1467T - 0.6363$

when $D^1 = D$, D-33 reduces to D-30, or

$$C_a = 1.43\ C_c \qquad \text{.....D-35}$$

Total Energy required for algal growth, H

Weight of algae (mg/d) C_c x Heat of combustion h (cal/g)

Theoretically:

$$H = C_c h \qquad \text{..... D-36}$$

Actual

$$C_c' h \qquad \text{..... D-37}$$

Average available daily solar radiation,

I cal/cm^2-d

The quantum energy, E_q that will be stored in the algal cells

$$E_q = F' S \quad \text{.... D-38}$$

F' = conversion efficiency (0.02 - 0.09)

S = visible solar energy

Total area of pond required (cm^2)

Theoretically:

$$H/E_q = C_c h/F'S \quad \text{.... D-39}$$

Actual:

$$H/E_q = C_c^1 h/F'S \quad \text{.... D-40}$$

BOD applied per unit surface area (mg/cm^2-day)

Theoretical:

$$\frac{L_o}{C_c h} (Q)F'S \quad \text{.... D-41}$$

Actual:

$$\frac{L_o}{C_c^1 h} (Q) F'S \quad \text{.... D-42}$$

Volume of the pond, V

$$V = Qt \quad \text{.... D-43}$$

Depth, D

$$D = V/A \quad \text{..... D-44}$$

Effective Volume, $\forall$

$$\forall = Qt_g \quad \text{..... D-45}$$

t_g = Mean (Actual) detention time (day)

$t_g \geq t$

t = Nominal (theoretical) detention time

APPENDIX E

CULTURED COLIFORMS

TOTAL COLIFORM:

MF-Endo media contains lactose, protein digest, vitamins, selective chemicals, and Schiff's Reagent. As the membrane incubate for 24 hours at 35°C + 0.5, the media diffuses through the pores in the filter, supplying nutrients to the multiplying bacteria. Many kinds of bacteria from the water sample can grow and form colonies under these conditions, but only the coliform will ferment lactose. One by-product of this reaction is an acid aldehyde complex that will combine with the Schiff's Reagent to form an iridescent green coating over the growing colonies with a greenish-gold "sheen" when viewed under microscope.

FAECAL COLIFORM

The media, M-FC used contains lactose, protein digest, vitamins, selective chemicals and aniline blue dye. The membrane is incubated for 20 hours at 44°C following pre-incubation of 4 hours at 30°C. This higher temperature allows only coliform of faecal origin to grow into visible colonies. The non-faecal coliforms due to heat shock will not grow. As the faecal coliforms grow they ferment lactose, producing acid which reacts with the aniline dye to produce a blue colour. When viewed with a stereomicroscope, all colonies exhibit a blue colour as faecal coliforms.

FAECAL STREPTOCOCCI:

The selective agent in this media (Agar) is sodium-azide. Colonies of FS are distinguished by their ability to reduce triphenyl-tetrazolium chloride in the medium to a red formazan. Incubation period is always 4 hrs a 30°C followed by 44 hrs at 44°C.