A STUDY OF FAECAL COLIFORM REMOVAL IN DANDORA WASTE STABILIZATION PONDS

By

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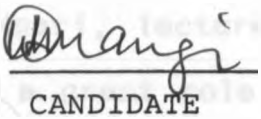
DECLARATION

THIS THESIS IS MY ORIGINAL WORK AND HAS NOT BEEN PRESENTED FOR A DEGREE IN ANY OTHER UNIVERSITY



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THIS THESIS HAS BEEN SUBMITTED FOR EXAMINATION WITH MY APPROVAL AS UNIVERSITY SUPERVISOR

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DEDICATION

TO MY DEAR PARENTS FOR THEIR FAITH AND ENCOURAGEMENT

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ABSTRACT

The study investigates faecal coliform removal both in laboratory and full scale waste stabilization ponds. Although it is widely recognized that waste stabilization ponds are very effective in faecal bacterial removal, there is little published quantitative information about the degree of faęcal bacteria removal in ponds in Kenya. In the study, faecal coliforms were enumerated using the membrane filtration method.

Studies from laboratory scale waste stabilization ponds showed that the die-off rate constant was slower in the ponds with lower initial coliform counts, and that the die-off rate constant decreases down the pond series and with increased in retention time.

Dandora treatment works which is the biggest pond installation in Kenya was used in the full scale study. It was revealed that the works is currently overloaded. The estimated overall retention time is 25.6 days, while the total design retention time is 49 days. The mean overall percentage removal is 99.84%. The die-off rate constants in the ponds ranged from 0.54 d^{-1} to 0.98 d^{-1}. However, it was noted that even with over 99% faecal coliform removal, the number remaining is alarmingly high.

An operation curve for estimating faecal coliform in the final effluent at Dandora Treatment Works when the retention time and raw sewage faecal coliform counts are known is presented.

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CHAPTER ONE

1.0 INTRODUCTION

Continued population growth, especially in urban areas, places increasing demands upon our water resources. Efficient and economical removal of pollutants from waste - water generated from residential, commercial and industrial sources is becoming increasingly critical.

The simplicity and low cost of waste stabilisation ponds have made them an attractive proposition in both developed and developing countries. Stabilisation ponds are now a well established method of biological treatment of wastewater. Wherever suitable land is available at reasonable cost they are usually significantly cheaper than other processes, and they can produce very high quality effluent. Maintenance requirements are very simple and little or no energy (other than solar energy) is needed. Stabilization ponds are thus ideal in developing countries, but they are also suitable elsewhere.

Stabilization ponds are used primarily to reduce biochemical pollution and faecal bacteria contamination in wastewater before discharge to receiving water bodies. The design of waste stabilisation ponds for coliform removal has mostly been empirical. Numerous authors (Sarikaya et al 1987) have indicated a need for more data and study to improve the existing models of coliform die-off in wastewater ponds or in fresh water

systems in general. Like other biological treatments, there are no doubts as to whether Stabilisation ponds can satisfy most effluent standards without disinfection.

Serious water pollution usually produces an obvious deterioration in the quality of the environment and should thus be avoided wherever possible. The potential hazards of faecally contaminated water supplies are too apparent in developing countries and there can be no doubt that efficient water control measures should be high on the list of priorities for developing programmes. In developing countries like Kenya, where effluent discharge is into a river, the bacteriological parameter in the effluent is of more relevance because firstly, most water courses would not be able to provide the dilution that is desirable and secondly, it is likely, that, water will be abstracted directly for domestic purposes downstream.

During the early days of water bacteriology there was a realization of the difficulties involved in trying to demonstrate directly the presence of pathogenic micro-organisms in water, and thus an indirect approach to the problem was developed. It was suggested that water should be examined for evidence of excretal or sewage pollution the assumption being that if pollution of this type were present the water must be regarded as potentially dangerous.

Coliforms were chosen as indicator of water quality, primarily based on the work of Escherich, who in 1885

identified Bacillus coli (from which the name 'Coliform' is derived) as being characteristic of feces of warm-blooded animals. The presence of these organisms in water was assumed to indicate a potential health hazard because of their association in the intestine with a variety of pathogenic micro-organisms: Salmonella, Shigella, Vibrio, Mycobacterium, Pasteurella, Leptospira and enteric viruses. There are two principal groups of coliform bacteria: the latter are exclusively faecal in origin whereas the former, also occur naturally in Soils, Vegetation and water. Thus only the faecal coliforms are definite indicators of faecal pollution.

Although detection of indicator bacteria suggests possible occurrence of pathogenic organisms in water, the potential health hazard is dependent on retention of critical density levels and associated virulence for the pathogens in a given time frame during transmission via the water route. The removal of enteric bacteria from the aquatic environment is no doubt the result of a combination of several factors which are said to influence either the rate of removal or extent of removal. Among the most important are: Temperature, predators of enteric bacteria, sedimentation, solar radiation, pH. antagonistic environmental conditions, and the required nutrients.

Dandora sewage treatment works is the major treatment works for Nairobi and is the largest pond installation in Kenya. No investigation on its efficiency in faecal coliform removal has been previously carried out. The

objectives of the present study were: To investigate the efficiency of Dandora waste stabilization ponds and laboratory scale waste stabilization ponds in faecal coliform removal, to estimate the faecal coliform die-off constants both in Dandora waste stabilization ponds and lastly to compare the results obtained with those reported in Literature.

CHAPTER TWO

2.0. WASTE STABILIZATION PONDS

Wastewater treatment involves the separation of the solid fraction from the liquid phase, treating the solids and the liquid arising from this separation to reduce as far as possible the organic pollutants. This allows final disposal of the stabilized wastewater constituents into the environment without detrimental effects. Stabilization ponds provide suitable treatment and are, in addition, very effective in removing pathogens.

2.1 DEFINITION OF WASTE STABILIZATION PONDS

Waste stabilisation ponds are large shallow basins enclosed by earthen embankments in which raw sewage is treated by entirely natural processes involving both algae and bacteria (Mara 1976). Since these processes are unaided by man the rate of oxidation is rather slow and as a result long hydraulic retention times are employed, 30-50 days not being uncommon. Stabilisation ponds, e.g. facultative ponds, maturation ponds and even anaerobic ponds are reported to be effective in removal of the coliform bacteria (Polprasart 1983).

2.2 CLASSIFICATION OF WASTE STABILIZATION PONDS

The major classification of waste stabilisation ponds include the following:

(a) The Facultative ponds

(b) The Maturation ponds

(c) The Anaerobic ponds

2.2.1 FACULTATIVE STABILIZATION PONDS

Facultative ponds are those in which the upper layer is aerobic, the central zone supports both aerobic and anaerobic bacteria (facultative) whereas the bottom sludge has anaerobic conditions (Gloyna 1971). Although some of the oxygen comes from re-aeration through the surface, most of it is supplied by the photosynthetic activity of the algae which grow naturally in the pond where considerable quantities of both nutrients and incident light energy are available. The pond bacteria utilize this oxygen to oxidize the organic matter. One of the major end-products of bacterial metabolism is carbon dioxide which is readily utilized by the algae during photosynthesis. Thus there is an association mutual benefit (symbiosis) between the algae and bacteria in the waste stabilization pond. (Mara 1976)

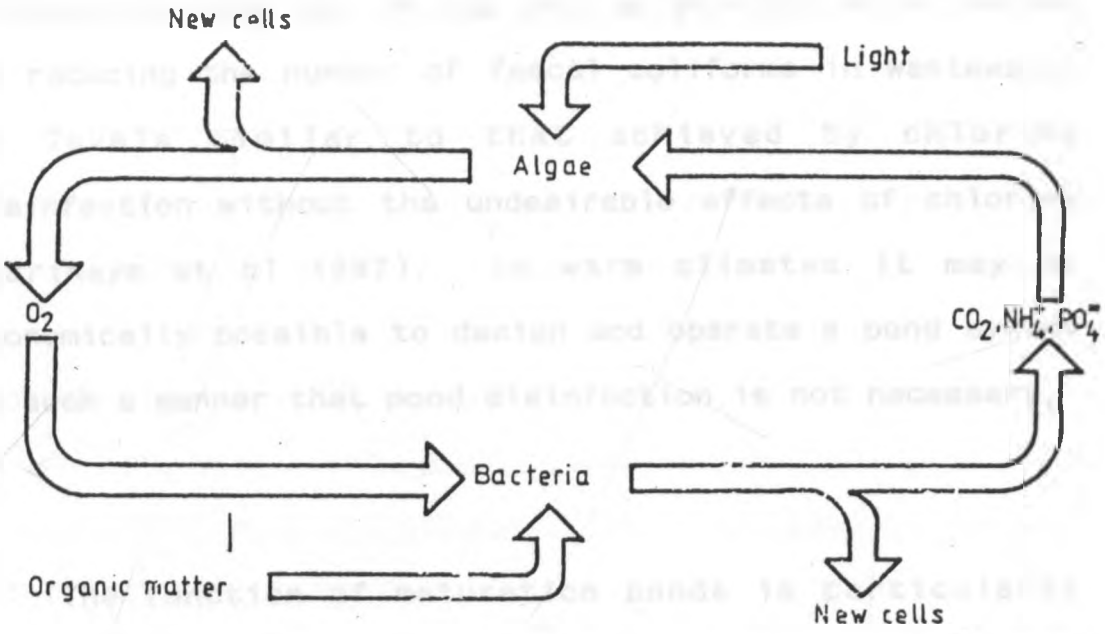


FIG. 2.2.1 ALGAE-BACTERIA SYMBIOSIS IN WASTE STABILIZATION PONDS (MARA 1976)

2.2.2 MATURATION PONDS

The primary function of maturation ponds is to reduce the number of pathogenic bacteria, viruses and the cysts and of intestinal parasites through extended detention time (Parpia et al 1973). The maturation ponds have been proposed and applied in the past as an alternative method of reducing the number of faecal coliforms in wastewater to levels similar to that achieved by chlorine disinfection without the undesirable effects of chlorine (Sarikaya et al 1987). In warm climates it may be economically possible to design and operate a pond system in such a manner that pond disinfection is not necessary.

The function of maturation ponds is particularly vital in tropical countries where the wastewater plant effluent is directly discharged into a stream which is also used as a source of drinking water without treatment. Maturation pond treatment reduces the growth of fungal and filamentous bacteria in the receiving water courses (Marais 1974). Maturation ponds have been advantageously used to rear fish (Mara 1976).

2.2.3 ANAEROBIC WASTE STABILISATION PONDS

Anaerobic ponds are deep ponds designed to receive a high-strength wastewater which can be degraded in the absence of dissolved oxygen. They are most advantageously used for pretreatment of strong wastewater before the partially clarified effluent is discharged into a facultative pond for further treatment. Although most designers systematically use anaerobic ponds for pretreatment of domestic wastewater prior to secondary treatment, they are very suitable for treatment of strong industrial wastes (Kilani 1985).

2.3 STABILIZATION PROCESSES AND FACTORS AFFECTING STABILIZATION PONDS

2.3.1 STABILISATION PROCESSES

The decomposition of organic material may take place under aerobic or anaerobic conditions. The aerobic process requires a continuous supply of free dissolved oxygen and is the most efficient method of reducing the organic content of dilute liquid wastes. However, where solids must be liquefied and where the waste are highly concentrated, as in the case of settled organic solids from domestic wastewater, night-soil, and wastes from

abattoirs, the anaerobic process is extremely effective.

2.3.1.1 AEROBIC PROCESS

In the aerobic metabolism of organic matter, much of the carbon serves as a source of energy for the organisms and is respired as carbon dioxide (CO_2). The organisms involved are mostly bacteria, but also include fungi and protozoa. They use the remainder of the carbon, together with phosphorus and nitrogen, to form new cells. The major reactions likely to occur in an aerobic waste stabilisation pond system are as follow:

$(CH_2O)_X + xO_2 \rightarrow xCO_2 + xH_2O$.

Protein (organic N) -> Ammonia -> Nitrite -> Nitrate.

Organic Sulfur -> Sulphate

Organic phosphate -> H_3PO_4 -> $CaPO_4$ (Gloyna 1971)

In aerobic pond systems or in the aerobic zone of the facultative pond system, the free dissolved oxygen is produced by algae during photosynthesis. Photosynthesis is essentially the process by which some organisms (algae) utilize light energy for synthesis of the sugars required for their metabolism, from carbon dioxide and water, producing oxygen as an end product.

2.3.1.2 ANAEROBIC TREATMENT PROCESSES

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

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

$5(CH_2O)_x \rightarrow (CH_2O)_x + 2CH_3COOH + Energy.$

(2) The growth of methane bacteria results into the decomposition of organic acids into methane and carbon dioxide.

$2\ 1/2CH_3COOH \rightarrow (CH_2O)_x + 2CH_4 + CO_2 + Energy.$

Both aerobic and anaerobic waste stabilization processes discussed above will usually take place in a facultative pond.

2.3.2 FACTORS AFFECTING STABILISATION PONDS

Stabilisation ponds being installations that are considerably influenced by the forces of nature, the waste stabilizing process in the systems is dictated or

influenced considerably by a host of natural, physical, chemical and biological factors. Some of these factors are briefly discussed below.

2.3.2.1 SUNLIGHT

The photosynthetic process in aerobic and facultative stabilisation ponds is light dependent, since photosynthesis cannot occur in the absence of light. Fortunately it is not necessary to keep a pond aerobic for its entire depth. It is difficult to ensure full light penetration because utilization of light energy in waste stabilisation ponds is complicated by many factors.

Gloyna (1971) observed that at low light intensities the efficiency of light utilization by algae is almost linear and that photosynthesis is primarily controlled by light intensity. At higher intensities there is a saturation plateau where increased light does not increase photosynthesis. He also suggested that light transmission and light saturation relationships in ponds could be represented by equations 2.3.1. and 2.3.2 respectively.

$$I = I_o e^{-c'Nad} \quad \text{---------} \quad 2.3.1$$

$$Fa = \frac{I_S}{I_o} \left(\ln \frac{I_o}{I_s} + 1 \right) \qquad \text{2.3.2}$$

Where:

I	=	Light intensity after passage through media (erg/cm^2/s).
I_o	=	Original light intensity (erg/cm^2/s)
C'	=	light absorption coefficient (cm^2/mg)
Na	=	concentration of algal cells (mg/cm^3
d	=	pond depth (cm)
Fa	=	Fraction of light that is utilized by the algae.
I_s	=	Light saturation intensity (erg/cm^2/s)

2.3.2.2 TEMPERATURE

Temperature is highly important in the design of waste stabilisation ponds. Arthur (1983) observed that the pond liquid temperature is probably the parameter which has the greatest bearing on pond performance, and is usually two to three degrees centigrade above the average ambient temperature. Temperature affects photosynthetic oxygen production as well as other biological reactions. Gloyna (1971) suggested 4-37^{o}C as the temperature range

for algal growth. However he noted that some algal species can tolerate higher temperatures and also that some have been observed to grow quite well under a cover of ice.

Biological reactions, within limits, tend to follow well-known chemical rate relationships. The time-temperature relationship can be conveniently expressed as follows.

$$\frac{R_T}{R_o} = \theta^{(To-T)} = \frac{K'_{To}}{K'_T} \qquad \text{----------2.3.21}$$

Where:

R_o = detention time for some specified temperature, To, for which θ or the rate coefficient are known (days).

θ = temperature reaction coefficient

K'_{To} = removal rate coefficient for temperature T_o (per day).

From the above equation (2.3.21), it can be observed that as the temperature increases up to a given limit, the total retention time needed to achieve the same BOD

reduction decreases. Bottom conditions are normally improved by increases in temperature, but conditions at the top may not be improved because undesirable flora frequently develop at water temperatures exceeding 37°C. Some of the beneficial green algae do not appear to function efficiently at temperatures higher than 37°C. Furthermore it is likely that ponds with higher water temperatures will be a little more sensitive to shock or sudden increases in the quantity of organic material and will be subject to less efficient BOD removal rates. It must also be recognized that bacterial activity becomes more intense at higher temperatures, whereby the dissolved oxygen is used at a higher rate.

2.3.2.3. MIXING

Wind and heat are the two factors of major importance which influence the degrees of mixing that occurs within a pond. Mixing fulfils a number of vital functions in a pond: it minimises hydraulic short-circuiting and the formation of stagnant regions and it ensures a reasonably uniform vertical distribution of BOD, algae and oxygen. Mixing is the only means by which the large numbers of non-motile algae can be carried up into the zone of effective light penetration (top 150 - 300 mm of pond)(Mara 1976). Mixing is also responsible for the transportation of the oxygen produced in the zone of effective light penetration to the bottom layer of the

pond. Good mixing thus increases the safe BOD load that can be applied to a pond.

In the absence of mixing thermal stratification quickly occurs. The warm upper layers are separated from the cold lower layers by a thin static region of abrupt temperature change known as the thermocline. Stratification is usually characterized by a substantial reduction in the numbers of algae in the zone of effective light penetration and by a consequent reduction in oxygen production and hence waste stabilization.

2.3.2.4. pH

The pH of the pond contents follows a daily cycle increasing with photosynthesis to a maximum which may be as high as 10 (Mara 1976). This happens because at peak demand algae remove CO_2 from solution more rapidly than it is replaced by bacterial respiration: as a result bicarbonate ions present dissociate to provide not only more Co_2 but also the alkaline hydroxyl ion which increases the pH value:

$$HCO^-_3 \rightarrow CO_2 - OH^-$$

Gann et al as reported by Mara (1976) observed that the optimum growth of bacterial population predominated by

Pseudomonas p- Achromobacteria-flavobacterium group, lies between 7.2 and 7.5 and will die off as pH less or equal to 5.5.

2.3.2.5 TOXICITY

In many cases, toxicity is dependent on concentration. Elements which are vital and necessary in minute concentrations for the growth of an organism may become toxic to some organism if present in higher concentrations. Toxicity may interfere greatly with the enzymatic system in ponds. It may reduce metabolic rate and present an unfavourable environment for optimum growth of micro-organisms. Toxicity contributes to the inability of stabilisation ponds to be effectively used to treat some industrial wastewaters, as the waste to be treated should not only contain organic matter suitable as food for micro-organism but should also be free of any material that is toxic to their living and growth.

2.3.2.6 MICRO-ORGANISMS

The microorganisms are generally classified into two major groups, plants and animals. The plants are made up of virus, rickettsiae, bacteria, fungi and algae; while the animals consist of protozoa, rotifers, and crustaceans (Mckinney 1962). The numbers and types of microorganisms

in sewage will be largely a function of the environment. The micro-organisms upon whose activities the operation of biological waste treatment processes depends are the bacteria, algae and protozoa (Mara 1976).

Some bacteria and a few protozoa are human pathogens, as are many viruses. Microbes, however, have a more positive role to play in sewage treatment. Bacteria are the primary degraders of organic wastes in biological waste treatment plants. The design of such plants should therefore enable bacteria to grow.

Coliform and other intestinal bacteria do not play any significant role in the sewage treatment processes; they are merely passengers in the system.

2.3.2.7 NUTRIENTS

Domestic wastewaters contain all the nutrients required to maintain a bacterial and an algal community. Industrial wastewaters, on the other hand, may not contain sufficient nutrients as they are frequently deficient in nitrogen or phosphorus or both.

When the food is sufficient for optimum bacterial growth it is normally adequate for supporting the algal population as well. The required BOD-phosphorus-nitrogen

ratio is about 100:5:1 (Gloyna 1971). In a pond where seepage is minimal and detention periods are long, however, there may be considerable reuse of both nitrogen and phosphorus by bacteria, algae, and other aquatic organisms.

2.4 HYDRAULIC PROPERTIES OF WASTE STABILISATION PONDS

Treatment performance of waste stabilisation ponds is a function of both the hydraulic transport and the biological and chemical transformation processes within the pond. Various theoretical and empirical models have been used to describe the hydraulic process (Raymond et al 1981). The transport models which have been used to describe ponds include: plug flow, completely-mixed flow and dispersion flow. The first two represents two ideal conditions. In practice the hydraulic regimes lies between these extremes and is described as dispersion flow.

2.4.1 PLUG FLOW

In this type of flow, longitudinal diffusion and mixing are not considered, i.e. a given mass of liquid passes through the tank without mixing. All elements in the system travel at the same velocity without dispersion (Raymod et al 1981). The plug flow model is given by the

following formulae.

$$\frac{Le}{Li} = e^{-kt} \qquad \text{--------} \quad 2.4.1$$

Where Le = effluent concentration

Li = influent concentration

k = first order reaction

t = mean residence or detention time

This type of flow is characterized by high concentration of reactant at the influent and does not respond well to hydraulic shocks. It has, however, an advantage that to accomplish a given extent of a particular reaction for which the rate increases with increasing concentration of reactant, plug flow reactor requires smaller volume than completely mixed reactor.

2.4.2. COMPLETELY-MIXED FLOW

The contents of the pond are homogeneous and equal to the effluent concentration. The design equation representing the completely mixed reactor, is as follows:

$$\frac{Le}{Li} = \frac{1}{1+Kt} \qquad \text{----------} 2.4.2$$

Equation (2.4.2) is commonly used in designing aerated lagoons, waste stabilization ponds and activated sludge processes. Whenever equation (2.4.2) is incorporated it is assumed that there is complete mixing which is not true in a large stabilisation pond, aerated lagoon, or an aeration tank of the activated sludge process.

2.4.3 DISPERSED FLOW

Wehner and Wilhelm (Mara 1976) derived an equation for chemical reactors which exhibit a non-ideal mixing property.

$$\frac{Le}{Li} = \frac{4a\, e^{1/2N}}{(1+a)^2 e^{a/2N} - (1-a)^2 e^{-a/2d}} \qquad \text{--------2.4.3}$$

$$\text{Where, } a = \sqrt{1+4td} \text{ and } d = \frac{D}{UL} = \frac{Dt^*}{L^2} \qquad \text{------- 2.4.4}$$

in which d = diffusivity constant or dispersion number (dimensionless), D = axial dispersion coefficient, V = fluid velocity, L = characteristic length of travel path of a typical particle in the tank. Biological waste

treatment systems should be designed using the equation (2.4.3), short-circuiting in tanks, exit and entrance hydraulic devices and other hydraulic mixing characteristics can by represented by the value of d. (The values of d are zero and infinity, respectively, for plug flow and completely mixed systems).

The temperature, influent waste qualities, nutrient deficiencies, organic load and other biological factors can be accounted for by the value of K. The hydraulic load of course is represented by the value of the actual (mean) residence or detention time (t).

The design equation (2.4.3) may look complicated for use by the design engineer. However, charts may be prepared (Thirumurthi Charts) for ready use, thereby avoiding calculations. See Figure (2.4).

2.5 DESIGN OF WASTE STABILISATION PONDS

Ponds have been used for centuries to store and treat animal and house-hold wastes. However, only within the last two decades have specific design criteria been developed in terms of volumetric requirements, organic loading rates, and detention periods (Gloyna 1971). Significant studies on bacteria reduction in ponds, operational practices and the toxicity of industrial

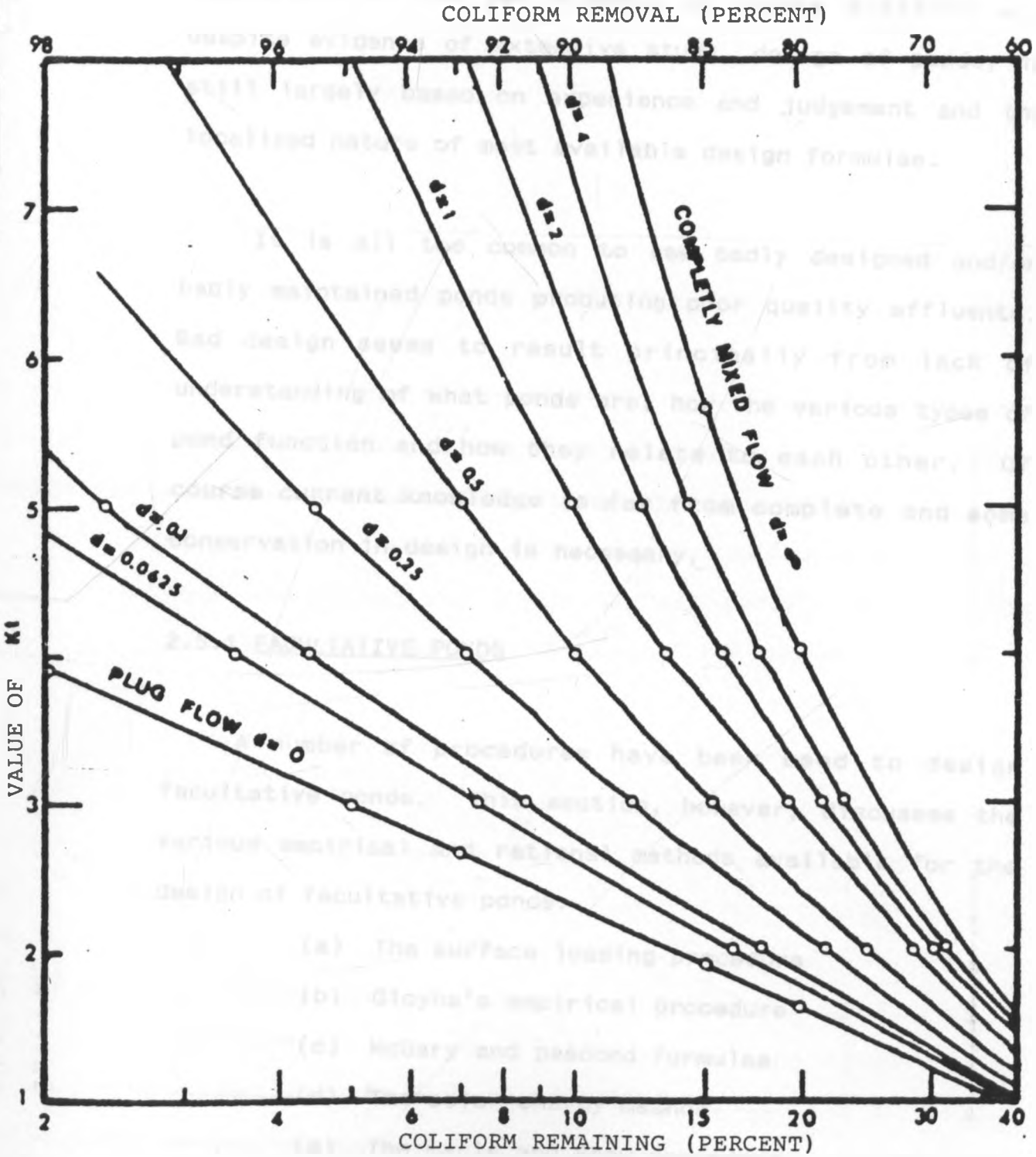


FIG. 2.4 THIRUMURTHI CHART FOR WEHNER-WILHELM EQUATION 2.43 (Thirumurthi, 1969)

wastes have produced basic data for better designs and operations.

The complexities of the waste stabilisation process in ponds and the waste influence of various climatic conditions on the performance of ponds explains why despite evidence of extensive study, design of ponds, is still largely based on experience and judgement and the localised nature of most available design formulae.

It is all too common to see badly designed and/or badly maintained ponds producing poor quality effluents. Bad design seems to result principally from lack of understanding of what ponds are, how the various types of pond function and how they relate to each other. Of course current knowledge is far from complete and some conservation in design is necessary.

2.5.1 FACULTATIVE PONDS

A number of procedures have been used to design facultative ponds. This section, however, discusses the various empirical and rational methods available for the design of facultative ponds.

(a) The surface loading procedure

(b) Gloyna's empirical procedure

(c) McGary and pescond formulae

(d) The solar energy method

(e) The maris and Shaw procedure.

(f) Indian empirical procedure

The rational methods include:

(a) The first order kinetics

(b) The procedure suggested by Thirumurthi(1969)

2.5.1.1 THE EMPIRICAL METHODS

(a) THE SURFACE LOADING PROCEDURE

Experience has shown that certain generalizations can be made concerning the acceptable organic load ( ) of a facultative waste stabilisation pond. Table 2.5.1 shows BOD loading values that have been used successfully in various geographical areas, but obviously great care must be exercised in using these values for design purposes.

TABLE 2.5.1: GENERALIZED BOD LOADING PER UNIT AREA PER DAY UNDER VARIOUS CLIMATIC CONDITIONS

SURFACE LOADING Kg BOD_5/ha per day	POPULATION per ha	DETENTION TIME (days)	ENVIRONMENTAL CONDITIONS
Less than 10	Less than 10	More than 10	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 temperate summer temperatures for short season.
50-150	1000-3000	100-33	Temperate to semi-tropical, occasional ice cover, no prolonged cloud cover
150-350	3000-7000	33-17	Tropical, uniformly distributed sunshine and temperature, and no seasonal cloud cover

(b) GLOYNAS'S EMPIRICAL PROCEDURE

Using the results of many small laboratory ponds, larger pilot plants and over 200 operating ponds, Gloyna (1971), developed a design formulae relating pond volume to temperature, influent BOD and wastewater flow as shown below:

$$V = (3.5 \times 10^{-5}) Q L_u \theta^{(35 - T_m)} ff' \text{ ------- } 2.5.1$$

Where:

V = pond volume (m^3)

Q = flow rate (L/d)

L_u = Ultimate influent BOD_u(mg/L) or COD

θ = temperature coefficient

f = algal toxicity factor

f' = sulphide or other immediate chemical oxygen demand.

T_m = Temperature

Gloyna suggested that the algal toxicity factor, f can be assumed to be equal to 1.0 for domestic wastes and many industrial wastes, and the sulphide oxygen demand could be taken to be 1.0 for sulphate ion of less than 500

mg/L. Although reported values for temperature reaction coefficient, θ, vary from 1.036 to 1.085, Gloyna recommended the use of a value of 1.085.

The guidelines shown in table 2.5.2. were recommended for the determination of pond depth to be used in conjunction with equation 2.5.1. The pond surface area, which is considered to be more critical in the case of facultative ponds than volume, may be determined from:

$$A = \frac{V}{d} \qquad \text{--------} \quad 2.5.2$$

Once the depth, d where A = Surface Area

V = volume

d = depth

TABLE 2.5.2: RECOMMENDED DEPTHS OF FACULTATIVE PONDS IN RELATION TO ENVIRONMENTAL CONDITIONS AND TYPE OF WASTE

RECOMMENDED DEPTH (M)	ENVIRONMENTAL CONDITIONS AND TYPE OF WASTE
1.0	Uniform warm temperature; presettled waste-water
1.0 - 1.5	Uniform warm temperature; untreated waste-water
1.5 - 2.0	Moderate seasonal temperature fluctuations; raw waste-water containing settleable solids
2.0 - 3.0	Wide seasonal temperature variations; large amounts of settleable grit or settleable solids.

(c) McGARY AND PESCOD FORMULAE

McGary and pescod as reported by Mara (1976) showed that the maximum BOD_5 surface loading that could be applied to a facultative pond before it failed was related to the mean monthly ambient air temperature as follows:

$$\vec{\lambda}_s = 11.2(1.054)^T \text{ ------- } 2.5.3$$

Where:

$\vec{\lambda}_s$ = maximum BOD_5 loading, kg/ha.d

T = temperature, ^oF

Since ponds are not normally designed to operate just at their point of failure, a safety factor is needed in equation 2.5.3 for design purposes. Mara (1976) suggested a factor of safety of 1.5. Hence the design equation for the BOD_5 loading λ_s is as follows:

$$\lambda_s = 7.5(1.054)^T \text{ ----------2.5.4}$$

Where λ_s = design loading, kg/ha.d

Assuming a straight line relationship between λ_s and temperature, McGary and Pescod also obtained, from

regression analysis, an alternative design formula for λ_s as follows:

$$\lambda_s = 20T-120 \quad \text{----------2.5.5}$$

Where T is in ^{o}C.

The design equation for A is then simply obtained from equation 2.5.5 as:

$$A = \frac{LiQ}{2T-12} \quad \text{------ 2.5.6}$$

Arthur (1983) considered equation 2.5.5. too conservative and recommended:

$$\lambda_s = 20T-60$$

For temperatures below $20^{o}C$

(d) THE SOLAR ENERGY

The basis of this procedure is the relation between daily BOD removal and the daily oxygen production by algae in a pond. In facultative ponds, algae photosynthesis accounts for the bulk of the oxygen content. Thus oxygen production in facultative ponds depends on algal concentration and the efficiency of light energy

conversion, which in turn depends on the light penetration and algal species present.

The energy requirements for the formation of one gram of algae, as reported in literature, varies from 4 to 6 kilo-calories and solar energy conversion efficiency varies from 2 to 10 percent. Using a value of 6 kilocalories as the energy required for the formation of one gram of algae and a value of 6 percent for the solar energy conversion efficiency, an expression relating weight of algae produced to available solar radiation has been developed as follows:

$$\text{mass of algae (m)} = \frac{I_L(\text{langleys}) \times 10^8(\text{cal/ha/d}) \times 6}{6 \times 10^6 \text{ cal kg} \times 100} \times (\text{Efficiency}) \qquad \text{-------2.5.8}$$

Where:

I_L = Intensity of solar radiation langleys/day or $cal/cm^2/day$.

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

$$O = 1.6\ I_L \text{ ----------2.5.9}$$

Where:

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

Joyngoudar et al (Mara 1976) equates the oxygen production and the removal of ultimate BOD in a pond. Oswald and Gotaas (Mara 1976) uses a relatively conservative criterion of equating the oxygen production with the ultimate BOD applied (rather than removed) per ha/day. Thus equation 2.5.9 can be written as:

$$BOD_{ult} = 1.6\ I_L \text{ ------- 2.5.10}$$

Where:

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

Rewriting equation 2.5.10 in terms of BOD_5 loading kg/ha/day, L

$$L_s = 1.065\ I_L \text{ ----------2.5.11}$$

The mid-depth Area, A, can be calculated from equation

2.5.12

$$A = \frac{10 L_i Q}{L_s} \qquad \text{-------- 2.5.12}$$

Where:

A is in m^2, L_i in mg/L. Q in m^3/day and L_s in kg/ha/day.

(e) MARAIS AND SHAW PROCEDURE

Based on field data from Southern Africa and Southern USA Marais and Shaw related the maximum BOD_5 consistent with the maintenance of predominantly aerobic conditions to the depth as follows:

$$L_e = \frac{N}{2D+8} \qquad \text{----------2.5.13}$$

Where:

N = a constant

D = depth m

L_e = Effluent pond BOD_5 mg/L

According to Gloyna (1971) Marais and Shaw also fixed a value of N = 600 for the purpose of design, although field data indicated a value of N = 1000. Field data also indicated that BOD could be formulated in terms of equation 2.5.14

$$L_e = \frac{Li}{K_1 t+1} \qquad \text{----------2.5.14}$$

Where:

L_i = influent pond BOD_5 (mg/L)

t = retention time

K_1 = First order BOD removal rate constant (d^{-1})

In Southern Africa the value of K_1 was found to be 0.23 d^{-1} which Marais and Shaw reduced for design purposes to 0.17 d^{-1}.

Marais (Mara 1976) subsequently modified equation

2.5.14 by allowing K_1, at any temperature T, K_T to be related to the value of K at 35°C which was found to to be 1.2 day^{-1}.

$$K_T = 1.2\ (1.085)^{T-35} \quad \text{------} \quad 2.5.15$$

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

$$\frac{K35}{K_T} = \theta^{35-T} = \frac{RT}{R_{35}} \quad \text{---------} \quad 2.5.16$$

Substituting equation 2.5.15 into 2.15.4

$$Le = \frac{L_i}{1+1.2\ (1.085)^{35-T}} \quad \text{------} \quad 2.5.17$$

The values $K_{35}(1.2)$ and θ (1.085) were obtained from laboratory model ponds fed with a synthetic milk waste and cannot be used with confidence for the design of full-scale ponds.

Mara (1976) suggested that a suitable design equation for K_T was equation 1.5.16

$$K_T = 0.30 \quad (1.05)^{T-20} \quad \text{---------} \quad 2.5.18$$

Here the reference temperature is $20^{o}C$ and the design value of K_{20} is conservatively taken as 0.30 d.

(f) INDIAN EMPIRICAL PROCEDURE

Experience of pond operation in India has yielded a design which relates the permissible loading to latitude as follows: (Mara 1976):

$$= 375-6.25L \quad \text{-------} \quad 2.5.19$$

Where L = latitude (range in India: $8-36^{o}N$)

2.5.2.1. THE RATIONAL METHODS

The rational design procedure hinge around the influence of the hydraulic characteristics or properties of stabilization ponds on the BOD removal capacity (or efficiency) of the ponds. The three possible hydraulic flow patterns that could describe wastewater flow through a biological reactor have been identified as plug flow, complete mixing and dispersed flow. The rational design procedures mostly used for design of facultative ponds assume either a complete mixed flow reactor or a dispersed

flow reactor.

(a) COMPLETE MIX FLOW DESIGN EQUATION

As mentioned in section 2.4.2, the complete mix flow reactor is considered as one in which the contents of the reactor undergo instantaneous and complete mixing, and the effluent from the reactor is identical in every respect to the contents of the reactor. A mass balance equation written across such a reactor results in the following equation describing BOD removal in the reactor.

$$\frac{L_e}{L_i} = \frac{1}{1+K_1 t^*} \qquad \text{-------- 2.5.2.1}$$

Where:

L_i = influent BOD_5 in mg/L

L_e = effluent BOD_5 in mg/L

K_1 = First order BOD removal rate constant (d^{-1})

t = is the detention time in days.

(b) DISPERSED FLOW EQUATION

The actual flow through ponds has been observed and reported to be non-ideal, since although some mixing does exist, the contents of ponds are far from being completely mixed.

Thirumurthi (1969) suggested that the removal of BOD from ponds, exhibiting non-ideal or dispersed flow pattern may be described by the following chemical reactor equation developed by Wehner and Wilhelm.

$$\frac{L_e}{L_i} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}} \qquad \text{-----25.2.3}$$

Where: $a = \sqrt{1+4K_1 t}\ d$

$d = \frac{DL}{UL} = \frac{D_L t^*}{L^2}$ = a dimensionless dispersion number.

D_L = coefficient of longitudinal dispersion,

m^2/hr

L = Mean path lenghth of typical particle in a reactor, m

U = Mean velocity of travel, m/hr.

The design equation 2.5.2.2 may look complicated for use by the design engineers. However, charts may be prepared (Thirumuthi chart figure 2.4) for ready use, thereby avoiding calculations.

Also as an approximation, the second term in the denominator which is small may be neglected in which case the formula is simplified as:

$$\frac{Le}{Li} = \frac{4ae}{(1+a)^2} \qquad \text{----------2.5.2.3}$$

$$(1-a)$$

$$\frac{\quad}{2d}$$

The error may be significant when value of d exceeds 2.0 However, for waste stabilization ponds, will seldom exceed 1.0 because of low hydraulic loads.

2.5.2 MATURATION PONDS

Maturation ponds, which are largely aerobic should be designed to achieve faecal bacterial removals since the bulk BOD_5 is removed in the anaerobic and facultative ponds. The design procedure assumes that faecal coliform removal is a first order kinetic reaction given by:

$$N_e = \frac{Ni}{1+K_B(T)t^*} \qquad \text{------- 2.5.2.1}$$

Where:

N_e = Bacterial concentration in No. FC/100ml of effluent

N_i = Bacterial concentration in No. FC/100ml

$K_B(T)$ = First order FC removal rate at T^oC in days

t^* = detention time.

Due to this form of removal, it has been shown that removal is more efficient with a greater number of ponds for the same total detention time, and with these ponds each having the same detention time. The following equation may be used for calculation of the valued $K_B(T)$. To simplify matter $K_{B(T)}$ may be assumed to be the same for each pond in the series.

$$K_{B(T)} = 2.6\ (1.19)^{T-20} \quad \text{------} \quad 2.5.2.2$$

For two or more ponds in series, the descriptive equation for effluent bacteria becomes:

$$\frac{N_e}{N_i} = \frac{1}{(K_{B(T)}t_1{*}+1)(K_{B(T)}t_2{*}+1)\text{----}(K_{B(T)}t_n{*}+1)} \quad \text{....2.5.2.3}$$

Where n is the number of ponds.

For a series of n ponds with equal detention time equation 1.5.2.3.becomes

$$\frac{N_e}{N_i} = \frac{1}{(K_{B(T)}t{*}+1)^n} \quad \text{---------- 2.5.2.4}$$

Maturation ponds are generally shallow (1.2 to 1.5 m) to maintain largely aerobic conditions, with the added advantage that viral removals are marginally better in shallow than in deep ponds. Mara (1983) suggested the use of two maturation ponds in series, each with a retention time of 7 days of three or more maturation ponds with retention time of five days.

Although faecal coliforms are commonly used to

indicate the removal of faecal organisms in a series of ponds, there is evidence that some pathogenic bacteria do not die off as quickly as do faecal coliforms - for example, a Salmonella was found to have a K_b value of 0.8 d^{-1} in the same pond as faecal coliforms with a k_b value of 2.0 d^{-1} (Feachem et al 1983). Also drug-resistant coliforms are known to die off more slowly than those without resistance genes.

2.5.3 ANAEROBIC PONDS

(a) EMPIRICAL PROCEDURES

The existing design criteria, procedures of recommendations available in the literature could be summarized under the following four major design parameters.

(1) Depth of pond
(2) Detention time
(3) A real loading rate in terms of BOD_5/h/d
(4) Volumetric loading rate in terms of BOD_5 or volatile solids/m^2/day.

In warm climate of developing countries, Arthur (1983) recommended that Anaerobic ponds should be designed

on the basis of volumetric organic loading between 0.1 and 0.4 kg BOD/m^3/day. Values around 0.1 should be used for areas where there is a pronounced cold season (around 12^oC), and 0.4 where there are uniform annual very warm temperatures (27-30^oC). Mara (1975) recommended that the volumetric loading should not exceed 0.40 kg BOD_5/m^2/day with a loading of 0.25 kg BOD_5/m^2/d being the most favoured.

There is theoretically no limit to how deep an anaerobic pond should be, although a depth of about 4 m is about optimal from the point of view of treatment efficiency. Mara (1975) recommended a depth range of 2 to 4 m. Depth of less than 2.5 m should not be used if possible, although still shallower depths may be necessary due to local soil and ground conditions. In the tropics Gloyna (1971) recommended a liquid detention time of 1-5 days longer detention time may cause the upper layer of the pond to become aerobic and reduce obligate anaerobic conditions necessary for maximum efficiency.

Surface areal loading rates of between 400 and 600 kg BOD/ha/day have been recommended in the literature.

(b) RATIONAL DESIGN EQUATION

Vincent et al as reported by kilani 1985 showed that for tropical and subtropical conditions such as Zambia,

the reduction of BOD in anaerobic pons can be approximated from the expression:

$$L_e = \frac{L_i}{Kn\left(\frac{L_i}{L_e}\right)^{n+1}} \qquad \text{---------3.5.3}$$

Where: L_i and L_e are as defined previously

K_n = Design coefficient (K_n = 6 for Zambia)

n = Constant (n = 4.8 for Zambia)

K = Detention time for completely mixed system

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

2.6 ADVANTAGES AND DISADVANTAGES OF WASTE STABILISATION PONDS

The simplicity and low cost of waste stabilisation ponds have made them an attractive option in both developed and developing countries. Stabilisation ponds are now a well established method of biological wastewater

treatment. Mara (1976) listed the following advantages of waste stabilisation ponds:

1. They can achieve any required purification at the lowest cost and with the minimum of maintenance by unskilled operators.
2. The removal of pathogens is considerably greater than that in other methods of sewage treatment.
3. They are well able to withstand both organic and hydraulic shock loads.
4. They can effectively treat a wide variety of industrial and agricultural wastes.
5. They can easily be designed so that the degree of treatment is readily altered.
6. The method of construction is such that, should at some future date the land be required for some other purpose, it is easily reclaimed.
7. The algae produced in the pond are a potential source of high-protein food which can be conveniently exploited by fish farming.

The major disadvantages of ponds are:

1. They require much larger areas of land than other forms of sewage treatment.
2. The algae in stabilisation pond effluent may impair the quality of the receiving stream as a source of

domestic water supply. Also the green algae colouration in the receiving stream be detrimental to recreational uses of the stream.

3. Poorly maintained anaerobic and facultative ponds, may create serious problems of odour nuisance, vegetation growth, fly and mosquito breeding.

Mara (1976) noted that in hot climates ponds should always be considered the first method of choice for sewage treatment and a very good case must be made for not using them.

2.70 TERTIARY TREATMENT

Tertiary treatment processes originally were not designed primarily for pathogen removal, but some of them do have good pathogen removal characteristics.

2.7.1 RAPID SAND FILTRATION

This is perhaps the most common tertiary treatment found in larger treatment works. High loading rates ($200m^3/m^2/d$) and frequent backwashing (1-2 days) prevent the build up of much biological activity in the filter. The pathogen content of the effluent may be reduced but not substantially, and probably insufficiently to justify investment in this filtration method by the health

benefits it yields.

2.7.2 SLOW SAND FILTRATION

The low loading rates of the filters (2-5 $m^3/m^2/d$) causes them to occupy a large land area. Substantial biological activity builds up, especially in the upper layers of the very high. Removal of 4 log units of excreted viruses and bacteria may be expected from a well-run unit. Complete retention of protozoan cysts and helminth eggs has been recorded. Slow sand filters are therefore highly effective in removing pathogen from a secondary effluent, but their land requirement makes them suitable only for small treatment works.

2.7.3.2 LAND TREATMENT

Secondary effluents may be applied to land in three ways; application to land for deep percolation and ground water recharge, application to land for collection in underdrains, and application to sloping grass plots for first two systems can have extremely high pathogen removal performances (Faechem et al 1983), whereas the grass plot system is less effective because some of the effluent runs over the surface of the soil rather than through it. There is little or no information about the application of these processes in the tropics or in developing countries. If poorly managed, they will probably lead to the creation

of a foul and unsanitary systems pose the potential threat of groundwater contamination.

2.2.4 EFFLUENT CHLORINATION

Effluent chlorination has a number of serious limitations. Regrowth of coliforms and E coli. following chlorination has been widely reported (Deaner, D.G. 1969, Feachem et al 1983) and the regrowth of pathogenic bacteria has not been fully ruled out. Moreover, all bacterial in the effluent are affected by chlorine, many of which are essential for the effluents natural self-purification. If the effluent is discharged into a river or lake, the chlorine may adversely affect the ecology of the receiving water and hinder its natural oxidation processes.

2.7.5 OTHER TERTIARY TREATMENT PROCESSES

Several other tertiary treatment processes are in use or under experimentation, including coagulation, carbon adsorption, irradiation, and ozonation. These processes are in general, too technically complex and costly to be appropriate for sewage treatment in developing countries.

CHAPTER THREE

3.0 MICROBIOLOGY OF WASTE STABILIZATION PONDS

3.1 CLASSIFICATION OF MICROORGANISMS

The principal groups of organisms found in surface water and wastewater are classified as protista, plants and animals. The category protista includes bacteria, fungi, protozoa, and algae. Seed plants ferns, and mosses and liverworts as plants. Invertebrates and vertebrates are classified as animals. Viruses, which are also found in wastewater are classified according to the host infected (Metcalf 1979).

It is desirable that the designers and operators of biological wastewater treatment plants should have at least a superficial understanding of taxonomy, the process of classifying and naming biological organisms. The full classification of an organism contains a large number of elements. The commonly recognized elements in order of decreasing size are

	(TYPHOID BACTERIA)
KINGDOM	Bacterium
PHYLUM	Schizomyletes
CLASS	Prokaryotae
ORDER	Pseudomonadales
FAMILY	Enterobacteraceae
GENUS	Salmonella
SPECIES	Typhi (gottschalk 1979)

The conventional scientific name consists of only two parts - the genus and species. The generic name is always place first and capitalized. i.e. Salmonella typhi, a member of Salmonella genus - bacteria specifically responsible for typhoid.

In bacteriology a species is theoretically a single kind a bacterium, all individual cells of which are identical or nearly so. In actuality this identify of cells rarely exists.

A genus is theoretically and ideally a group of

species all of which bear sufficient resemblance to one another to be considered closely related and easily distinguishable from members of other groups or genera. The boundaries of some genera are sharply defined by as few as three characteristics: as in the genus Bacillus - (1) aerobic (2) endo-sporeforming (3) rods. These are very definite, distinct, constant and readily determined characters. The boundaries of other genera are sometimes more difficult to define for example, the genera Salmonella, Escherichia, Shigella and Aerobactor. All these are nonsporeforming, gram-negative, facultative robs of identical size and appearance, nonpigmented and fermenting glucose. All occur more or less frequently in the intestinal tract and are all motile except Shigella (Frobisher, 1969).

3.1.1 BACTERIA

The bacteria are the smallest organisms of interest in a biological wastewater treatment plant. Consequently, their metabolic rate in high and under optimum environmental conditions they will invariably predominate over fungi and protozoa.

Most bacteria are the same size and survive primarily on their unusual metabolic reactions. The Pseudomonas SP can metabolize almost every type of organic matter and survive in almost every environment. Thus it is the prime

bacterial genus responsible for the degradation of organic matter of sanitary significance. The Alcaligenes and Flavobacterium SP are almost as important as Pseudomonas SP in that they metabolize primarily proteins. Whenever proteins are found as may be in domestic waste-water or with wastewater that contain the cellular releases of dead bacteria Alcaligens SP and Flavobacterium SP will predominate (Mitchell 1972).

3.1.2 ALGAE

Algae are unicellular or multicellular autotrophic, photosynthetic protists (Metcalf et al 1972). It was recognized early by Caldwell as reported by Parker (1979), that a sewage purified in ponds become green and developed a profuse growth of algal species. Photosynthetic action of the algal cells converts CO_2 to organic cell material with the liberation of oxygen.

$$nCO_2 + nH_2O \rightarrow (CH_2O)n + O2 \qquad \text{----------3.1}$$

The oxygen liberated in the dissolved form and under appropriate conditions can produce supersaturation values of 20-30 mg/L. This oxygen is available for saprophytic bacterial respiration with the oxidation of organic carbon compounds to CO_2 and water.

$$Org.C + O_2 \rightarrow CH_2 + H_2O \qquad \text{----------3.2}$$

which in turn generates CO_2 as a basis for further photosynthesis. Typical plate count values at 37°C are 1-2 x 10^6 orgs/mL and at 22°C 2-5 x 10^6 orgs/mL (Parker 1979)

The algal species which develop are limited: In general they are:

True green algae (Chlorophyceae)

Ankistrodesmus

Chlorella

Scenedemus

Flagellates

Euglena

Chlamydomonas

Volvox

Palmella

Diatoms

Nitzehia

Navicula

Blue green algae (cyanophyceae)

Anacystis

Anabaena

Oscillatoria.

Much attention has been focused on the algal populations of ponds because of their key roles as oxygen generators and also because they represent a potentially useful harvestable biomass for use as food and fertiliser. Algal sensitivity to toxic substances in sewage will affect overall pond performance and the species and total biomass present are good indicators of the efficiency and degree of treatment occurring. The appearance of algae in an anaerobic pond will in most cases indicate underloading and the presence of dissolved oxygen in the pond which may inhibit the development of either or both acid forming and methanogenic bacteria (the latter are most sensitive to oxygen). This will lead to impaired pond performance and possibly the production of odours. The only exception to this observation is the presence of a thin film of flagellate algae of the genus Chlamydomonas which often occur even in efficiently operating anaerobic ponds which have not formed a surface crust. It seems that in this case photosynthetic oxygen produced does not penetrate deep enough into ponds to cause a problem and in fact this algal species may well be utilising organic material in the light and producing none or only small amounts of photosynthetic oxygen.

Algal speciation can be used as an indicator of pond type and BOD surface loading (Pearson et al 1986). In general flagellate algal genera (e.g. Euglena, Chlamydomonas, Phacus, Pyrobotrys) predominate in

facultative ponds. The non motile green algae (e.g. Microactinium, Scenedesmus and Chlorella) and diatoms are more dominant in maturation ponds. The number of species also increases with purification such that only two or three species may exist in highly loaded facultative ponds (i.e. above 400 kg BOD_5 $ha^{-1}{}_d{}^{-1}$ in N.E. Brazil). Whereas as many as fifteen species might be present in a final maturation pond of a series with a surface loading of only 10-20 kg BOD_5 $ha^{-1}d^{-1}$. (Pearson et al 1986).

3.1.3 FUNGI

Fungi are anaerobic multicellular plants which are in most cases more tolerant to acid conditions and a drier environment than bacteria. They utilize much the same food as the bacteria in chemosynthetic reactions but, because their protein content is somewhat lower than the bacteria, their nitrogen requirement is less. Fungi form rather less cellular matter than bacteria from the same amount of food. They are capable of degrading highly complex compounds and some are pathogenic to man. Fungi occur in polluted water and in conditions with high C:N ratios (McKinney 1962)

The fungi form normal protoplasm with one-half the nitrogen required by bacteria. Thus, it is not surprising that fungi predominate over bacteria in nitrogen-deficient environment. A nitrogen-deficient environment for the bacteria is not really so deficient for the fungi

(McKinney 1962)

3.1.4 PROTOZOA

The protozoa are unicellular organisms 10 - 100 um in length which reproduce by binary fission. Most are aerobically heterotrophic and often utilize bacterial cells as their main food source. They cannot synthesize all the necessary growth factors and rely on the bacteria to provide these items. The protozoa are widespread in soil and water an may sometimes play an important role in biological waste-treatment processes. There are four main types of protozoa. Sacodina-amoeboid flexible cell structure with movement by means of extruded pseudopodia (false feet); Mastigophora-utilize flagella for motility; Ciliata-Motility and food gathering by means of cilia; Sporozoa-non-motile spore-forming parasites not found in water.

3.1.5 VIRUSES

Viruses are the simplest form of organism ranging in size from about 0.01 to 0.3 um and they consist essentially of Nucleic acid and protein. They are all parasitic and cannot grow outside another living organism. Because of the inability for viruses to grow outside a suitable host they are on the borderline between living matter and inanimate chemicals. Identification and

enumeration of viruses requires special apparatus and techniques. Sewage effluents normally contain significant numbers of viruses and they are also present in most surface waters subject to pollution.

3.2 BIOLOGY OF WASTE STABILIZATION POND SYSTEM

Waste stabilization ponds are the habitat of an enormous variety of living things. All living things found in these ponds reproduce their kind to the extent that food is available (Gloyna 1971).

Purification and stabilization of wastes in ponds is dependent almost entirely on biologically initiated chemical transformations. The operationally controllable parameters are limited to waste loading intensity, pond dimensions, depth and cell arrangement-series or parallel, with or without recirculation. It is recognized that different biological populations proliferate in ponds achieving different stages of purification (Parker 1979). Microbiological groups of concern in pond systems are those relevant to transformation of the materials in the waste, and which proliferate within the pond systems, (bacteria, algae) and those of public health concern present in the raw waste and which can be expected to be reduced or eliminated by the treatment (pathogens, coliforms, helminth, etc.)

The biology of waste stabilization ponds is usually described in terms of the simplified mutualistic relationship between algae and bacteria (Pearson et al 1916). This simple biological model provides the engineer with an adequate explanation for the degradation of organic material in a pond system and the observed reduction in BOD and COD between influent and effluent. It does not however, seek to explain the other vital aspects of sewage treatment in ponds namely the efficient destruction of pathogenic microbes.

3.3. EXCRETED PATHOGENS

Four groups of pathogens - viruses, bacteria, protozoa, and worms - cause human excreta related diseases. In addition, excreta disposal (e.g. ponds, pit latrines and landfill disposal) may favour the breeding of insects, particularly mosquitoes, flies and cockroaches, which will always have nuisance value and may act as vectors of human disease agents that may themselves not be found in feces or urine.

Numerous viruses may infect the intestinal tract and

be passed in the feces, whereupon they may infect new human hosts by ingestion or inhalation. One gram of human feces may contain 10^9 infections virus particles, regardless of whether the individual is experiencing any discernible illness. Although they cannot multiply outside a suitable host cell, the excreted viruses may survive for many weeks in the environment, especially if temperatures are low ($<15^{o}C$). The groups of pathogenic excreted viruses of particular importance are shown in table 3.1.

TABLE 3.1: VIRUS PATHOGENS EXCRETED IN FECES

VIRUS	DISEASE	RESERVOIR
Adenoviruses	Numerous Conditions	Man
Enteroviruses		
Polioviruses	Poliomyelitis, paralysis and other conditions	Man
Echoviruses	Numerous conditions	Man
Covackie viruses	Numerous Conditions	Man
Hepatitis A Virus	Infectious hepatitis	Man
Reoviruses	Numerous Conditions	Man and Animals
Rotaviruses, Norwalk agent and other viruses	Diarrhoea	Probably man

Bacterial pathogens excreted in feces are shown in table 3.2. They most commonly enter a new host by ingestion (in water, on food, on fingers, in dirt) but some may enter through lungs (after inhalation of aerosol particles) or through the eye (after rubbing the eye with faecally contaminated fingers). At some time during the course of an infection, large numbers of the bacteria will be passed in the feces, thus allowing the spread of infection to new hosts.

TABLE 3.2: BACTERIAL PATHOGENS EXCRETED IN FECES

BACTERIUM	DISEASE	RESERVOIR
Campylobacter fetus ssp. jejuni	Diarrhoea	Animals and man
Pathogenic E. coli	Diarrhoea	Man
Salmonella		
S. typhi	Typhoid fever	Man
S. paratyphi	Paratyphoid fever	Man
Other Salmonellae	Food Poisoning and other Salmonelloles	Man
Shigella SPP	Bacillary dysentery	Man
Vibrio		
V. cholerae	Cholera	Man
other vibrios	Diarrhoea	Man
Yersinia enterolitica	Diarrhoea and septicemia	Animals and Man

Many species of protozoa can infect man and cause disease. Among them are several species that are harboured in the intestinal tract of man and other animals, where they may cause diarrhoea or dysentery. Infective forms of these protozoa are often passed as cysts in the feces, and man is infected when he ingests them. Only three species of human intestinal protozoa are considered to be frequently pathogenic: Giardia lamblia, Balantidum coli, and Entamoeba histolytica.

Helminths (except for Strongyloides) do not multiply within the human host, and this is of great importance in understanding their transmission, the ways they cause disease, and the effects of environmental changes on their control.

The helminths are classified into two main groups: the roundworms (nematodes) and those worms that are flat in cross-section. The roundworms may cause mechanical obstruction (Ascaris), rectal prolapse (Trichuris), itching around the anus (Enterobius) or anaemia (hookworms). They also direct food to themselves and produce abdominal pain in some cases (many cases, however, are symptomless). Adult tapeworms create health problems mainly by depriving their host of nutrients. Some flat worms inhabit and damage the liver (clonorchis) or lungs Paragonimus. Schistosomes live outside the intestine in small blood vessels: their eggs that fail to escape from the host may damage several organs.

3.4 FAECAL INDICATOR BACTERIA

Faecal indicator bacteria are selected from among those commensal species that exclusively live in the intestinal tract of man and other warm-blooded animals without causing disease. Because they are always and naturally present in feces and are excreted in large numbers (up to 10^9 or 10^{10} cells per gram of feces) their presence in water indicates beyond doubt that the water has been contaminated with faecal material and possibly with excreted pathogens. If a water is shown to contain faecal indicator bacteria, it is considered unsafe for human consumption. This is the rationale for the bacteriological testing of public water supplies that was developed in Europe and North America at the turn of the century when the major concern of water supply engineers was to reduce the incidence of epidemics of strictly waterborne diseases. It is still an epidemiologically valid testing technique for disinfected water supplies throughout the world. But it has certain limitations when applied indiscriminately in the examination of wastes and wastewaters, particularly in hot climates.

The ideal faecal indicator bacterium should be:

1. A normal member of the intestinal flora of healthy people.

2. Exclusively intestinal in habitat and hence exclusively

faecal in origin when found in the environment.

3. Absent from nonhuman animals (a requirement not met by any of the indicator bacteria currently used).

4. Present whenever faecal pathogens are present, and present only when faecal pathogens might reasonably be expected to be present.

5. Present in higher numbers that faecal pathogens.

6. Unable to grow outside the intestines, with a die-off rate slightly less than that of faecal pathogens.

7. Resistant to natural antagonistic factors and to water and wastewater treatment process to a degree equal to or greater than that of faecal pathogens.

8. Easy to detect and count.

9. Non-pathogenic.

No one bacterial species or group completely fulfils all these requirements, but a few come close to doing so. Three main groups of bacteria are used as faecal indicators in conventional water bacteriology: the faecal coliforms, the faecal streptococci and the anaerobic bacterium Clostridium perfringens. Recently, some other members of the anaerobic intestinal flora notably Bifidobacterium spp. Pseudomonas aeruginosa has also been proposed, but its status as an intestinal organism is in doubt. (Feachem et al 1983).

3.4.1 COLIFORM BACTERIA

The coliform group is defined as including "all aerobic and facultative anaerobic Gram-negative non-spore-forming bacilli which ferment lactose with gas formation". (Feachem, et al 1983). There are two principal groups of coliform bacteria; the faecal coliforms (comprising mainly the bacterium Escherichia coli) and the total coliform group that includes the faecal coliforms and comprises mainly species of the genera Citrobacter sp, Entrobacter sp, Escherichia sp and Klebsiella sp. The former are exclusively faecal in origin, whereas the latter, although commonly found in faeces also occur naturally in unpolluted soils and water. Of the total coliform organisms found in fresh faeces of warm-blooded animals, more than >90% are E. coli, the remainder being species of Citrobacter, Enterobacter and Klebsiella spp.

Only the faecal coliforms (and especially E. coli) are definitive indicators of faecal pollution. In water bacteriology the total coliforms are regarded as "presumptive" indicators of pollution and should be absent from disinfected water supplies. In wastewater bacteriology, however, the total coliforms are considerably less important because many, are nonfaecal in origin. In hot climates the total coliforms can multiply in the environment under suitable conditions so that their presence

in high numbers may not necessarily relate to either the occurrence or degree of faecal pollution. In general, only faecal coliforms (or better still, E. coli) should be used as indicators of faecal bacterial pathogens in wastes, wastewater and in treatment and reuse processes.

3.4.2 CLOSTRIDIUM PERFRINGENS

The bacterium Clostridium perfringen is anaerobic, spore-forming, Gram-positive and measures approximately 4-6 micrometers in length by 1-2 micrometers in width. It is exclusively faecal in origin and is also pathogenic, causing gas gangrene and food poisoning (Feachem et al 1983). Because it is a spore-forming organism it can persist for long periods outside the intestine, and therefore can be used as an indicator of occasional or intermittent pollution or of previous pollution of waters in which the presence of neither faecal coliforms nor faecal streptococci can be demonstrated (Feachem et al 1983). Clostridium Perfringen is also more resistant than both faecal coliforms and faecal streptococci to antagonistic substances such as chlorine.

In wastewater bacteriology however, its long persistence is a disadvantage because residual, dormant populations of the bacterium in waters may not reflect the true degree of pathogenic contamination.

3.4.3 FAECAL STREPTOCOCCI

The faecal streptococci are a group of bacteria that are morphologically similar (Gram-positive cocci, measuring approximately 1 micrometer in diameter and occurring in short chains) and are mostly found in the intestines of man and other warm-blooded animals. The group includes species mainly associated with animals as well as two non-faecal strains.

Aside from the possible problem of non-faecal strains, faecal streptococci have major advantages as faecal indicators. They are enumerated by a single-step membrane filter procedure at 37^{o}C, a temperature readily attained in small field laboratories. They are less prone to regrowth and generally survive somewhat longer than faecal coliforms and may thus be better indicators of excreted bacterial pathogens (that have little regrowth tendency) and excreted virus (that survive for longer than faecal coliforms in cool waters).

3.4.4 PSEUDOMONAS AERUGINOSA

The organism is an opportunistic human pathogen that causes infection in wounds (especially burns) and also ear and urinary tract infections, meningitis, respiratory infections and other conditions (Feachem et al 1983). Pseudomonas aeruginosa is being increasingly implicated as a cause of ear infection and skin rash following exposure in

inadequately disinfected swimming pools and whirlpool baths.

Pseudomonas aeruginosa is a gram-negative, aerobic, non-sporulating rod measuring approximately 0.5 and 2 micrometers. It occurs, normally at low concentrations of about 50 organisms per gram, in the feces of a small proportion (about 3 to 15 percent) of healthy people. It also occurs widely in nature as a free-living organism (Feachem et al 1983): it can therefore have little usefulness in studies of faecal contamination.

3.4.5 BIFIDOBACTERIA AND OTHER ANAEROBIC BACTERIA

Bifidobacteria are non-sporulating, anaerobic organisms that occur in the intestines of man and other animals; they are Gram-positive V or Y shaped cells, with each branch measuring about 0.8 by 3 to 4 micrometers. The most common species in man are Bifidobacteria adolescentis and Bifidobacterium longum. Bifidobacteria have recently been proposed as indicator organism for use in tropical waters because the lactose-fermenting species are exclusively faecal in origin. They therefore overcome the principal disadvantage of faecal coliform counts in tropical samples - contain a significant proportion of strains that can ferment lactose and produce indole at 44^{o}C but do not derive from feces. An additional advantage of Bifidobacteria is that, because they are strict anaerobes and grow poorly below 30^{o}C, they have very low multiplication potential in

extraintestinal environments. Work on Bifidobacteria has only commenced relatively recently, and there is little information on their survival in extra-intestinal environment other than in the river water (Feachem et al 1983).

Feces contain large numbers of other nonsporutating anaerobes, such as Bacterioides sp. the anaerobic Gram-positive cocci (Peptococcus spp and Peptostreptococcus Spp.) and Eubacterium spp. Current research is investigating the usefulness of these organisms as faecal indicators, but at present there are insufficient data on their extraintestinal ecology to know whether or not use of all or some of them as indicators will be practicable. Moreover, current techniques for their detection and enumeration are rather complex for routine use.

3.5 FACTORS AFFECTING BACTERIA REMOVAL IN WASTE STABILIZATION PONDS

Several hypotheses have been put forward as to the cause of great reductions of enteric and pathogenic organisms in wastewater. The factors that are considered to be responsible or partially so for bacterial removal in waste stabilization ponds are reviewed briefly below but it should be emphasized that their relative importance apart from perhaps that of time and temperature is largely

unknown.

1. EXTRACELLULAR ALGAL TOXINS: The production of toxic unicellular products by algae attributes to the high rate of bacterial die-off in waste stabilization ponds. Chlorella sp

has been found to liberate extracellular fatty acids such as chlorellin that seem to have a marked antibacterial activity according to Pratt et al 1944 and Speehr et al (1949) (cited by Amin and Ganapati 1972).

2. pH: Work by Parhad and Rao 1974 indicated that the high pH levels found in waste stabilization ponds were responsible for the reduction of bacteria. They studied the growth pattern of E. coli and algae in wastewater ponds and found that the increase in pH accompanies reduction of E. coli. Coliform concentrations seemed to be reduced during the periods of high pH levels, but the bacteria are never eliminated completely. The pH of the wastewater decreases during the evening and night, and there is a continual influx of coliforms into stabilization pond thus, minimizing the effect of increased pH.

Oswald 1960 found that pH changes in algal-bacterial cultures was directly proportional to algae concentration. Mohanrao 1973 found that increase in pH due to photosynthetic activity of algal was the main

cause of reduction of indicator organisms such as coliforms, E. coli and S. faecalis in waste stabilization ponds in India.

3. NUTRIENTS: Some quantities of organic carbon must be available in the ponds for bacteria to survive. Coliform reduction was found to be associated closely with BOD removal, indicating that coliforms are removed because of their inability to compete successfully for nutrients and due to microbial antagonism (Polprasart et al 1983).

4. TEMPERATURE: Bacteria are able to survive wide limits of temperature but the range in which they can grow and carry on their activities falls between 0 and 90^{o}C (Frobisher 1968). Marais (1974) presented a consolidation theory for a kinetic model for reduction of faecal bacteria in stabilization ponds incorporating the effect of temperature on the death rate. The die-off rate constant K is very sensitive to temperature and is approximately $K = 2.6\ (1.19^{T-20o})$, T in degree centigrade. It is presumed in this relationship that the ponds are mixed and aerobic or facultative and valid between 5 to 21^{o}C, with low wind velocities, periods of stratification occur causing the lower liquid depth of the pond to be anaerobic. There is decline in K value under anaerobic conditions resulting into high rate of faecal organisms survival.

5. SUNLIGHT: Ultraviolet rays are the invisible components of the sun's radiation. They are able to kill cells, temporarily delay cell division and also the synthesis of certain substances by cells change the manner in which substances pass across cellular membranes, cause abnormalities in chromosomes, and produce mutations.

Sarikaya and Saatci (1987) analysed the effect of solar radiation on the die-off of coliforms and they found out that a linear relationship exists between the die-off rate constant and the light intensity. Direct inactivation of pathogenic organisms with sunlight is limited to the pond surface. The removal is mainly attributed to photosynthesis by algae which occurs in presence of sunlight.

6. RETENTION TIME AND POND ARRANGEMENT: Ponds are characterized by long mean hydraulic retention times ranging from a few weeks in hot climates to several months in cold climates. Thus, ponds provide a considerably greater opportunity for faecal bacterial removal than other treatment processes. It is now well established (Mara 1976) both theoretically and from field observation, that removal of faecal bacteria is greater in a series of ponds than in a single pond providing the same overall hydraulic retention time, and that this efficiency increases with the number of ponds in the series. Mara and Silva (1979) report

the reduction of faecal coliform bacteria in a series of five ponds in N.E. Brazil, with a total retention time of 29 days an average temperature of 26°C, from 5×10^7 per 100 ml in raw sewage to 17 per 100 ml in the final effluent; this represents a very high overall reduction of 99.99996 percent.

3.6 KINETICS OF FAECAL REMOVAL IN STABILIZATION PONDS

Removal kinetics have been studied in details by only a few investigators. The most favoured approach (Marais 1974) is based on the following assumptions.

(1) Mixing in the ponds is instantaneous and complete. Thus the concentrations in the pond and in the effluent are identical. Ponds normally go through a daily cycle of gentle mixing and stratification due to wind and temperature effects. As the retention time in a pond is rarely less than about 7 days, each slug of influent is subject to an average of seven mixing periods during its residence in the pond. Relative to the retention time ponds can normally be assumed to be adequately mixed insofar as the kinetic relationship are concerned.

(2) Reduction of bacteria takes place according to Chicks law, i.e.

$$\frac{dN}{dt} = -KN \qquad \text{---------- 3.6.1}$$

in which

t = time, in days

K = decay constant dependent on the temperature, in (day^{-1}) units.

The value of K is strongly temperature dependent and is usually described by Arrhenius equation of the form

$$K_T = K_{20}^{\ T-20} \qquad 3.6.2$$

where

K_T is the value of K at T^oC

K_{20} its value at 20^oC, and

the dimensionless Arrhenius constant.

Let Q_i and Q_e = Influent and effluent flow per day, respectively, in some unit volume/day; Q_i is not necessarily equal to Q_e due to evaporation losses; (Seepage losses are considered as part of the effluent flow); V = volume of pond in the same volume units as the flow; and N_i and N_e = concentration of faecal bacteria in influent and pond (or effluent) per unit respectively.

A mass balance over time dt gives; change in mass of bacteria in pond = VdN. This is a consequence of: Increase due to inflow N_iQ_idt, decrease due to outflow

= -NQdt; and decrease due to die-off = -(NV)Kdt, i.e.

$$VdN = N_i Q_i dt - KVNdt - NQ_e dt$$

$$\text{i.e. } \frac{dN}{dt} + \left(K + \frac{Q_e}{V} \right) N_e = \frac{Q_i}{V} N_i \qquad 3.6.3$$

The parameters Q_i, Q_e and N_i are general functions of time. Furthermore, K is temperature dependent and as the seasonal temperature changes, in this fashion is also a function of time. Term V may be constant but if the pond depth is lowered during the dry period, V also becomes a function of time. These characteristics make equation 3.6.3 virtually intractable to analytical methods of solution. However, under a general specified time behaviour of these parameters, solutions can be obtained by numerical methods.

Normally, Q_i, Q_e and N_i have a daily cyclic variation. (Over a short period of time when the temperature can be taken as constant, K can be taken as approximately constant, K will vary appreciably due to seasonal temperature changes and other influences). It is to be expected that N will show concomitant daily cyclic variation. However mean values may, under certain restrictions (which normally approximate to the pond), adequately describe the mean kinetic behaviour of the pond.

At steady state in a single pond

$\frac{dN}{} = 0$. Solving for N_e in equation 3.6.3

dt

$$N_e = \frac{N_i \frac{Q_i}{V}}{K + \frac{Q_e}{V}} \qquad 3.6.4$$

let $t_i = \frac{V}{Q_i}$ = influent retention time

$t_e = \frac{V}{Q_e}$ = effluent retention time

Equation 4 reduces to

$$N_e = \frac{N_i}{Kt_i + \frac{t_i}{t_e}} \qquad 3.6.5$$

With short influent retention times say 10 days or less, losses due to evaporation will be small compared to the inflow $t_e = t_i$

$$N_e = \frac{N_i}{(Kt_i + 1)} \qquad \text{-------3.6.6}$$

For convenience the subscript in t_i is dropped i.e.

$$N_e = \frac{N_i}{(Kt + 1)} \qquad \text{------3.6.7}$$

where t is the retention time based on the influent flow.

Let t_1, t_2, t_3 ----- t_n be influent retention times in a series of n ponds. (Due regard must be taken to seepage and evaporation losses in estimating t_1, t_2, ----- t_n). If K remains constant, then from equation 7 the effluent quality in ponds 1 to N is given by:

$$N_1 = N_i(Kt_1 + 1)^{-1}$$

$$N_2 = N_1(Kt_2 + 1) = \frac{N_i}{(Kt_1 + 1)\ (Kt_2 + 1)} \quad \text{----3.6.8}$$

$$N_n = \frac{N_i}{\prod_{j=1}^{n} (Kt_1 + 1)} \quad \text{----------3.6.9}$$

If $t_1 = t_2 = t_3$ -----$t_n = t$, then

$$N_e = \frac{N_i}{(Kt+1)^n} \quad \text{----------3.6.10}$$

A more recent and more rigorous analysis of removal of faecal bacteria in ponds is given by Dissanayaka (1980) who studied the removal of faecal coliforms in laboratory-scale, pilot-scale, and full-scale ponds in Bangkok (Thailand). He found that the first-order rate constant for faecal coliform removal

(K_b in reciprocal days) was best described by the following multiple linear regression equation:

$$e^{Kb} = 0.7716(1.0281)^{T}(1.0016)^{C_s}(0.9994)^{\lambda} \qquad 2.6.11$$

where T is the temperature in degrees Celcius, C_s the average concentration of algae in the pond in milligrams per litre, and the organic loading on the pond in kilograms of chemical oxygen demand per hectare per day. The intensity of ultraviolet radiation was shown to be an unimportant factor in influencing the value of K_b, and no account was taken of predation by microinvertebrates (which as noted by Dissanayake is insignificant). When used with the Wehner and Wilhelm (1956) model for first-order-removal of faecal coliforms in dispersed flow reactors, this equation was found to be very satisfactory in predicting faecal coliform removal in full-scale ponds. Dissanayake (1980) also gives regression equations for predicting the value of C_s, so that his model for faecal coliform removal can be used by design engineers. Application of Dissanayake's model has of course been limited because of its recentness, and further work is required to determine the global applicability of its regression constants. Nonetheless, the model at least gives some idea of the relative importance of the principal environmental factors involved in removal of faecal bacteria in ponds.

At present it appears, therefore, that design engineers have no alternative but to follow the design

procedure based . on work of Marais (1974) and Mara (1976) for the removal of faecal bacteria in a series of ponds, even though its only environmental parameter is temperature, it is clear, however, that in the future pond design will have to include the effect of other variables such as algal biomass and organic loading. The pioneering approach shown by Dissanayake (1980) requires that it be followed by further work to determine its validity as a design tool.

CHAPTER FOUR

4.0 DANDORA SEWAGE TREATMENT WORKS

Dandora waste stabilization ponds are the major waste treatment works of Nairobi and is the largest pond installation in Kenya. The works is owned and operated by the Nairobi City Commission. The layout of the works is shown in figure 4.1.

Wastewater flows arriving at the works are excess flows arriving from Kariobangi sewage treatment works and the industrial area pond sewage from several connections into the trunk sewer between Kariobangi and the Dandora ponds.

The ponds are situated about 27 km east of Nairobi at latitude 1^{o} 15' S and longitude 37^{o} 00' E on a gently sloping land adjacent to the Nairobi river. The elevation of the site is just below the 1500 metre A.O.D contour. The annual average rainfall in Nairobi is some 850 mm. The driest months are December, January and February and the cold months of June and July. The wettest months are April and May.

Dandora pond system comprises two streams of waste stabilization ponds each stream containing two facultative and two maturation ponds. An inlet works provides facilities for screening, grit removal, flow measurement and emergency overflows. Each series is designed to treat 1500 m^3/d dry weather flow. The physical dimensions of

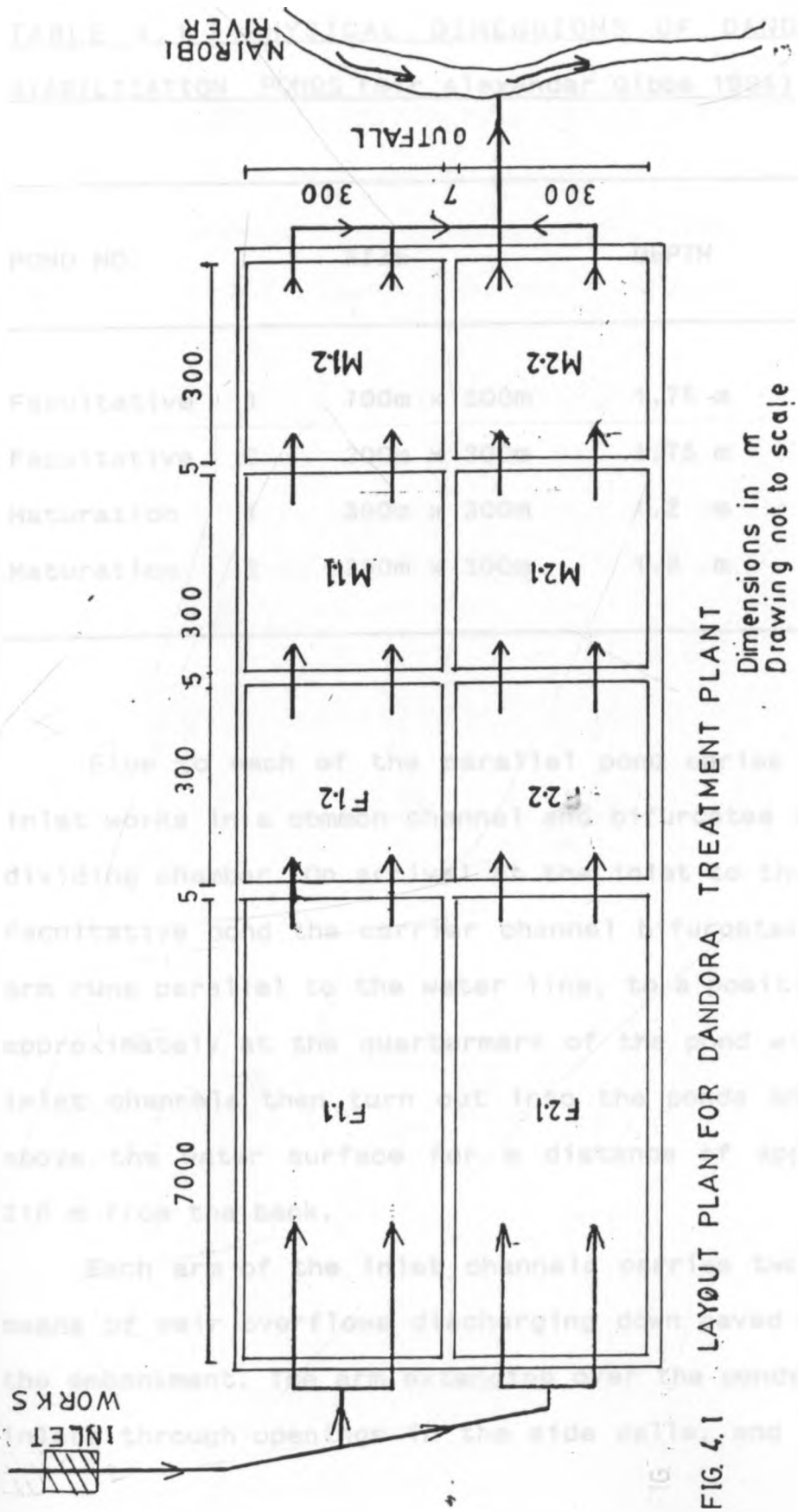


FIG. 4.1 LAYOUT PLAN FOR DANDORA TREATMENT PLANT
Dimensions in m
Drawing not to scale

the ponds are shown in table 4.1.

TABLE 4.1 PHYSICAL DIMENSIONS OF DANDORA WASTE STABILIZATION PONDS (Sir Alexander Gibbs 1984)

POND NO.		SIZE	DEPTH
Facultative	1	700m x 300m	1.75 m
Facultative	2	300m x 300m	1.75 m
Maturation	1	300m x 300m	1.2 m
Maturation	2	300m x 300m	1.2 m

Flow to each of the parallel pond series leaves the inlet works in a common channel and bifurcates at a dividing chamber. On arrival at the inlet to the first facultative pond the carrier channel bifurcates and each arm runs parallel to the water line, to a position approximately at the quartermark of the pond width. The inlet channels then turn out into the ponds and run above the water surface for a distance of approximately 210 m from the bank.

Each arm of the inlet channels carries two inlets by means of weir overflows discharging down paved sections of the embankment. The arm extending over the ponds has three inlets through openings in the side walls, and the end of

the channel is itself an inlet.

There are two interpond connections between successive ponds in each stream. Each interpond connection comprises a reinforced concrete intake with timber board scumplates and timber board weirs. A 610 mm internal diameter concrete pipe penetrates the embankment and discharges on the floor of the next pond over erosion protection apron.

Treatment effluent is discharged into the Nairobi river. The outfall has a flow measuring facility which was not functioning when this project was undertaken. The design criteria adopted for Dandora treatment works is shown in table 4.2.

TABLE 4.2 DESIGN CRITERIA FOR DANDORA TREATMENT WORKS

(Sir Alexander Gibbs 1984)

PARAMETER	UNIT	DESIGN FIGURE
Raw Sewage:		
Flow	m^3/day	30000
BOD_5 Loading	mg/L	400
Facultative Pond I:		
Retention	Days	25 each
Facultative Pond II Retention	Days	10 each
Maturation I and II		
Retention	Days	7 each
Temperature Constant	-	0.27
Final Effluent BOD_5	mg/L	20
Final Effluent COD	mg/L	30
Final SS	mg/L	30
Residual Coliforms in Effluent of	%	<0.01% influent

The maintenance of Dandora waste stabilization ponds is not satisfactory. Frequent grass cutting and scum removal is hampered by the presence of hippos and crocodiles in the ponds.

4.1 PREVIOUS FINDINGS AT DANDORA TREATMENT WORKS

Little or no data exists on influent and effluent faecal coliform counts in waste stabilization ponds in Kenya. The performance of waste stabilization ponds in Kenya has mostly been evaluated in terms of BOD, COD and SS. Mara D.D during his visit to Kenya in 1983 as reported by Sir Alexander Gibb (1984) carried out test on raw sewage and effluents at various points along Dandora waste stabilization pond series. The results are shown in table 4.1.

Table 4.3: RESULTS OF GRAB SAMPLES TAKEN AT DANDORA WASTE STABILIZATION PONDS (Alexander Gibb 1984)

Sample	BOD_5 (mg/l)	COD (mg/l)	SS (mg/l)	FC per 100ml	Chla (mg/l)
Raw Sewage	520	2173	200	7.3×10^7	---
F2.1	74	366	484	3.3×10^6	2762
F2.2	60	468	468	1.1×10^6	1512
M2.1	25	289	190	5.2×10^5	484
M2.2	22	232	120	2.8×10^5	498

Although little can be meaningfully gained from a single set of grab samples certain features are apparent. The reduction of BOD occurred primarily in the first pond as would be expected. The final effluent BOD of 22 mg/l was satisfactory and the overall BOD removal of 98% was excellent.

The removal of faecal coliforms was 99.62%. The number of faecal coliforms in the final effluent (2.8 x 10^5 per 100 ml) was outside that recommended for discharge to receiving water courses.

Mwangi, L.W (1987) reported an overall BOD removal of 83% for 1986 at Dandora waste stabilization ponds which indicated poor performance of the ponds. The average effluent BOD was 46 mg/l which was outside the Nairobi city commission standard for discharge to water courses (appendix F2).

CHAPTER FIVE

5.0 EXPERIMENTAL INVESTIGATION AND ANALYTICAL METHODS

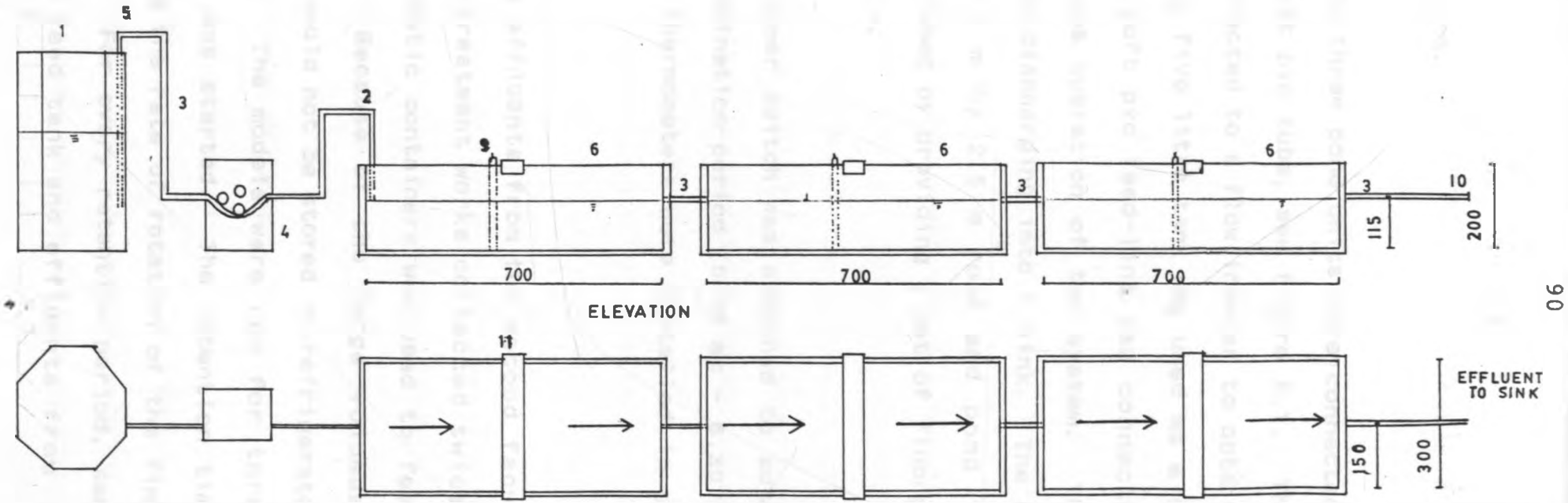
5.1 EXPERIMENTAL INVESTIGATION

Two experimental phases were undertaken that included studies of a laboratory scale waste stabilisation ponds and full-scale waste stabilisation ponds at Dandora sewage treatment works.

5.1.1 LABORATORY-SCALE WASTE STABILISATION PONDS

Three rectangular laboratory waste stabilisation pond units made of 6 -mm thick clear perspex glass were used in this phase. The units were made by sealing the butt joints with chloroform. The dimensions of the ponds were length 700 mm, width 300 mm and depth 200 mm. The level of the wastewater in each pond unit was 115 mm.

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



KEY

1 Sewage storage tank
2 Inlet
3 6mm PVC tubing
4 Flow inducer
5 Suction tube
6 Waste stabilization ponds
7 Light arrangement
8 Timer switch
9 Thermometer
10 Outlet
11 Stiffener

Dimensions in mm
Drawing not to scale

FIG. 5.1
VERTICAL AND HORIZONTAL PROFILES OF LABORATORY SCALE WASTE STABILIZATION PONDS

to failure.

The three pond units were connected in series with a 6 mm soft pvc tube, see figure 5.1. The inlet to pond I was connected to a flow inducer to obtain a constant flow. A twenty five litre tank was used as a feed tank to which a 6 mm soft pvc feed-line was connected to facilitate continuous operation of the system. The last pond unit (P3) was discharging into a sink. The set up was housed in a 4.5 m by 2.5 m room and pond illumination was accomplished by providing a set of fluorescent lamps 1.2 m long 40 w.

A timer switch was attached to automatically adjust the illumination period (6.30 am - 6.30 pm). Maximum and minimum thermometers were installed in each pond and the room.

The effluents from the second facultative pond at Dandora treatment works collected twice a week with 24 litre plastic containers was used to feed the laboratory models. Because of the large volumes involved, the samples could not be stored in refrigerator while awaiting feeding. The models were run for three weeks before sampling was started. The retention time was varied by adjusting the rate of rotation of the fingers of the flow inducer. For every retention period, samples were taken from the feed tank and effluents from the three pond

units, results of analysis of the samples are given in the next chapter and appendix B. Flow was measured with a stop watch and a measuring cylinder.

5.1.2 FULL-SCALE WASTE STABILIZATION PONDS

The wastewater treatment facility at Dandora has been described in Chapter 3 . One stream of the pond system was used in this study. The sampling stations are numbered 1 to 5 in Fig. 5.2. The sampling programme was started on 1/1/89 and pursued for 5 months at approximately 3 days interval. The sampling period covered both the warm, sunny and dry season (January to mid-March) and the main rainy season (mid-March to May).

The samples were collected between 9.30 and 11.30 a.m. of each sampling day and analysed within one and a half hours after collection. Cooperation was provided by the Nairobi City Commission employees in the measurement of flows and temperature at the treatment works.

5.2 ANALYTICAL METHODS

Laboratory examination and sampling were carried out in accordance with APHA (1985) - Standard methods and the oxoid manual of culture media, ingredients and other

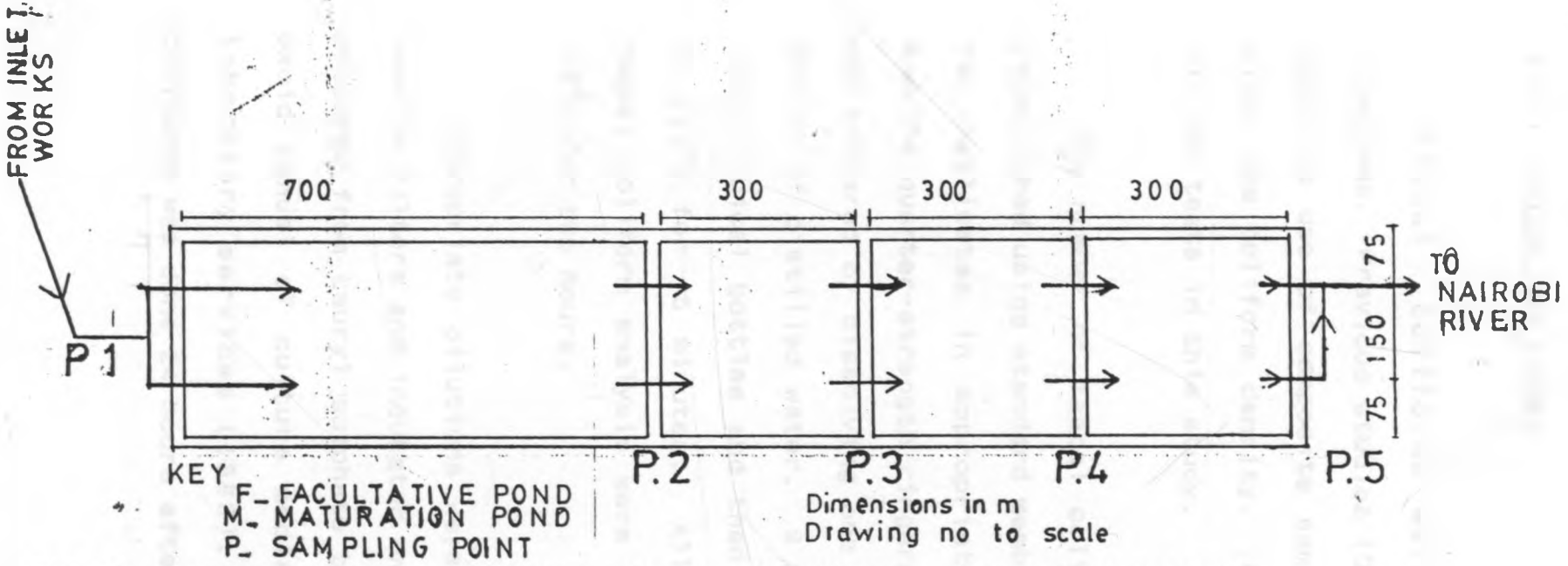


FIG.5.2. STREAM USED FOR FIELD STUDY AT DANDORA TREATMENT WORKS

laboratory services 1983. The procedures used are described below briefly.

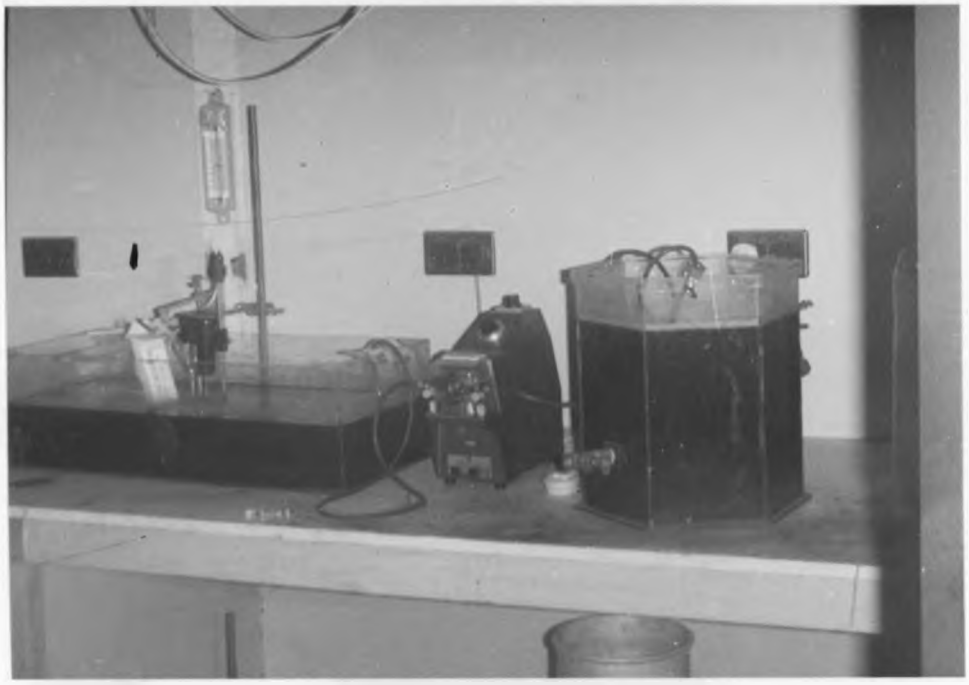
5.2.1 FAECAL COLIFORMS

Faecal coliforms were selected as the test organisms. Previous studies (Oragui et al 1987) indicated that the use of composite samples did not significantly alter the coliform density. Grab samples were used for all the tests in this study.

The number of faecal coliforms in the samples were established using standard membrane filter procedure using two replicates in appropriate dilutions prepared in sterile quarter-strength ringers solution. Dilution water was prepared by dissolving one ringers solution tablet in 500 ml of distilled water. 9 ml of the solution was put into clinical bottles and then sterilized by autoclaving at 121^{o}C for 15 minutes. All the glass ware used for faecal coliform analysis were sterilized in air oven at 170^{o}C for two hours.

Appropriate dilutions were filtered through 0.45 mm sterile filters and incubated on pads saturated with media prepared from Lauryl sulphate broth in accordance to the oxoid manual of culture media, ingredients an other laboratory services (1983). Enumeration of faecal coliforms was done 24 hours after incubation.

(i) Pond 3, 2



(ii) Pond 1, Flow inducer and Feed tank

FIG. 5.3.1 APPEARANCE OF LABORATORY SCALE WASTE STABILIZATION PONDS.

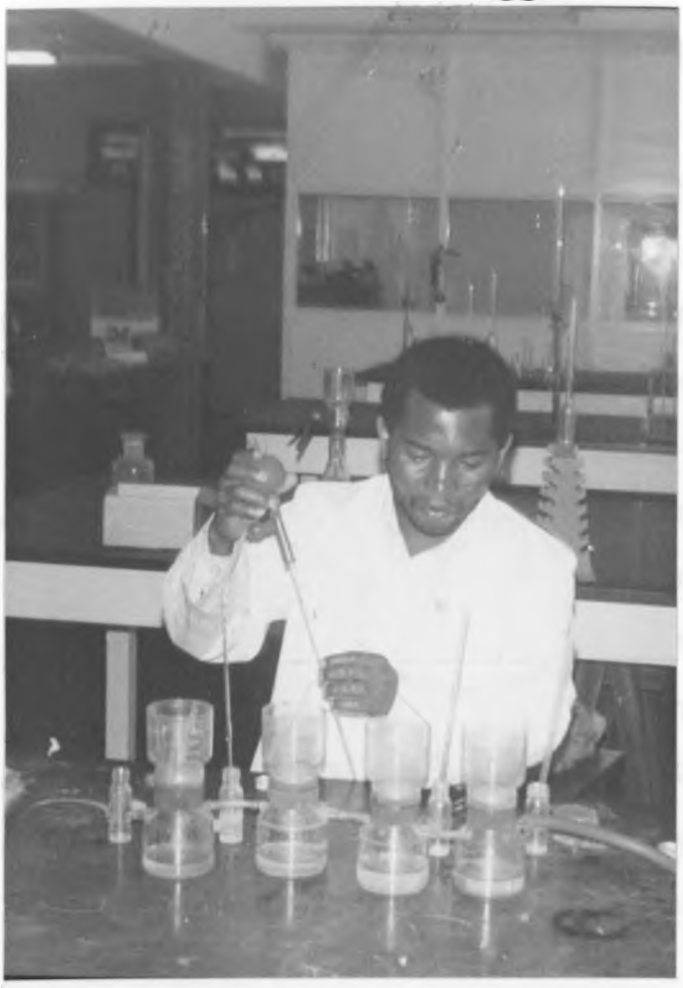


FIG. 5.3.2 MEMBRANE FILTRATION UNITS

FIG. 5.3.3 FILTER BEING ROLLED ONTO AN ABSORBENT PAD

5.2.2. BIOCHEMICAL OXYGEN DEMAND (BOD)

Dilution water was prepared by adding 1 ml of each of the following: Phosphate buffer, ferric chloride, magnesium sulphate and calcium chloride solutions to every 1 litre of distilled water. Dilutions of 1:10, 1:50 and 1:100 were prepared by pipetting the appropriate volume of sample into each of the three BOD bottles, filling each bottle with dilution water and ensuring that no air was trapped in the bottle. 3 blanks were prepared with just dilution water. The bottles were then incubated at $20^{o}C$ for 5 days. The dissolved oxygen (DO) in each bottle was then measured using Azide modified method.

The BOD of the sample was approximated as follows

$$BOD_5 = (DO_b - DO_d) \times \frac{\text{Volume of same}}{\text{Volume of BOD bottle}}$$

Where DO_b = DO concentration in the blank

DO_d = DO concentration in Bottle after 5 days, mg/L

5.2.3 CHEMICAL OXYGEN DEMAND (COD)

10ml of sample was placed in a 250ml refluxing flask and 0.25 of solid mercuric sulphate, 5ml of 0.35m potassium dichromate 15 ml of concentrated sulphuric acid

reagent were added. This was repeated for blank using 10 ml of distilled water instead of the sample. Few glass beads were added to each flask and then they were fitted to the condenser system, making sure that the ground glass joint was snug. The flow of cooling water was started and heaters switched. Refluxing was carried out for 90 minutes.

The flasks were around to cool and then rinsed each with 50ml of distilled water. The contents of each flask were diluted with 70 ml of distilled water. 2 drops of Ferroin indicator solution were added and titrated with 0.1n ferrous ammonium sulphate solution to an end point characterised by a change of colour from blue-green to reddish-brown.

The COD of the sample was given By

$$\text{COD mg/L} = \frac{(a-b)xnx8000}{\text{ml of sample}}$$

Where a = ml of titrant used for the blank

b = ml of titrant used for the sample

n = normality of ferrous ammonium sulphate

5.2.4. SUSPENDED SOLIDS

Filter papers were preweighed and stoned in a dessicator. A sample volume of between 20 ml and 50 ml was filtered through the preweighed filter paper and washed with a small quantity of distilled water. The filter papers were then carefully removed and placed in a crucible then dried at 105^{o}C for at least one hour. The filter papers were then weighed. The suspended solids were calculated as follows:

$$SS\ (mg/L) = \frac{W_2 - W_1}{\text{Volume of sample (ml)}}$$

Where W_2 = Weight of filter paper after drying (g)

W_1 = Weight of paper prior to filtration (g)

5.2.5 ALGAL SPECIATION

Microscopic examination were first carried out using a x 10 objective which enabled the detection of large algae cells such as Euglena for smaller cells such as hlorella and Scenedesmus a x40 objective was used.

5.2.6 CHLOROPHYLL A (chla)

A 4.7 cm GFC filter paper was placed in a filter holder. 3.5 ml of 0.1 m/L $mgco_3$ suspension was filtered through the filter and then the filter paper was placed in a boiling tube and 10 ml of 90% V/v methanol was added. The sample was boiled for approximately 2 minutes in a water bath.

Filter paper debris were removed from the methanol solution by settling them out after the tubes were left to stand. Samples were then made upto exactly 10ml volumes with 90% methanol using graduated centrifuge tubes. 3ml of extract were placed in a 3 cm culvette and absorbance read first at 750m and then at 665 nm using a blank of 90% methanol. 0.05 ml of 0.6Hcl, were added, mixed and left for 30 seconds, followed by 0.05ml of 0.6m dimethylaniline mixed and again left for 30 seconds. Without altering the wavelength settings, the absorbance at 665mm and at 750 mm were re-read.

chlorophyll A concentration was calculated using the following equation:

$$\text{Ca mg/L} = 35.32\ (A_b - A_a)\ \frac{v}{V}$$

Where

Ca = Chlorophyll A

Ab = Absorbance 665mm - Absorbance 750nm after acidification

Aa = Absorbance 665nm - Absorbance 750 mm after acidification.

v = Volume of methanol (ml)

V = Volume of sample filtered (litres)

CHAPTER SIX

6.0 ANALYSIS AND DISCUSSION OF RESULTS

6.1 ESTIMATION OF RETENTION TIME

The flow measurement device of the inlet work was working while that of the outlet was out of order during the study period. To estimate the interpond flows, flow records of the inlet and outlet taken when both devices were working are used (Appendix A).

The following assumption are made:

1 The flow to the pond system is distributed equally to the two streams.

2 The 8% difference in mean inflow rates of the two periods (January-April 1988 when the inflow and outflow were both recorded and January - April 1989) will not bring significant difference on the losses of the two periods.

3. The fraction of change in storage in the entire pond system (dQ) stored in the ith pond is proportional to the ratio of the capacity of the ith pond to the capacity of

the entire system.

The figure (6.1) below shows a schematic diagram of the Dandora pond system.

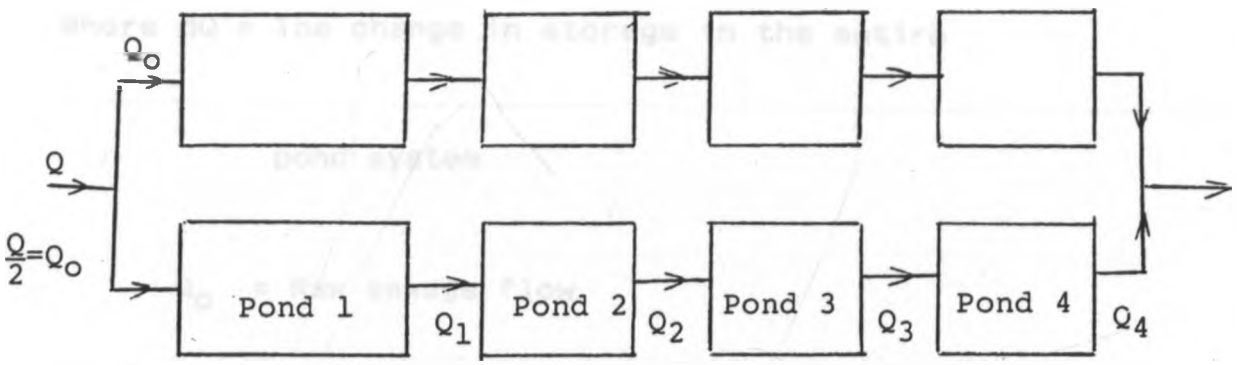


FIG. 6.1 SCHEMATIC DIAGRAM OF DANDORA POND SYSTEM

The flow balance mode of Dandora treatment pond system can be summarised as follows:

$$dQ = Q_o - Q_4 \quad \text{----------6.1.1}$$

Where dQ = The change in storage in the entire pond system

Q_o = Raw sewage flow

Q_1 = Effluent from pond 1

Q2 = Effluent from pond 2

Q_3 = Effluent from pond 3

Q_4 = Effluent from pond 4

INTERPOND FLOWS

$$Q_J = Q_o - dQ \frac{\sum_{j=i}^{j=4} V_j}{\sum_{j=i}^{4} V_i} \qquad j = 1, 4 \quad \text{----6.1.2}$$

INFLOW AND OUTFLOW TO THE POND SYSTEM

Average inflow January-April 1989 = $3.38.10^3 m^3/h$

Average inflow January-April 1988 = $3.10 \times 10^3 m^3/h$

Average inflow January-April 1988 = $1.75 \times 10^3 m^3/h$

Considering one stream of the pond system and assuming 50/50 split,

Average inflow (1989) = $1.69 \times 10^3 m^3/h$

Average loss (1988 = 1989) = $0.875 \times 10^3 m^3/h$

Let V_1 = Design volume of pond F2.1

V_2 = Design volume of pond F2.2

V_3 = Design volume of pond M2.1 and

V_4 = Design volume of pond M2.2

V= DxWxL, where D = depth of pond

W = width of pond

L = length of pond

V_1 = 367500 m^3

V_2 = 157500 m^3

V_3 = 108000 m^3

V_4 = 741000 m^3

Using equation 6.1.2

Q_1 = 1256 m^3/h Q_3 = 943 m^3/h

Q_2 = 1070 m^3/h Q_4 = 815 m^3/h

where V_i = design capacity of the ith pond

Theoretical retention period T

$$T = \frac{\text{Volume}}{\text{Flow}^*}$$

* - Mean of influent and effluent flow.

SUMMARY

Table 6.1 ESTIMATED RETENTION TIMES AT DANDORA TREATMENT WORKS

POND NO.	RETENTION (DAYS)
FACULTATIVE 2.2	10.4
FACULTATIVE 2.2	5.6
MATURATION 2.1	4.5
MATURATION 2.2	5.1
OVERALL	25.6

One of the main factors in pathogenic destruction is detention time. The others, temperature, solar radiation, etc. seems less relevant for pond design since ambient conditions will govern, leaving retention time as the main process variable. Table 6.1 shows the retention time at Dandora treatment works estimated using the flow balance model described above. The average total retention time is 25.6 days, while the total design retention time was 49 days. The decrease in retention shows that the plant has been operating above its design capacity.

To facilitate simulation of faecal coliform die-off, the interpond flows and volumes of wastewater stored in each pond are needed. Direct estimation of the interpond flow and storage volumes from the inflow data for the entire system was not possible because no control structures had been installed between the ponds, and the outflow measuring device was not working. Therefore the interpond flows were estimated through an interactive simulation procedure using flow-balance simulation models.

The data in appendix A shows the flow readings of the inlet of Dandora treatment works monitored during the study period. Figure 6.1.2 shows the daily inflow variations during the study period. Two distinct regions can be seen from the figure. January to early March

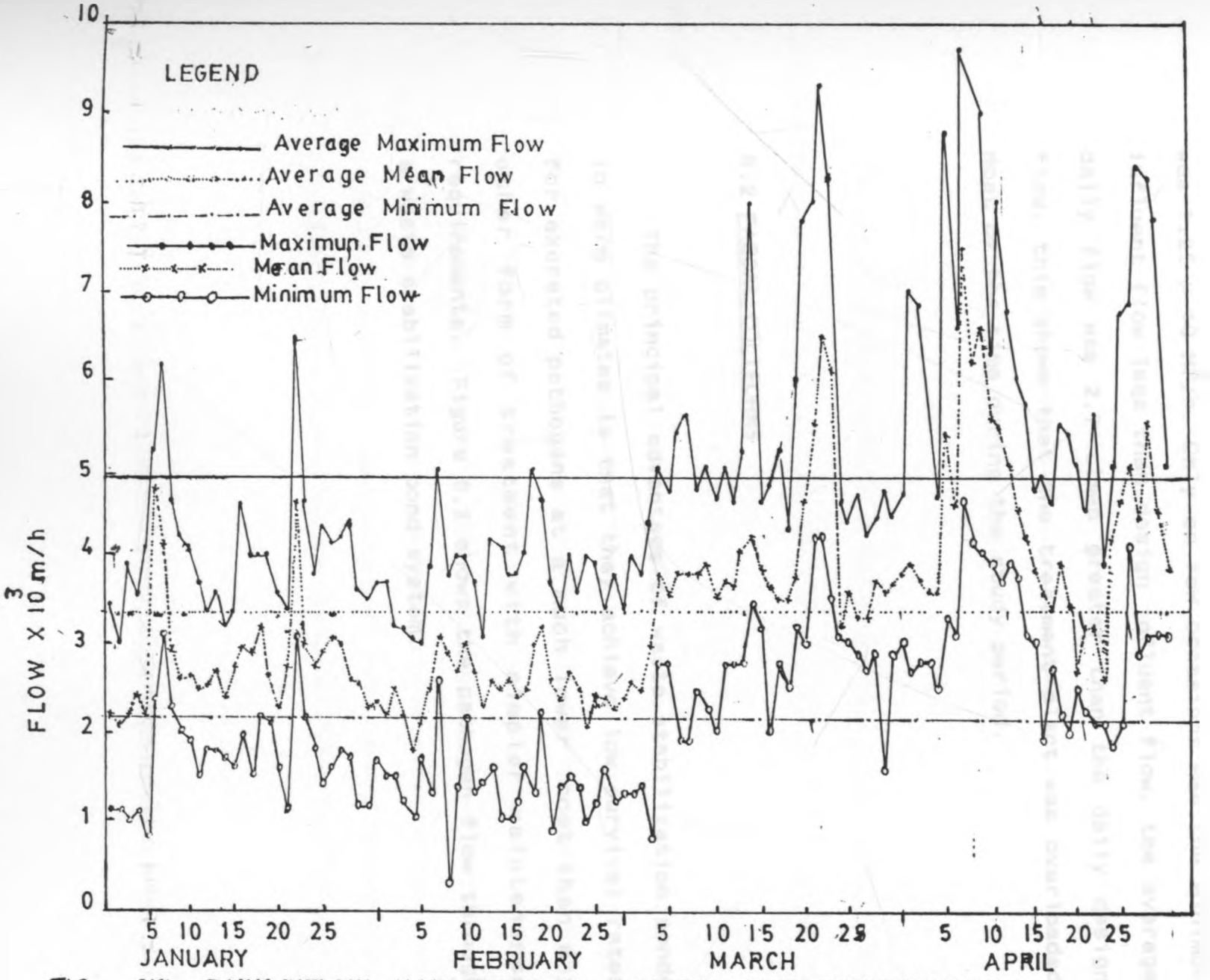


FIG. 6.12. DAILY INFLOW VARIATIONS AT DANDORA TREATMENT WORKS

represents the dry sunny and warm season while March and April represents the rainy season.

The average maximum flow, average mean flow and average minimum flow were 4.9 x $10^3 m^3/h$, 3.4 x $10^3 m^3/h$ and 2.2 x 10^3 m^3/h respectively. The design of the work was 1.25 x 10 M^3/h. Only on few occasions was the minimum influent flow less than design influent flow, the average daily flow was 2.7 times greater than the daily design flow, this shows that the treatment plant was overloaded most of the time during the study period.

6.2 FAECAL COLIFORMS

The principal advantage of waste stabilization ponds in warm climates is that they achieve low survival rates for excreted pathogens at a much lower cost than any other form of treatment with simpler maintenance requirements. Figure 6.2 shows the pathogen flow through a waste stabilization pond system.

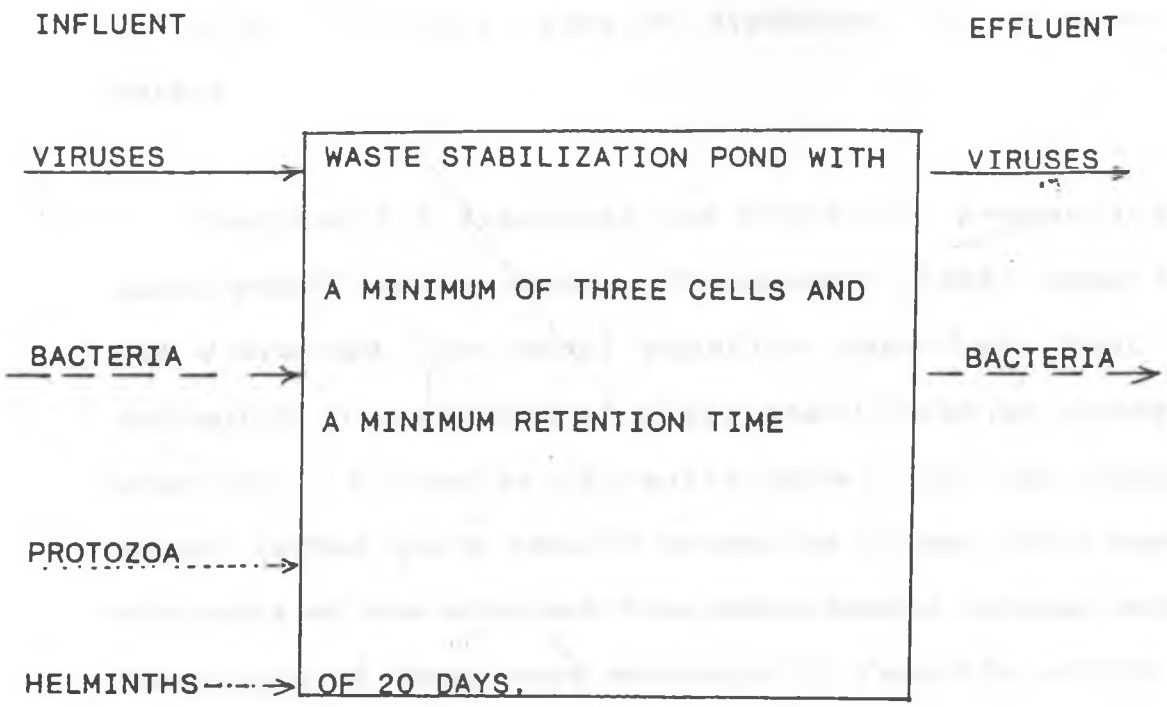


FIG 6.2 PATHOGEN FLOW THROUGH A WASTE STABILIZATION POND SYSTEM

(Feacmem et al 1983)

Feachem et al (1983) noted that a well-designed pond system - incorporating a minimum of three cells, and having a minimum total retention time of 20 days produce an effluent that will contain only small concentrations of excreted bacteria and viruses. Excreted helminth eggs and protozoa cysts will be completely eliminated. Bacterial or viral pollution can be further reduced (or eliminated) by adding more ponds to the system. The effluent is

suitable for direct reuse or discharge into a receiving waters.

Section 2.4 discusses the hydraulic properties of waste stabilisation ponds. Thirumurthi (1969) noted that the dispersed flow model equation describes best the hydraulic flow pattern of waste stabilization ponds in practice. A precise hydraulic model for the Dandora lagoon system would require extensive tracer experiments, wind data at the site and flow measurements between ponds. Since none of these were economically feasible within the limitations of this project it was decided to adopt a simple completely mixed-flow model in this study. The assumption of complete mixing seems reasonable since the ponds are shallow and present broad surface area for wind stirring.

6.2.1 LABORATORY SCALE WASTE STABILIZATION PONDS

Throughout the laboratory waste stabilization pond sampling, the daily pond temperatures were 22 +1^{o}C. The experimental results of faecal coliform count obtained from the series of three laboratory waste stabilization ponds are shown in table 6.2.1. The mean percentage removal of faecal coliforms in each pond unit generally decreases down the series except for p3 with a detention time of 2.7 days. It was not established why at that particular detention time the percentage removal deviated

TABLE 6.2.1 SUMMARY OF RESULTS OF FAECAL COLIFORM REMOVAL IN THE LABORATORY SCALE WASTE STABILIZATION PONDS

LOCATION	RETENTION TIME (D)	FAECAL COLIFORMS PER 100ml			PERCENTAGE REMOVAL	OVERALL PERCENTAGE REMOVAL	DIE-OFF RATE CONSTANT
		MAX	MEAN	MIN.			
P1 INFLUENT	2.7	1.18×10^{6}	3.58×10^{5}	7.25×10^{4}	93.52	93.52	5.34
EFFLUENT		3.25×10^{4}	2.32×10^{4}	9.5×10^{3}			
P2 INFLUENT	2.7	3.25×10^{4}	2.32×10^{4}	9.5×10^{3}	67.2	97.87	0.98
EFFLUENT		1.7×10^{4}	7.61×10^{3}	2.6×10^{3}			
P3 INFLUENT	2.7	1.7×10^{4}	7.61×10^{3}	2.6×10^{3}	68.33	99.33	0.8
EFFLUENT		5.55×10^{3}	2.41×10^{3}	7×10^{2}			
P1 INFLUENT	3.2	6.4×10^{4}	1.9×10^{4}	1×10^{3}	89.26	89.26	2.61
EFFLUENT		7.4×10^{3}	2.04×10^{3}	3×10^{2}			
P2 INFLUENT	3.2	7.4×10^{3}	2.04×10^{3}	3×10^{2}	64.46	96.18	0.57
EFFLUENT		1.6×10^{3}	7.25×10^{2}	1.5×10^{2}			
P3 INFLUENT	3.2	1.6×10^{3}	7.25×10^{2}	1.5×10^{2}	52.97	98.21	0.35
EFFLUENT		5.85×10^{2}	3.41×10^{2}	9×10^{1}			
P1 INFLUENT	4.5	9.5×10^{3}	5.57×10^{3}	1×10^{3}	89.65	89.65	1.94
EFFLUENT		1.25×10^{3}	5.76×10^{2}	1.2×10^{2}			
P2 INFLUENT	4.5	1.25×10^{3}	5.76×10^{2}	1.2×10^{2}	71.35	97.04	0.56
EFFLUENT		2.85×10^{2}	1.65×10^{2}	1×10^{2}			
P3 INFLUENT	4.5	2.85×10^{2}	1.65×10^{2}	1×10^{2}	53.76	98.63	0.26
EFFLUENT		1.07×10^{2}	7.63×10^{1}	1.2×10^{1}			

from the general trend.

The overall percentage removal of faecal coliforms varied from 98.63% to 99.33% (Tables 6.2.1 and 6.2.2). An interesting observation from the tables is that the highest overall removal also represented the highest survival of faecal coliforms. The results indicate that percentage removal is not good enough to compare the removal of faecal coliforms in treatment plants.

There seems to be a tendency for the die-off to be slower in the ponds with the lower initial coliform counts. It was difficult to control the initial concentration of the coliforms in order to see their effect on the die-off rate constant. The die-off rate constant decreases from P1 to P3 and also decreases with increase in detention time (fig. 6.2.1.1.).

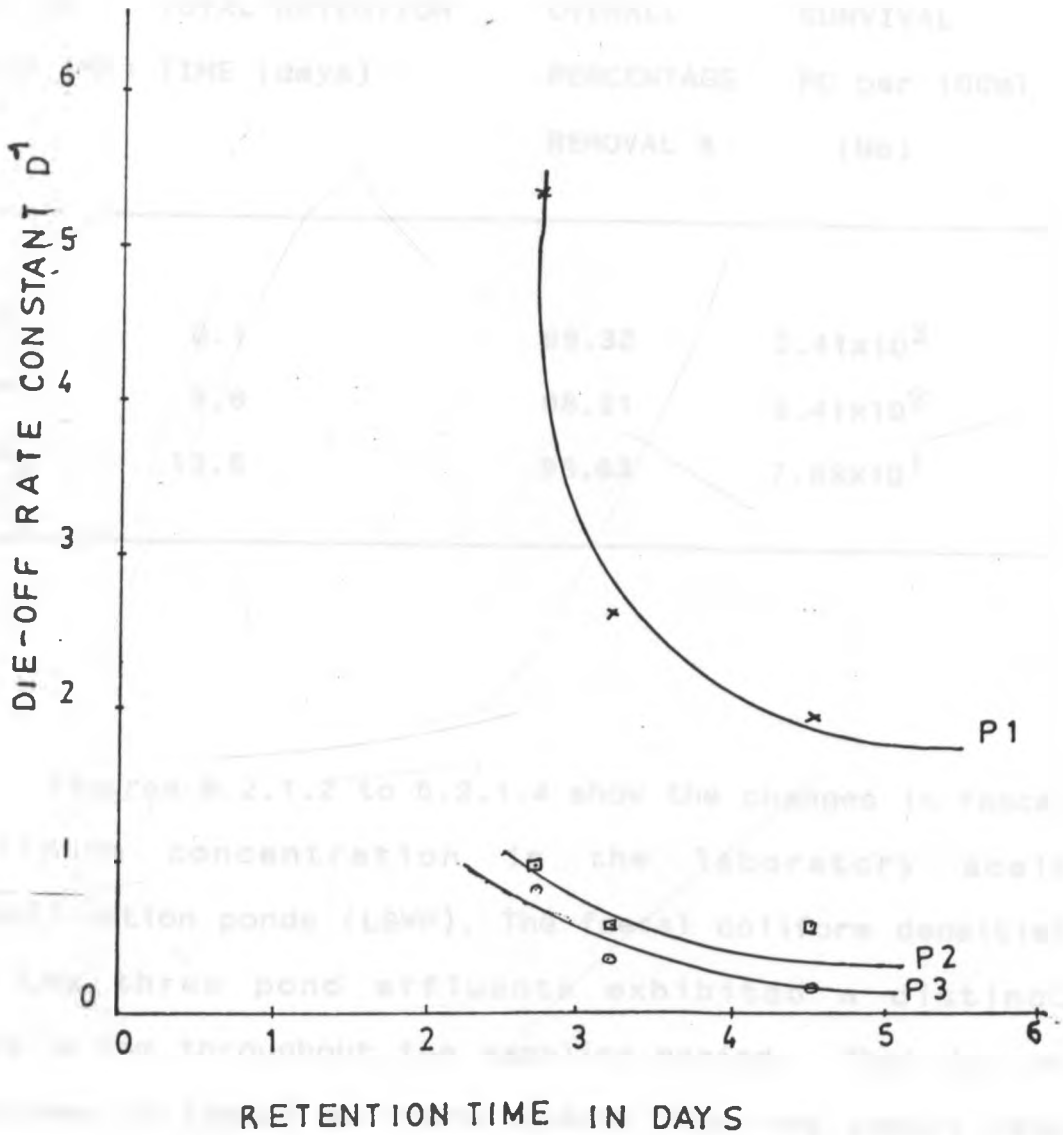


FIG 6.2.3 DIE-OFF RATE CONSTANT AGAINST RETENTION TIME

TABLE 6.2.1.2 AVERAGE INFLUENT FC COUNTS, RETENTION TIME, PERCENTAGE REMOVAL AND SURVIVAL OF FC IN LSWP.

INFLUENT FC per 100ml (Ni)	TOTAL RETENTION TIME (days)	OVERALL PERCENTAGE REMOVAL %	SURVIVAL FC per 100ml (Ne)
3.58×10^5	8.1	99.33	2.41×10^3
1.9×10^4	9.6	98.21	3.41×10^2
5.57×10^3	13.5	98.63	7.63×10^1

Figures 6.2.1.2 to 6.2.1.4 show the changes in faecal coliform concentration in the laboratory scale stabilization ponds (LSWP). The faecal coliform densities in the three pond effluents exhibited a distinct parallelism throughout the sampling period. That is, an increase in faecal coliform density from one sample date to the next in pond 1 was usually accompanied by a rise in ponds 2 and 3. Similarly a decrease in faecal coliform concentration in pond 1 was usually indicative of a decrease in the final lagoon. As expected, the pond 1

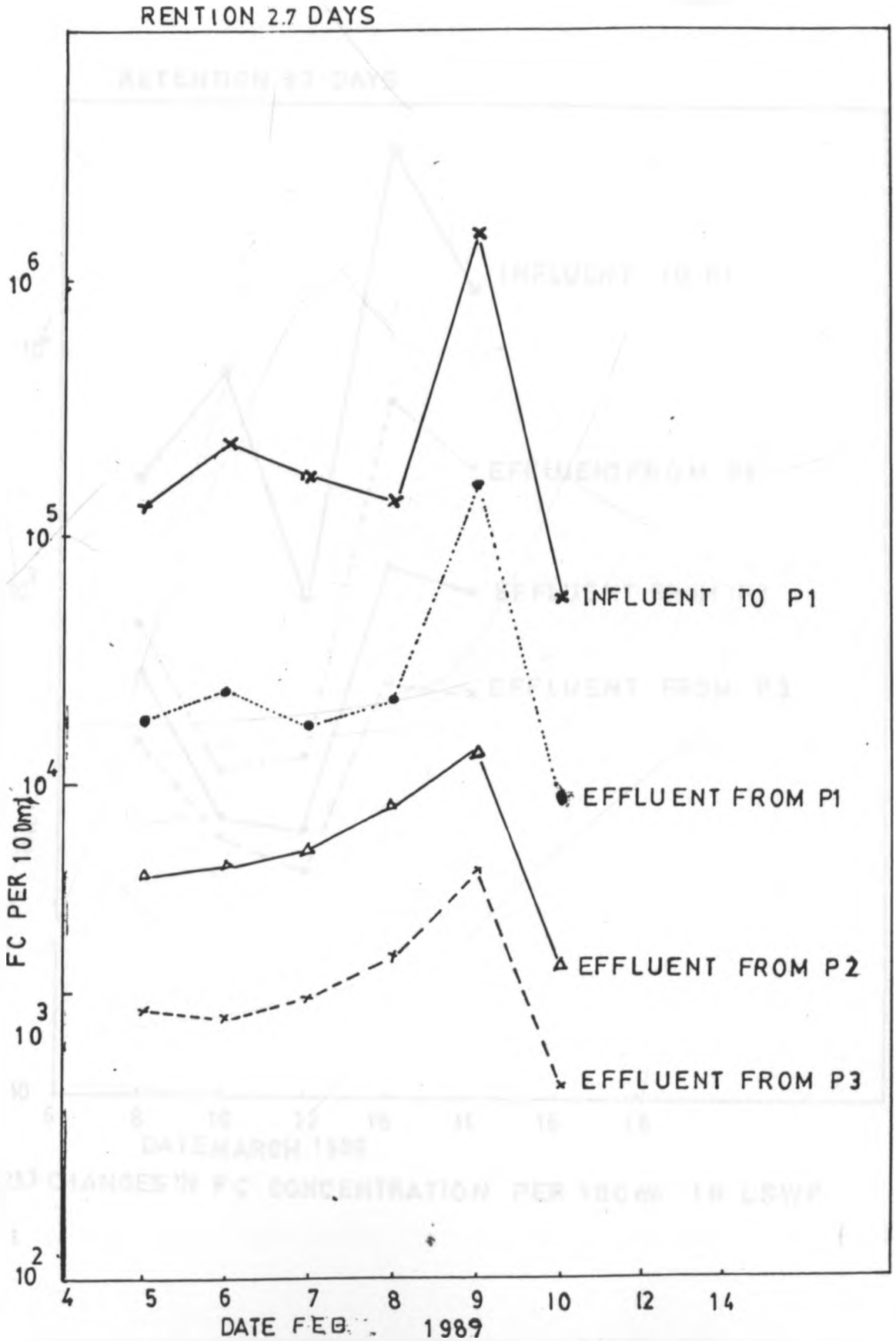


FIG 6.21 CHANGES IN FC CONCENTRATION PER 100 ml IN LSWP

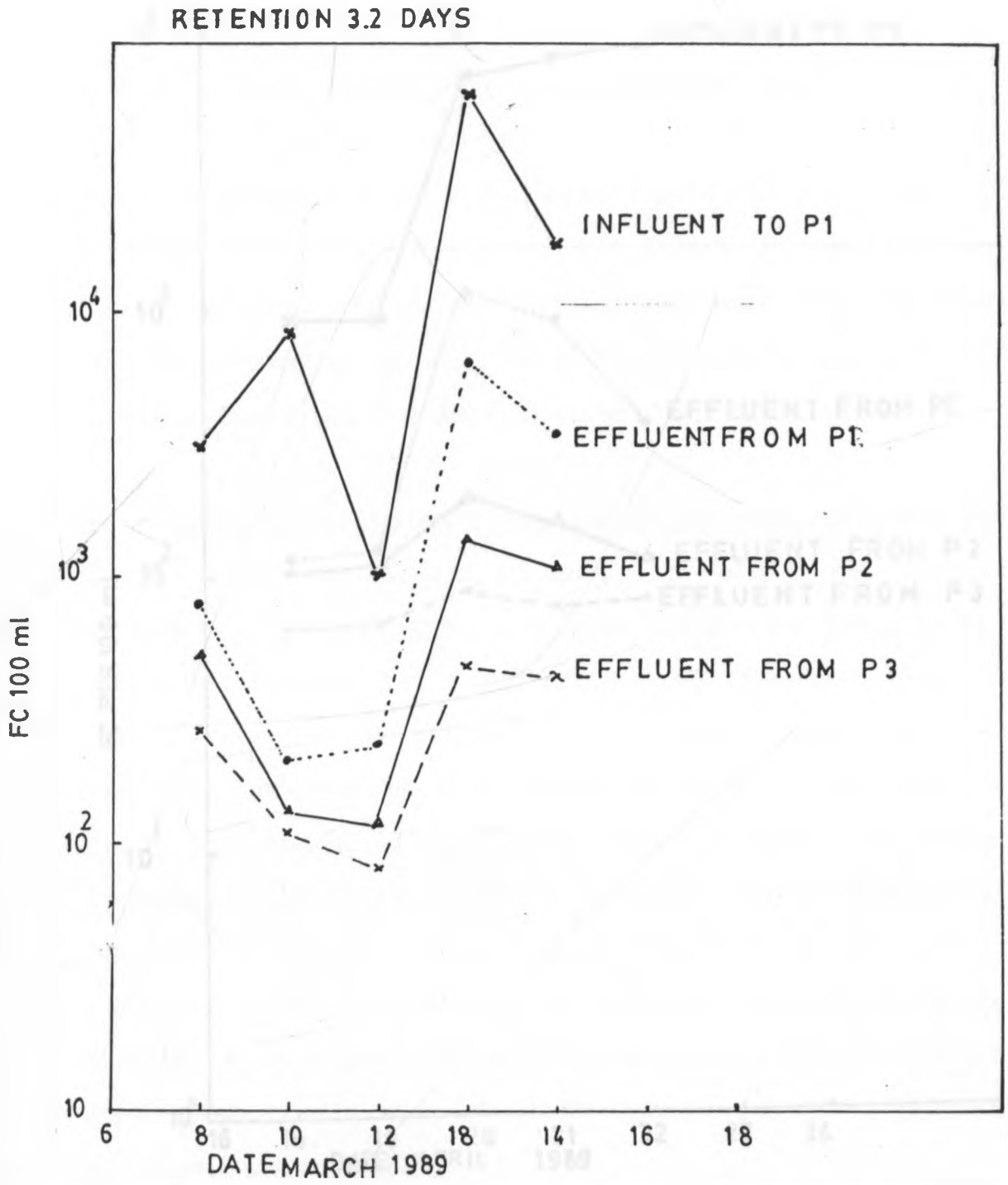


FIG6.213 CHANGES IN FC CONCENTRATION PER 100 ml IN LSWP

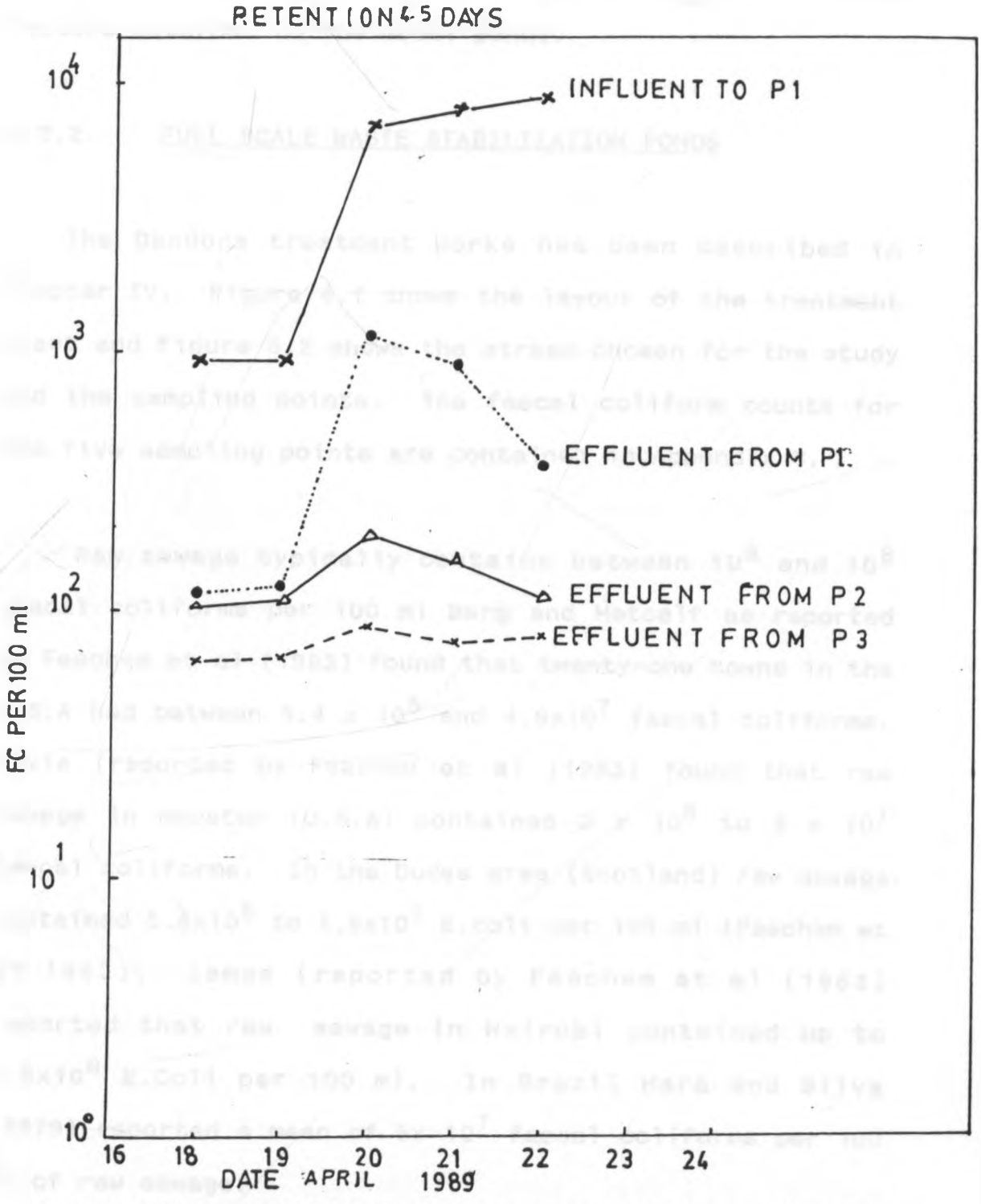


FIG 6.2.14 CHANGES IN FC CONCENTRATION PER 100 ml IN LSWP

influent faecal coliform concentration did not necessarily parallel the others, as its coliform count was intimately tied to that of the F2.2 effluent at Dandora treatment works and storage of the effluent which depended on factors external to the model ponds.

6.2.2. FULL SCALE WASTE STABILIZATION PONDS

The Dandora treatment works has been described in chapter IV. Figure 4.1 shows the layout of the treatment plant and figure 5.2 shows the stream chosen for the study and the sampling points. The faecal coliform counts for the five sampling points are contained in Appendix C.

Raw sewage typically contains between 10^5 and 10^8 faecal coliforms per 100 ml Berg and Metcalf as reported by Feachem et al (1983) found that twenty-one towns in the U.S.A had between 3.4×10^5 and 4.9×10^7 faecal coliforms. Davis (reported by Feachem et al (1983) found that raw sewage in Houston (U.S.A) contained 3×10^6 to 3×10^7 faecal coliforms. In the Dudee area (Scotland) raw sewage contained 5.8×10^6 to 1.5×10^7 E.coli per 100 ml (Faechem et al 1983). James (reported by Feachem et al (1983) reported that raw sewage in Nairobi contained up to 1.6×10^8 E.Coli per 100 ml. In Brazil Mara and Silva (1979) reported a mean of 5×10^7 faecal coliforms per 100 ml of raw sewage.

The faecal coliform counts for the influent at Dandora treatment works varied from $6.9x10^7$ to $3.5x10^6$ per 100 ml, with a mean of 2.9×10^7 per 100 ml Table 6.2.2. The raw sewage coliform counts falls within the range reported in literature above. The maximum faecal coliform counts recorded of $6.9x10^7$ per 100ml differs from the value reported by James $1.6x10^8$ per 100ml. The difference could have occurred due to one or both of the following reasons. (1) The increase in water supply, thus providing a higher dilution of the raw sewage (2) Industrial undertakings have developed which discharge their wastewater into sewers leading to Dandora treatment works. The concentration of indicator bacteria in sewage may be affected by Industrial wastes that often contain chemicals toxic to enteric bacteria. Data on raw sewage from different parts of U.K showed E. coli concentration of $1.7\text{-}3.7x10^8$ per 100 ml where sewage was principally of domestic origin, compared with only $9x10^5$ per 100 ml where sewage flow was 60% of industrial origin (Feachem et al 1983).

Figure 6.2.3.1 shows the changes in faecal coliform concentration per 100 ml in the five sampling points. Similar to the laboratory waste stabilization ponds the faecal coliform densities at sampling points 2 to 5 exhibited a distinct parallelism throughout the sampling period. The average removal of faecal coliforms in each pond unit and the overall removal at Dandora treatment

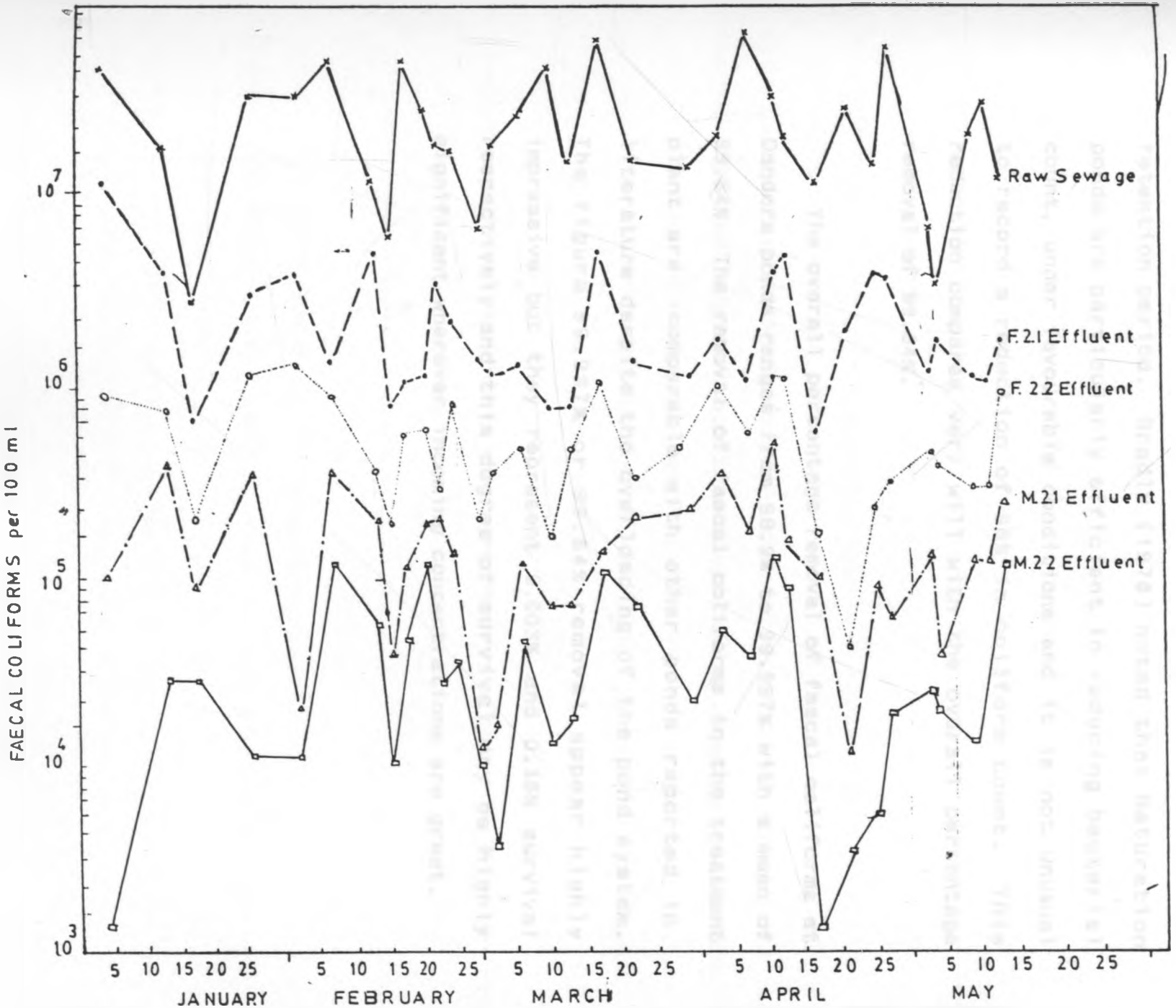


FIG. 6.231 CHANGES IN FAECAL COLIFORM CONCENTRATION IN DANDORA POND SYSTEM

works is shown in Table 6.2.2 and appendix C. Pond F2.1 showed the highest percentage removal of 91.02 and F2.2 a percentage removal of 75.67% M2.1 and M2.2 achieved 70.2% and 74.2% removal respectively. F2.1 achieved a high percentage removal probably because of the longer retention period. Bradly (1976) noted that Maturation ponds are particularly efficient in reducing bacterial count, under favourable conditions and it is not unusual to record a reduction of 98% in coliform count. This reduction compares very will with the overall percentage removal of 99.84%.

The overall percentage removal of faecal coliforms at Dandora ponds ranges from 98.9% to 99.997% with a mean of 99.84% The removal of faecal coliforms in the treatment plant are comparable with other ponds reported in literature despite the overloading of the pond system. The figure 99.997% or 99.84% removal appear highly impressive but they represent 0.003% and 0.16% survival respectively and this degree of survival may be highly significant wherever incoming concentrations are great.

TABLE 6.2.2 FAECAL COLIFORM REMOVAL IN THE DANDORA WASTE STABILIZATION PONDS.

LOCATION		FC per 100 ml INFLUENT	FC per 100 ml EFFLUENT	DIE-OFF RATE CONSTANT	PERCENTAGE REMOVAL	OVERALL% REMOVAL
	MAX	$6.9X10^7$	$1.25X10^7$			
F2.1	MEAN	$2.91X10^7$	$2.6X10^6$	0.98	91.02	91.016
	MIN	$3.5X10^6$	$5.76X10^5$			
	MAX	$1.25X10^7$	$3.93X10^6$			
F2.2	MEAN	$2.61X10^6$	$6.35X10^5$	0.56	75.67	98.64
	MIN	$5.76X10^5$	$2X10^5$			
	MAX	$3.93X10^6$	$5.8X10^5$			
m2.1	MEAN	$6.35X10^5$	$1.86X10^5$	0.54	70.7	99.361
	MIN	$2.00X10^5$	$1.25X10^4$			
	MAX	$5.8X10^5$	$1.5X10^5$			
M2.2	MEAN	$1.86X10^5$	$4.80X10^4$	0.56	74.2	99.84
	MIN	$1.25X10^5$	$1.69X10^3$			

Consider the average influent coliform count of $6.9x10^7$ FC per 100 ml, then the 99.835 FC removal will produce an effluent with 113850 FC per ml. In areas where the effluent is to be reused or where it is to be discharged to a downstream used as a source of drinking water, such effluent quality may be inadequate. The world health organization Standards for effluent to be used for unrestricted irrigation stipulates that the faecal coliform count should be less that 100 per 100 ml.

In Dandora treatment works, the effluent faecal coliform count deviates from the expected count fig.6.2.3.2. The big difference in the expected faecal coliform counts from the measured ones has resulted probably because of reduction of the retention times due to hydraulic overloading and the assumption that the contents of the ponds were completely mixed. At temperatures higher than 15^oC the density decrease of water per degree of temperature increase is comparatively high, most sewage lagoons at least during the day have a resistance to mixing. The thermal stability in inflowing sewage will not completely mix with the water of a lagoon but rather move into the water layer with the same density (temperature). A short-circuit-flow which by-passes a large portion of the faecal coliform removal will be much smaller than that calculated from pond volume and average discharge. Thus effluent quality is expected to deviated

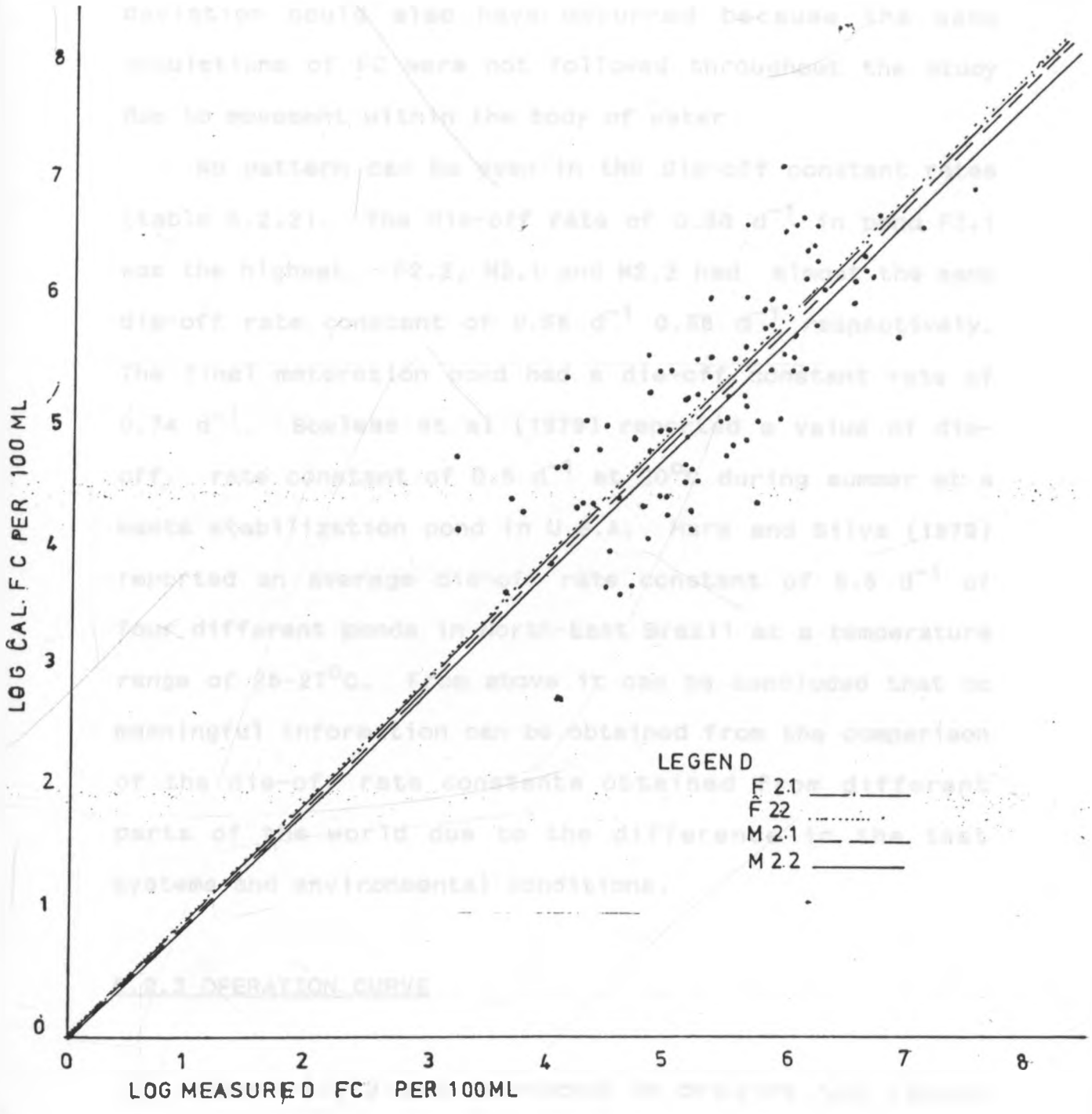


FIG. 6.23.2 CALCULATED FC AGAINST MEASERED FC [APPENDIX C 2]

from the expectations because the design equation for complete mixing does not apply in this case. The deviation could also have occurred because the same populations of FC were not followed throughout the study due to movement within the body of water.

No pattern can be seen in the die-off constant rates (table 6.2.2). The die-off rate of 0.98 d^{-1} in pond F2.1 was the highest. F2.2, M2.1 and M2.2 had almost the same die-off rate constant of 0.56 d^{-1} 0.56 d^{-1} respectively. The final maturation pond had a die-off constant rate of 0.74 d^{-1}. Bowless et al (1979) reported a value of die-off. rate constant of 0.5 d^{-1} at 20^{o}C during summer at a waste stabilization pond in U.S.A. Mara and Silva (1979) reported an average die-off rate constant of 9.5 d^{-1} of four different ponds in North-East Brazil at a temperature range of 25-27^{o}C. From above it can be concluded that no meaningful information can be obtained from the comparison of the die-off rate constants obtained from different parts of the world due to the difference in the test systems and environmental conditions.

6.2.3 OPERATION CURVE

Figure 6.2.3 was developed to predict the lagoon hydraulic retention time required to achieve a specified faecal coliform removal at Dandora treatment works. The figure indicate that to reduce an influent faecal coliform concentration of 10^{6} organisms per 100 ml by a factor of

10^3, so that the effluent faecal concentration would be 10^3 organisms per 100 ml, would require a lagoon hydraulic retention time of approximately 30 days.

Figure 6.2.3. can be used to estimate the faecal coliform density in the final effluent when the retention time and raw sewage faecal coliform counts are known.

TABLE 6.2.3 RETENTION TIME REQUIRED IN DANDORA POND SYSTEM TO REDUCE AN INFLUENT COLIFORM LEVEL TO REQUIRED EFFLUENT COLIFORM LEVEL.

LOCATION	TOTAL RETENTION TIME (d)	EFFLUENT FC Ne (per 100ml)	Ni / Ne
F2.1	10.4	$2.61X10^6$	11.15
F2.2	16.0	$9.35X10^5$	31.12
M2.1	20.5	$1.86X10^5$	156.4
M2.2	25.6	$4.80X10^4$	606.2

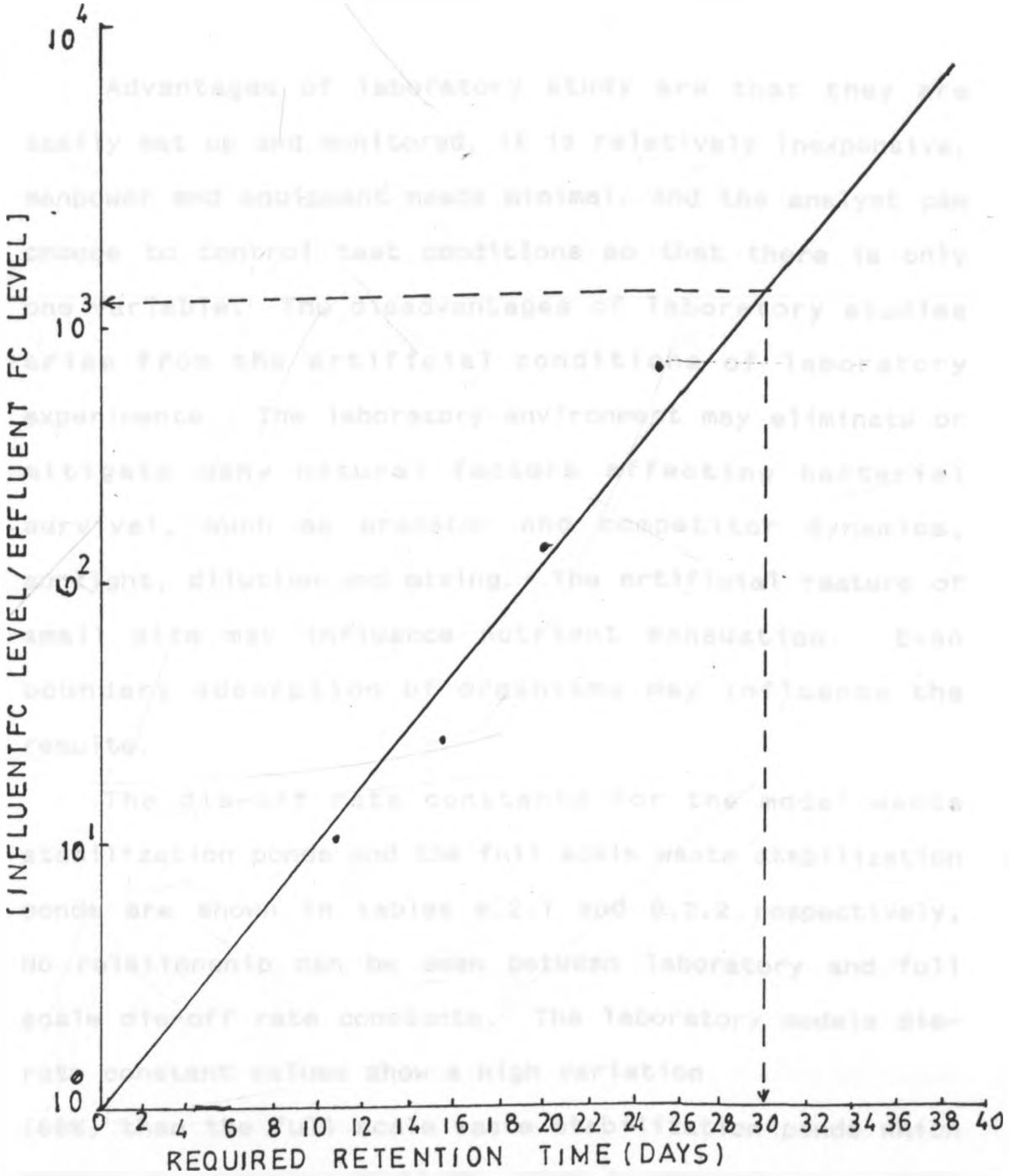


FIG 6.2.3 RETENTION TIME REQUIRED IN DANDORA POND SYSTEM TO REDUCE AN INFLUENT COLIFORM LEVEL TO A REQUIRED EFFLUENT COLIFORM LEVEL

6.2.4 COMMENTS ON LABORATORY RESULTS AND FIELD RESULTS:

Advantages of laboratory study are that they are easily set up and monitored, it is relatively inexpensive, manpower and equipment needs minimal, and the analyst can choose to control test conditions so that there is only one variable. The disadvantages of laboratory studies arise from the artificial conditions of laboratory experiments. The laboratory environment may eliminate or mitigate many natural factors affecting bacterial survival, such as predator and competitor dynamics, sunlight, dilution and mixing. The artificial feature of small size may influence nutrient exhaustion. Even boundary adsorption of organisms may influence the results.

The die-off rate constants for the model waste stabilization ponds and the full scale waste stabilization ponds are shown in tables 6.2.1 and 6.2.2 respectively. No relationship can be seen between laboratory and full scale die-off rate constants. The laboratory models die-rate constant values show a high variation (86%) than the full scale waste stabilization ponds which showed a variation of 44.5% . What is strongly suggested by these results is that laboratory models should not be used to establish die-off coefficient for indicator

bacteria, if such coefficients are to be used in mathematical models or formulation involving full scale waste stabilization ponds. If such coefficients are required then a full scale plant study should be carried out.

6.3 BIOCHEMICAL OXYGEN DEMAND (BOD), CHEMICAL OXYGEN DEMAND (COD), SUSPENDED SOLIDS (SS), AND pH

6.3.1 BOD AND COD

In wastewater engineering BOD and COD are frequently used to define influent and effluent characteristics and to assess the process efficiency. Tables 6.3.1.1 and 6.3.1.2 show BOD and COD analysis of grab samples at Dandora treatment works respectively. From the tables, the average percentage removal of BOD is 87.6% and 82% for COD.

The ratio of COD to BOD in all samples is high, particularly so in the final effluent (Average of 5.14) indicating the presence of more organic matter not readily biodegradable and industrial wastes. Considering the ratios of COD:BOD of the treated effluents, it is apparent that the ratio increases as the biological treatment proceeds. The increase of COD:BOD ratio is logical since biodegradable matter is consumed during treatment and nonbiodegradable organics remain to give relatively much

higher COD values than BOD after treatment. The final average effluent BOD (50 mg/L) is outside the Nairobi city Commission standard for discharge to water course (Appendix F).

TABLE 6.3.1.1 BOD (mg/L) (UNFILTERED) AT DANDORA TREATMENT WORKS

DATE	RAW SEWAGE	F2.1 EFFLUENT	F2.2 EFFLUENT	M2.1 EFFLUENT	M2.2 EFFLUENT
12/1/89	500 mg/L	60 mg/L	44 mg/L	73 mg/L	75 mg/L
24/2/89	400 mg/L	70 mg/L	45 mg/L	36 mg/L	36 mg/L
29/3/89	380 mg/L	100 mg/L	128 mg/L	60 mg/L	78 mg/L
25/4/89	280 mg/L	64 mg/L	36 mg/L	50 mg/L	18 mg/L
AVERAGE	404 mg/L	74 mg/L	63 mg/L	57 mg/L	50 mg/

TABLE 6.3.1.2 COD (mg/L) (UNFILTERED) AT DANDORA TREATMENT WORKS

DATE	RAW SEWAGE	F2.1 EFFLUENT	F2.2 EFFLUENT	M2.1 EFFLUENT	M2.2 EFFLUENT
12/1/89	1480mg/L	319mg/L	262mg/L	283mg/L	298mg/L
24/2/89	1984mg/L	502mg/L	405mg/L	321mg/L	305mg/L
29/3/89	1280mg/L	385mg/L	344mg/L	293mg/L	259mg/L
25/3/89	1490mg/L	245mg/L	292mg/L	343mg/L	296mg/L
9/5/89	916mg/L	233mg/L	191mg/L	164mg/L	126mg/L
AVERAGE	1430mg/L	337mg/L	299mg/L	281mg/L	257mg/L
COD:BOD	3.5	4.55	4.74	4.9	5.14

6.3.2 SUSPENDED SOLIDS (SS)

The suspended solid concentrations in the influent and various ponds' effluents are shown in table 6.3.2. The SS concentration of the influent fluctuated from 700 mg/L to 548 mg/L with an average of 662 mg/L, that of the effluent varied from 148 mg/L to 40 mg/L with an average of 87.6 mg/L. It is observed that the variations of SS in

the pond effluents did not show any particular trend.

TABLE 6.3.2 SUSPENDED SOLIDS (UNFILTERED)

DATE	RAW SEWAGE	F2.1 EFFLUENT	F2.2 EFFLUENT	M2.2 EFFLUENT	M2.2 EFFLUENT
12/1/89	700mg/L	70mg/L	138mg/L	134mg/L	86mg/L
24/2/89	752mg/L	108mg/L	44mg/L	80mg/L	40mg/L
29/3/89	640mg/L	312mg/L	304mg/L	160mg/L	140mg/L
25/4/89	668mg/L	288mg/L	176mg/L	160mg/L	148mg/L
9/5/89	548mg/L	44mg/L	104mg/L	80mg/L	24mg/L
AVERAGE	662mg/L	159mg/L	153.2mg/L	123mg/L	87.6mg/L

Final average effluent suspended solids (87.6 mg/L) is not satisfactory and is well outside the Nairobi City Commission standard (30 mg/L) for discharge to water courses. The high figure of SS is probably due to the presence of excessive quantities of algae in the effluent.

6.3.3 pH

A close correlation between pH and coliform reduction

has been postulated with values of pH on the order of 10 or higher detrimental to coliform (Parhad and Rao 1974). Because pH in the Dandora pond system rarely exceeded 8.0 (appendix E) ranging from 7.0 to 8.0 it is doubtful that this factor is of significance to coliform removal at Dandora works.

6.4 TEMPERATURE

Many factors have been reported in the literature which are claimed to influence either the rate or extent of enteric bacteria die-off. Temperature has been one of the most frequently considered factor: Virtually all studies concluding that the rate of die-off increases as temperature increases.

Appendix D shows the daily maximum and minimum air temperatures at Dandora treatment works recorded during the study period. The maximum and minimum temperatures recorded were 38^{o}C and 12^{o}C respectively. The mean minimum temperature at Dandora treatment works was 17.7^{o}C with a standard deviation of 3.1^{o}C and the mean maximum temperature recorded was 31.8^{o}C with a standard deviation of 2.8^{o}C. The mean air temperature can be approximated to 24.8^{o}C.

The mean of the daily maximum and minimum temperatures at the mid-depth of the pond is a close estimate of the mean daily pond temperature. Table 6.4 shows the maximum and minimum temperatures recorded at

TABLE 6.4 MID-DEPTH TEMPERATURES AT DANDORA TREATMENT WORKS

DATE	LOCATION	MINIMUM TEMPERATURE	MAXIMUM TEMPERATURE	MEAN TEMPERATURE
13/2/89	F2.1	18.0°C	21.0°C	19.5°C
13/2/89	M2.2	21.0°C	25.0°C	23°C
14/2/89	F2.1	22.0°C	25.0°C	23.5°C
14/2/89	M2.2	20°C	25.0°C	22.5°C
16/2/89	F2.1	21°C	24°C	22.5°C
16/2/89	M2.2	22°C	25°C	23.5°C
17/2/89	F2.1	21°C	21.5°C	21.25°C
17/2/89	M2.2	22°C	22°C	22°C
3/5/89	F2.2	23°C	30.0°C	26.5°C
3/5/89	M2.2	24.0°C	29.5°C	26.75°C
4/5/89	F2.2	23°C	27.5°C	25.25°C
5/5/89	F2.2	23°C	28°C	25.5°C

mid-depth of ponds at Dandora treatment works. From the table, the minimum and maximum temperature recorded at mid-depth of the ponds were 21^{o}C and 30.0^{o} respectively. The mean of daily maximum and minimum temperature was 23.5^{o}C with a standard deviation of 2.2^{o}C.

The mean of the daily maximum and minimum temperatures (24.8^{o}) of air at Dandora treatment works is higher than the mean of daily maximum and minimum temperature (23.5^{o}C) at mid-depth of the ponds. However the mean air temperature can be a good approximate of the pond water temperature. This observation is based only on the results presented in this study, more data would be required to draw the above conclusion.

6.5 ALGAL SPECIATION AND CHLOROPHYLL A

6.5.1 ALGAE SPECIATION

Algal population are many times more dense in waste stabilization ponds than those found in natural unenriched waters, and are responsible for the most striking visual feature of ponds namely the bright green colouration of the pond water.

Table 6.5 shows algae speciation at Dandora treatment works. Speciation was done at the Ministry of Water Development research laboratory by Dr. S.W. Mills.

TABLE 6.5. ALGAE SPECIATION

DATE	EFFLUENT FROM	TYPES OF ALGAE FOUND
14/2/89	F2.1	Mostly *Chlorella*, some *Euglena* and *Oocystiscrasse*
	F2.2	As in F2.1
	M2.1	Mostly *chlorella* occasional *Euglena*
	M2.2	As in M2.1
3/5/89	F2.1	Mostly *chlorella* but some *Euglena* and *Oocyts*
	F2.2	As in F2.1
	M2.1	Almost all *chlorella*, occasional
	M2.2	As in M2.1

It is reported in literature (section 3.1.2) that as a general rule, the further down a pond series i.e. the less polluted the pond water, the more diverse the speciation. It is therefore possible to roughly assess the organic loading on an individual pond or series of ponds by simple microscopic examination of algae species. Two principal genera (Chlorophyta and Euglenophyta) were present at Dandora waste stabilization ponds. From the above information, it can be concluded that the ponds at Dandora treatment works were highly organically loaded.

Fogg and Davis and Gloyna (as reported by Moeller et al 1980) documented numerous studies indicating a direct relationship between heavy algal blooms and low coliform density. Davies and Gloyna stated that the existence of a greater variety of algal species indicates a more complex environment which in some cases increased coliform reduction rates. Little influence upon die-off of enteric bacteria is exerted by a single algae species.

6.5.2 CHLOROPHYLL A

The quantity of algal biomass within a pond is more important than the different species present. Because of the wide variation in sizes between different species of

waste stabilization pond algae, it is clear that the number of cells per unit volume is an inappropriate parameter when comparing the biomass of one pond with another. For instance, large Euglena cells may be 3 to 4 times larger than small Euglena cells or 85 times larger than Chlorella cells. It is thus quite common in a pond water sample for Euglena to be dominant by biomass but Chlorella dominant by numbers. A more apprppriate measure of algal biomass would be dry weight of the algae cells per unit volume. However, in a natural water sample, it is difficult to eliminate the dry weight contribution from other sources such as suspended particulate organic matter and bacterial cells. It is therefore necessary to find a parameter that is specific to algal cells and that is approximately proportional to the dry weight or volume of algal cells. A suitable parameter is the photosynthetic pigment chlorophyll A.

Several different types of chlorophyll occur in algae but the most important quantitatively is chlorophyll A and the proportion per dry weight of algal cells does not vary much from one species to the next. Although this is disputed, a more significant source of error is variation in the extractability of the pigment from cells of different species. It is easily extracted from algae with thin cell walls such as Euglena, by extraction is difficult from other algae, such as Chlorella, which possess thick cell walls.

TABLE 6.6 CHLOROPHYLL A (CHLA)

DATE	F2.1 ug/L	F2.2 ug/L	M2.1 ug/L	M2.2 ug/L
13/2/89	849	918	918	706
14/2/89	495	848	848	565
3/5/89	1677	1854	706	706
4/5/89	1530	918	647	706
5/5/89	1677	1589	706	706

The table shows chlorophyll A concentrations at Dandora treatment working ranging from 565 to 1677 ug/L chla. (Analysis was done by personnel at the ministry of water development research section). This falls within the reported range in literature. Pearson and Kong (1986) noted that algal population in an efficiently functioning facultative pond will be large with chlorophyll concentrations in the range of 1000 to 3000 ug/L or even

higher. The chlorophyll concentrations fluctuates quite considerably in ponds F2.1 and F2.2, the fluctuations are less pronounced in M2.1 and M2.2.

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

(1) Dandora treatment work is currently overloaded hydraulically. The average maximum flow of 4.88 x 10^3 m^3/h (117 x 10^3 m^3/d is 390% of the design flow. The average minimum flow of 2.2 x 10^3 m^3/h (52.8 x $10^3/d$) is 176% of the design flow while the average mean flow 3.4 x 10^3 m^3/h (81.6 x 10^3 m^3/d) is 272% of the design flow. The design flow is 1.25 x 10^3 m^3 /h (30 x 10^3 m^3/d. Nairobi City Commission which owns and operates the treatment work is aware of the overloading. Extensions onto the treatment works are under construction.

2. Faecal coliform densities both at Dandora waste stabilization ponds and laboratory waste stabilization ponds exhibit a distinct parallelism throughout the study period, that is an increase/decrease in faecal coliform density from one sample date to the next in a pond upper in the series is usually accompanied by a rise/fall in faecal coliform density in the ponds down the series.

3. Studies from laboratory waste stabilization model ponds showed that the die-off rate constant was slower in the ponds with lower initial coliform counts, and that the die-off rate constant decreases down the pond series and

with increase in retention time.

4. Dandora treatment works achieved an average overall percentage removal of 99.984% and a survival of 0.016%. The design residual coliforms in effluent was <0.01% of the influent. Despite the overloading the faecal coliform removal is satisfactory. The average influent faecal coliform count is 2.9×10^7 per 100 ml and the average faecal coliform concentration in the final effluent is 4.8×10^4 per 100 ml. This is outside the recommended standard for discharge to a receiving stream (5000 FC per 100 ml).

5. The overall percentage removal of bacteria in waste stabilization ponds is not good enough to provide description of bacteria concentration in the effluents. The percentage removal may appear highly impressive but the degree of survival may be highly significant wherever incoming concentrations are great.

6. A curve is presented for use in estimating faecal coliform density in the final effluent at Dandora treatment work when the retention time and raw sewage FC counts are known.

7. No meaningful information can be obtained from the comparison of the die-off rate constants obtained from different parts of the world due to the difference in the test systems and environmental conditions.

7.2 RECOMMENDATIONS

(1) In future studies on hydraulic characteristics at Dandora Treatment Works should be carried to establish the hydraulic flow regimes in the ponds.

(2) A study of faecal coliform removal from a series of pilot scale waste stabilization ponds should be undertaken at Dandora treatment works.

(3) Further studies, particularly of the effect of algae on faecal coliform removal and the relationship between faecal coliform removal and organic matter removal in waste stabilization ponds are desirable.

(4) From the present study, it appears that laboratory models should not be used to establish die-off coefficients for indicator bacteria for use in full scale waste stabilization ponds. If such coefficients are required then a full scale plant study should be carried.

(5) A study should be undertaken to establish the relationship between ambient air temperature and mid-depth pond temperature. The minimum ambient air temperature is normally adopted for pond design.

(6) Research into the development of more indicator organisms based on epidemiological data should in future be encouraged.

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APPENDIX

CONTENTS

(A) I DANDORA WASTE STABILIZATION PONDS

FLOW MEASUREMENTS

	1988			1989
DATE	AVERAGE INFLUENT FLOW x1000m^3/h	AVERAGE EFFLUENT FLOW x1000m^3/h	LOSS x1000m^3/h	AVERAGE INFLUENT FLOW x1000m^3/h
1/1	2.31	1.00	1.13	2.18
2/1	2.53	1.10	1.40	2.10
3/1	2.49	1.00	1.49	2.20
4/1	2.55	0.98	1.57	2.44
5/1	2.67	0.88	1.79	2.22
6/1	2.71	0.46	2.25	4.80
7/1	2.57	1.80	0.77	4.10
8/1	3.17	1.80	1.37	2.95
9/1	2.74	1.24	1.50	2.62
10/1	3.23	1.27	1.96	2.64
11/1	3.23	1.40	1.83	2.49
12/1	2.97	1.12	1.85	2.55
13/1	2.95	1.46	1.49	2.73
14/1	2.61	1.58	1.03	2.40
15/1	2.60	1.27	1.33	2.70
16/1	2.50	1.14	1.36	2.95
17/1	2.55	1.29	1.26	2.91
18/1	4.92	1.33	3.59	3.18
19/1	3.06	1.27	1.79	2.65
20/1	2.87	1.27	1.60	2.30
21/1	2.77	1.41	1.36	2.74
22/1	2.74	1.79	0.95	4.57
23/1	2.64	1.44	1.20	3.01
24/1	2.47	0.68	1.79	2.75
25/1	2.72	1.29	1.43	2.93
26/1	2.57	0.82	1.75	3.06
27/1	2.58	0.81	1.77	3.00
28/1	1:92	0.38	1.54	2.57
29/1	2.35	0.75	1.6	2.05

DANDORA WASTE STABILIZATION PONDS

FLOW MEASUREMENTS

	1988			1989
DATE	AVERAGE INFLUENT FLOW x1000m^3/h	AVERAGE EFFLUENT FLOW x1000m^3/h	LOSS x1000m^3/h	AVERAGE INFLUENT FLOW x1000m^3/h
30/1	2.35	0.58	1.72	2.28
31/1	2.32	0.68	1.64	2.36
1/2	2.68	0.65	2.03	2.23
2/2	2.78	0.65	2.13	2.50
3/2	2.22	0.54	1.68	2.11
4/2	2.70	0.60	2.10	1.77
5/2	2.77	0.57	2.20	2.20
6/2	2.67	0.58	2.09	2.47
7/2	2.47	0.54	1.93	2.63
8/2	2.57	0.51	2.06	2.87
9/2	2.65	0.54	2.11	2.69
10/2	3.06	0.48	2.58	3.12
11/2	2.92	0.53	2.39	2.73
12/2	3.02	0.48	2.54	2.27
13/2	3.11	0.50	2.61	2.63
14/2	3.05	0.40	2.65	2.51
15/2	3.28	0.44	2.84	2.60
16/2	3.25	0.44	2.81	2.41
17/2	2.73	0.45	2.28	2.53
18/2	2.75	0.42	2.33	3.04
19/2	2.63	0.45	2.18	3.23
20/2	2.59	0.41	2.18	2.57
21/2	3.40	0.40	3.00	2.43
22/2	3.20	0.40	2.80	2.69
23/2	3.13	0.38	2.75	2.49
24/2	2.99	0.35	2.64	2.03
25/2	2.90	0.35	2.55	2.37
26/2	2.47	0.35	2.22	2.42

(A)II DANDORA WASTE STABILIZATION PONDS
FLOW MEASUREMENTS

	1988			1989
DATE	AVERAGE INFLUENT FLOW x1000m^3/h	AVERAGE EFFLUENT FLOW x1000m^3/h	LOSS x1000m^3/h	AVERAGE INFLUENT FLOW x1000m^3/h
27/2	3.04	0.35	2.69	2.30
28/2	3.17	0.35	2.82	2.40
29/2	2.95	0.35	2.6	-
1/3	2.62	0.29	2.33	2.62
2/3	2.73	0.28	2.45	2.52
3/3	2.82	0.30	2.52	3.00
4/3	2.50	0.30	2.20	3.77
5/3	2.85	0.30	2.55	3.55
6/3	2.79	0.30	2.49	3.82
7/3	2.62	0.30	2.32	3.82
8/3	2.77	0.30	2.47	3.81
9/3	2.63	0.26	2.37	3.90
10/3	2.52	0.25	2.27	3.50
11/3	2.68	0.25	2.43	3.70
12/3	2.47	0.25	2.22	3.65
13/3	2.61	0.24	2.37	4.11
14/3	2.54	0.22	2.32	4.67
15/3	2.59	0.23	2.36	3.88
16/3	2.91	0.15	2.76	3.65
17/3	2.45	0.25	2.20	3.52
18/3	2.59	0.24	2.35	3.51
19/3	3.03	0.25	2.78	3.76
20/3	2.56	0.24	2.32	4.60
21/3	-	-	-	5.32
22/3	-	-	-	6.47
23/3	-	-	-	4.14
24/3	-	-	-	3.16
25/3	-	-	-	3.63

DANDORA WASTE STABILIZATION PONDS

FLOW MEASUREMENTS

	1988			1989
DATE	AVERAGE INFLUENT FLOW x1000m^3/h	AVERAGE EFFLUENT FLOW x1000m^3/h	LOSS x1000m^3/h	AVERAGE INFLUENT FLOW x1000m^3/h
26/3	-	-	-	3.27
27/3	-	-	-	3.31
28/3	-	-	-	3.70
29/3	4.49	3.00	1.49	3.63
30/3	3.46	2.95	0.51	3.68
31/3	-	-	-	3.83
1/4	2.92	2.95	-0.03	3.95
2/4	3.38	2.95	0.43	3.67
3/4	3.16	2.95	0.21	3.64
4/4	2.93	3.00	-0.07	3.61
5/4	3.81	2.95	0.86	5.40
6/4	6.05	3.00	3.05	4.55
7/4	5.60	3.00	2.60	7.51
8/4	4.00	3.00	1.00	6.16
9/4	3.91	3.00	0.91	6.61
10/4	5.89	3.00	2.89	5.55
11/4	3.03	3.35	0.32	5.45
12/4	3.36	3.66	-0.30	5.00
13/4	3.35	5.37	-2.02	4.48
14/4	3.08	5.29	-2.21	4.19
15/4	3.02	4.30	-1.28	3.85
16/4	3.45	4.30	-0.85	3.62
17/4	3.08	4.50	-1.42	3.41

Mean Average flow Jan-April 1989 = $3.38 \times 10^3 m^3/h$

Mean Average flow Jan-April 1988 = $3.10 \times 10^3 m^3/h$

Mean Average loss Jan-April 1988 = $1.75 \times 10^3 m^3/h$

(A) II DANDORA WASTE STABILIZATION PONDS

JANUARY, 1989 - DAILY PEAK FLOW - LOWEST FLOW - AVERAGE FLOW IN M^3/h

DATE	HIGHEST FLOW($x10^3$)	LOWEST FLOW($x10^3$)	AVERAGE FLOW($x10^3$
1.1.89	3.4	1.1	2.18
2.1.89	3.0	1.1	2.10
3.1.89	2.9	1.0	2.20
4.1.89	3.3	1.1	2.44
5.1.89	4.1	0.8	2.22
6.1.89	5.2	2.4	4.80
7.1.89	6.2	3.1	4.10
8.1.89	4.6	2.3	2.95
9.1.89	4.2	2.0	2.62
10.1.89	4.1	1.9	2.64
11.1.89	3.7	1.5	2.49
12.1.89	3.4	1.8	2.55
13.1.89	3.6	1.8	2.73
14.1.89	3.2	1.7	2.40
15.1.89	3.4	2.1	2.70
16.1.89	4.6	2.0	2.95
17.1.89	4.0	1.5	2.91
18.1.89	4.0	2.2	3.18
19.1.89	4.0	2.1	2.65
20.1.89	3.6	1.6	2.30
21.1.89	3.4	1.1	2.74
22.1.89	6.5	3.1	4.57
23.1.89	4.6	2.2	3.01
24.1.89	3.8	1.8	2.75
25.1.89	4.3	1.4	2.93
26.1.89	4.1	1.6	3.06
27.1.89	4.2	1.8	3.00
28.1.89	4.4	1.7	2.57
29.1.89	3.6	1.1	2.05
30.1.89	3.5	1.1	2.28
31.1.89	3.7	1.7	2.36

DANDORA WASTE STABILIZATION PONDS

FEBRUARY,1989 - DAILY PEAK FLOW - LOWEST FLOW - AVERAGE FLOW IN M^3/h

DATE	HIGHEST FLOW	LOWEST FLOW	AVERAGE FLOW
1.2.89	3.70	1.50	2.23
2.2.89	3.20	1.50	2.50
3.2.89	3.20	1.20	2.11
4.2.89	3.10	1.00	1.77
5.2.89	3.00	1.70	2.20
6.2.89	3.90	1.30	2.47
7.2.89	5.00	3.10	2.63
8.2.89	3.80	0.30	2.87
9.2.89	4.00	1.40	2.69
10.2.89	4.00	2.20	3.12
11.2.89	3.70	1.30	2.73
12.2.89	3.10	1.40	2.27
13.2.89	4.20	1.60	2.63
14.2.89	4.10	1.00	2.51
15.2.89	3.80	1.00	2.60
16.2.89	3.80	1.20	2.41
17.2.89	4.00	1.600	2.53
18.2.89	5.00	1.30	3.04
19.2.89	4.60	2.30	3.23
20.2.89	3.70	0.90	2.57
21.2.89	3.40	1.40	2.43
22.2.89	4.00	1.50	2.69
23.2.89	3.60	1.40	2.49
24.2.89	4.00	1.00	2.03
25.2.89	3.90	1.20	2.37
26.2.89	3.40	1.60	2.42
27.2.89	3.80	1.20	2.30
28.2.89	3.40	1.30	2.40

DANDORA WASTE STABILIZATION PONDS

MARCH,1989 - DAILY PEAK FLOW - LOWEST FLOW - AVERAGE FLOW IN $M^3/h.$

DATE	HIGHEST FLOW	LOWEST FLOW	AVERAGE FLOW
1.3.89	4.00	1.30	2.62
2.3.89	3.80	1.40	2.52
3.3.89	4.40	0.80	3.00
4.3.89	5.00	2.80	3.77
5.3.89	4.70	2.80	3.55
6.3.89	5.40	1.90	3.82
7.3.89	5.60	1.90	3.82
8.3.89	4.70	2.50	3.81
9.3.89	5.90	2.30	3.90
10.3.89	4.60	2.00	3.50
11.3.89	5.00	2.80	3.70
12.3.89	4.60	2.80	3.65
13.3.89	5.20	2.80	4.11
14.3.89	8.00	3.50	4.67
15.3.89	4.60	3.20	3.88
16.3.89	4.80	2.00	3.65
17.3.89	5.20	2.80	3.52
18.3.89	4.30	2.50	3.51
19.3.89	5.80	3.20	3.76
20.3.89	7.80	3.00	4.60
21.3.89	8.00	4.20	5.32
22.3.89	9.30	4.20	6.47
23.3.89	5.30	3.50	4.14
24.3.89	4.60	3.10	3.16
25.3.89	4.40	3.00	3.63
26.3.89	4.70	2.90	3.27
27.3.89	4.20	2.70	3.31
28.3.89	4.40	2.90	3.70
29.3.89	4.70	1.60	3.63
30.3.89	4.40	2.90	3.68
31.3.89	4.70	3.00	3.83

DANDORA WASTE STABILIZATION PONDS

APRIL,1989 - DAILY PEAK FLOW - LOWEST FLOW - AVERAGE FLOW IN M^3/h

DATE	HIGHEST FLOW	LOWEST FLOW	AVERAGE FLOW
1.4.89	7.00	2.70	3.95
2.4.89	6.90	2.80	3.67
3.4.89	5.40	2.80	3.64
4.4.89	4.70	2.50	3.61
5.4.89	8.80	3.30	5.40
6.4.89	6.60	3.10	4.55
7.4.89	9.70	4.70	7.51
8.4.89	9.40	4.10	6.16
9.4.89	9.00	4.00	6.61
10.4.89	6.30	3.90	5.55
11.4.89	8.00	3.70	5.45
12.4.89	6.80	3.90	5.00
13.4.89	6.00	3.70	4.48
14.4.89	5.70	3.10	4.19
15.4.89	4.70	3.00	3.85
16.4.89	4.90	1.90	3.62
17.4.89	4.60	2.70	3.41
18.4.89	5.50	2.20	4.96
19.4.89	5.40	2.00	3.42
20.4.89	5.00	2.50	2.71
21.4.89	4.50	2.20	3.23
22.4.89	5.60	2.10	3.22
23.4.89	3.90	2.60	2.60
24.4.89	5.00	1.80	3.69
25.4.89	6.80	2.10	4.62
26.4.89	6.90	3.60	4.97
27.4.89	8.40	2.90	4.44
28.4.89	8.30	3.10	5.51
29.4.89	7.80	3.10	4.54
30.4.89	5.00	3.10	3.85

Average maximum flow=$4.9x10^3m^3/h$ Standard deviation=$1.1x10^3m^3/h$

Standard deviation=$1.57x10^3m^3/h$ Average minimum flow=$2.2x10^3$

Average mean flow=$3.38x10^3m^3/h$ Standard deviation=$9.1x10^3m^3/h$

Design flow = $1.25x10^3m^3/h$.

B FAECAL COLIFORM COUNTS: LABORATORY SCALE WASTE STABILIZATION PONDS

RETENTION TIME: 2.7d

DATE	LOCATION	INFLUENT FC PER 100ml	EFFLUENT FC Per 100ml	%REMOVAL	OVERALL % REMOVAL
14/2/89	MRS	1.65×10^5			
	P1	1.65×10^5	2.45×10^4	85.15	85.15
	P2	2.45×10^4	5.15×10^3	78.98	96.88
	P3	5.15×10^3	1.6×10^3	68.93	99.03
16/2/89	MRS	3.2×10^5			
	P1	3.2×10^5	3.25×10^4	89.84	89.84
	P2	3.25×10^4	5.45×10^3	83.23	98.30
	P3	5.45×10^3	1.35×10^3	75.52	99.58
18/2/89	MRS	2.3×10^5			
	P1	2.3×10^5	2.3×10^4	90.00	90
	P2	2.3×10^4	6.45×10^3	71.96	97.20
	P3	6.45×10^3	1.95×10^3	69.77	99.15
20/2/89	MRS	1.8×10^5			
	P1	1.8×10^5	3×10^4	83.33	83.33
	P2	3×10^4	9×10^3	70.00	95.0
	P3	9×10^3	3.3×10^3	63.33	98.17
22/2/89	MRS	1.18×10^6			
	P1	1.18×10^6	2×10^5	83.00	98.31
	P2	2×10^5	1.7×10^4	91.00	98.56
	P3	1.7×10^4	5.55×10^3	67.35	99.53
24/2/89	MRS	7.25×10^4			
	P1	7.25×10^4	9.5×10^3	86.90	86.90
	P2	9.5×10^3	2.6×10^3	72.63	96.41
	P3	2.6×10^3	7×10^2	73.08	99.03

RETENTION TIME: 3.2d

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	%REMOVAL	OVERALL % REMOVAL
8/3/89	MRS	$4x10^3$			
	P1	$4x10^3$	$9x10^2$	77.5	77.5
	P2	$9x10^2$	$6x10^2$	77	85
	P3	$6x10^2$	$3.75x10^2$	37.5	90.6
10/3/89	MRS	$9.5x10^3$			
	P1	$9.5x10^3$	$3x10^2$	96.84	96.84
	P2	$3x10^2$	$1.75x10^2$	41.67	98.16
	P3	$1.75x10^2$	$1.2x10^2$	31.43	98.74
12/3/89	MRS	$1x10^3$			
	P1	$1x10^3$	$3.25x10^2$	67.5	67.5
	P2	$3.25x10^2$	$1.5x10^2$	53.85	85.00
	P3	$1.5x10^2$	$9x10^1$	40	91.00
14/3/89	MRS	$6.4x10^4$			
	P1	$6.4x10^4$	$7.4x10^3$	88.44	88.44
	P2	$7.4x10^3$	$1.6x10^3$	78.38	97.5
	P3	$1.6x10^3$	$5.85x10^2$	63.44	99.09
16/3/89	MRS	$2.65x10^4$			
	P1	$2.65x10^4$	$4.26x10^3$	83.92	83.92
	P2	$4.26x10^3$	$1.1x10^3$	74.18	93.33
	P3	$1.1x10^3$	$5.35x10^2$	51.36	96.76

RETENTION TIME: 4.5d

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	%REMOVAL	OVERALL % REMOVAL
18/4/89	MRS	$1x10^3$			
	P1	$1x10^3$	$1.2x10^2$	88	88
	P2	$1.2x10^2$	$1x10^2$	16	89
	P3	$1x10^2$	$7.2x10^1$	88	98.8
20/4/89	MRS	$1x10^3$			
	P1	$1x10^3$	$1.25x10^2$	87.5	87.5
	P2	$1.25x10^2$	$1.1x10^2$	12	89
	P3	$1.1x10^2$	$7.5x10^1$	31	92.5
22/4/89	MRS	$7.65x10^3$			
	P1	$7.65x10^3$	$1.25x10^3$	83.66	83.66
	P2	$1.25x10^3$	$2.85x10^2$	77.2	96.27
	P3	$2.85x10^2$	$9.75x10^1$	65.79	98.73
24/4/89	MRS	$8.7x10^3$			
	P1	$8.7x10^3$	$9.4x10^2$	89.20	89.20
	P2	$9.4x10^2$	$2x10^2$	78.72	97.70
	P3	$2x10^2$	$8.2x10^1$	59	98.77
26/4/89	MRS	$9.5x10^3$			
	P1	$9.5x10^3$	$4.46x10^2$	95.30	95.30
	P2	$4.46x10^2$	$1.3x10^2$	70.85	98.63
	P3	$1.3x10^2$	$9x10^1$	30.77	99.05

C I FAECAL COLIFORM COUNTS: DANDORA WASTE STABILIZATION PONDS

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	% REMOVAL	OVERALL % REMOVAL
4/1/89	RS	5.15×10^7			
	F2.1	5.15×10^7	1.25×10^7	75.73	75.728
	F2.2	1.25×10^7	9.7×10^5	92.24	98.117
	M2.1	9.7×10^5	1×10^5	89.69	99.806
	M2.2	1×10^5	1.75×10^3	99.82	99.997
13/1/89	RS	2.14×10^7			
	F2.1	2.14×10^7	4.6×10^6	78.50	78.505
	F2.2	4.6×10^6	8×10^5	82.61	96.26
	M2.1	8×10^5	4.65×10^5	41.88	97.827
	M2.2	4.65×10^5	3.7×10^4	92.04	99.827
17/1/89	RS	3.5×10^6			
	F2.1	3.5×10^6	7×10^5	80	80.000
	F2.2	7×10^5	2.8×10^5	60	92.000
	M2.1	2.8×10^5	9×10^4	67.857	97.428
	M2.2	9×10^4	3.7×10^4	58.89	98.943
26/1/89	RS	4.15×10^7			
	F2.1	4.15×10^7	2.83×10^6	93.18	93.18
	F2.2	2.83×10^6	1.45×10^6	48.76	96.506
	M2.1	1.45×10^6	4.15×10^5	71.38	99.000
	M2.2	4.15×10^5	1.3×10^4	96.87	99.969
2/2/89	RS	4×10^7			
	F2.1	4×10^7	4.6×10^6	88.50	88.500
	F2.2	4.6×10^6	1.75×10^6	61.96	95.625
	M2.1	1.75×10^6	2.9×10^5	83.43	99.275
	M2.2	2.9×10^5	1.3×10^4	95.52	99.967

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	%REMOVAL	OVERALL % REMOVAL
6/2/89	RS	5.8×10^7			
	F2.1	5.8×10^7	1.73×10^6	97.02	97.017
	F2.2	1.73×10^6	9.23×10^5	46.64	98.409
	M2.1	9.23×10^5	4.14×10^5	55.14	99.286
	M2.2	4.14×10^5	1.41×10^5	65.94	99.757
13/2/89	RS	1.07×10^7			
	F2.1	1.07×10^7	5.76×10^5	94.62	94.616
	F2.2	5.76×10^5	4.20×10^5	27.08	96.075
	M2.1	4.20×10^5	2.5×10^5	40.47	97.664
	M2.2	2.5×10^5	6.53×10^4	73.88	99.390
15/2/89	RS	6.7×10^6			
	F2.1	6.7×10^6	8.5×10^5	87.31	87.313
	F2.2	8.5×10^5	2.73×10^5	67.88	95.925
	M2.1	2.73×10^5	4.64×10^4	83.00	99.307
	M2.2	4.64×10^4	1.0×10^4	78.45	99.851
17/2/89	RS	5.7×10^7			
	F2.1	5.7×10^7	1.05×10^6	98.158	98.158
	F2.2	1.05×10^6	6.4×10^5	39.05	98.877
	M2.1	6.4×10^5	1.1×10^5	82.81	99.807
	M2.2	1.1×10^5	6.4×10^4	41.82	99.888
20/2/89	RS	3.5×10^7			
	F2.1	3.5×10^7	1.4×10^6	96.00	96.000
	F2.2	1.4×10^6	6.8×10^5	51.43	98.057
	M2.1	6.8×10^5	2.5×10^5	63.23	99.286
	M2.2	2.5×10^5	1.3×10^5	48	99.629

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	% REMOVAL	OVERALL% REMOVAL
22/2/89	RS	2.33×10^7			
	F2.1	2.33×10^7	4.03×10^6	82.70	82.704
	F2.2	4.03×10^6	3.93×10^5	90.248	98.313
	M2.1	3.93×10^6	2.73×10^5	93.05	98.828
	M2.2	2.73×10^5	3.6×10^4	86.81	99.845
24/2/89	RS	2×10^7			
	F2.1	2×10^7	2.97×10^6	85.15	85.15
	F2.2	2.97×10^6	8.6×10^5	72.52	95.92
	M2.1	8.16×10^5	1.63×10^5	80.02	99.185
	M2.2	1.63×10^5	5.63×10^4	65.46	99.718
28/2/89	RS	7×10^6			
	F2.1	7×10^6	6.25×10^5	72.85	72.857
	F2.2	6.25×10^5	2.93×10^5	84.58	95.814
	M2.1	2.93×10^5	1.3×10^4	95.56	99.814
	M2.2	1.3×10^4	1×10^4	23.08	99.857
2/3/89	RS	2.17×10^7			
	F2.1	2.17×10^7	1.27×10^6	94.15	94.15
	F2.2	1.27×10^6	4.27×10^5	66.38	98.03
	M2.1	4.27×10^5	2×10^4	95.32	99.908
	M2.2	2×10^4	4.5×10^3	77.5	99.979
6/3/89	RS	3.4×10^7			
	F2.1	3.4×10^7	1.7×10^6	95	95
	F2.2	1.7×10^6	5.6×10^5	67.06	98.35
	M2.1	5.6×10^5	1.4×10^5	75.00	99.588
	M2.2	1.4×10^5	5.25×10^4	62.5	99.846

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	% REMOVAL	OVERALL % REMOVAL
10/3/89	RS	$5.37x10^7$			
	F2.1	$5.37x10^7$	$8x10^5$	98.51	98.510
	F2.2	$8x10^5$	$2x10^5$	75	99.628
	M2.1	$2x10^5$	$7.67x10^4$	61.65	99.85
	M2.2	$7.67x10^4$	$1.65x10^4$	78.49	99.969
13/3/89	RS	$1.9x10^7$			
	F2.1	$1.9x10^7$	$8.15x10^5$	95.71	95.711
	F2.2	$8.15x10^5$	$5.3x10^5$	34.97	97.211
	M2.1	$5.3x10^5$	$7.9x10^4$	85.09	99.584
	M2.2	$7.9x10^4$	$2.25x10^4$	71.52	99.882
17/3/89	RS	$6.9x10^7$			
	F2.1	$6.9x10^7$	$5.7x10^6$	91.74	91.739
	F2.2	$5.7x10^6$	$1.08x10^6$	81.05	98.435
	M2.1	$1.08x10^6$	$1.7x10^5$	84.26	99.734
	M2.2	$1.7x10^5$	$1.25x10^5$	26.47	99.819
22/3/89	RS	$1.95x10^7$			
	F2.1	$1.95x10^7$	$1.8x10^6$	90.77	90.769
	F2.2	$1.8x10^6$	$3.05x10^5$	83.06	98.436
	M2.1	$3.05x10^5$	$2.65x10^5$	13.11	98.641
	M2.2	$2.65x10^5$	$7.55x10^4$	71.51	99.613
29/3/89	RS	$1.8x10^7$			
	F2.1	$1.8x10^7$	$1.4x10^6$	92.22	92.222
	F2.2	$1.4x10^6$	$3.0x10^5$	78.57	98.333
	M2.1	$3.0x10^5$	$3x10^4$	90	98.333
	M2.2	$3x10^4$	$1.65x10^4$	45	99.908

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	% REMOVAL	OVERALL % REMOVAL
3/4/89	RS	2.85×10^7			
	F2.1	2.85×10^7	2.3×10^6	91.93	91.930
	F2.2	2.3×10^6	1.1×10^6	52.17	96.140
	M2.1	1.1×10^6	4.05×10^5	63.18	98.579
	M2.2	4.05×10^5	6.0×10^4	85.18	99.789
7/4/89	RS	6.7×10^7			
	F2.1	6.7×10^7	1.15×10^6	98.28	98.284
	F2.2	1.15×10^6	6.25×10^5	45.65	99.067
	M2.1	6.25×10^5	2.25×10^5	64	99.664
	M2.2	2.25×10^5	4.9×10^4	78.22	99.927
11/4/98	RS	4.0×10^7			
	F2.1	4.0×10^7	4.8×10^6	88	88
	F2.2	4.8×10^6	1.24×10^6	74.17	96.9
	M2.1	1.24×10^6	5.8×10^5	53.23	98.55
	M2.2	5.8×10^5	1.5×10^5	74.14	99.625
13/4/89	RS	2.7×10^7			
	F2.1	2.7×10^7	5.4×10^6	80	80
	F2.2	5.4×10^6	1.23×10^6	77.22	95.444
	M2.1	1.23×10^6	1.95×10^5	84.15	99.278
	M2.2	1.95×10^5	9×10^4	53.85	99.667
17/4/89	RS	1.3×10^7			
	F2.1	1.3×10^7	6×10^5	95.38	95.385
	F2.2	6×10^5	2×10^5	66.67	98.462
	M2.1	2×10^5	1×10^5	50	99.231
	M2.2	1×10^5	1.69×10^3	98.31	99.987

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	% REMOVAL	OVERALL % REMOVAL
21/4/89	RS	3.7×10^7			
	F2.1	3.7×10^7	1.5×10^6	95.95	95.946
	F2.2	1.5×10^6	5×10^4	96.67	99.865
	M2.1	5×10^4	1.25×10^4	75	99.966
	M2.2	1.25×10^4	4.3×10^3	65.6	99.988
25/4/89	RS	1.75×10^7			
	F2.1	1.75×10^7	4.65×10^6	73.43	73.428
	F2.2	4.65×10^6	3×10^5	93.55	98.286
	M2.1	3×10^5	9×10^4	70.0	99.487
	M2.2	9×10^4	6×10^3	93.33	99.966
27/4/89	RS	6.4×10^7			
	F2.1	6.4×10^7	4.25×10^6	93.36	93.359
	F2.2	4.25×10^6	3.9×10^5	90.82	99.391
	M2.1	3.9×10^5	6.83×10^4	82.487	99.893
	M2.2	6.83×10^4	2.52×10^4	63.10	99.961
3/5/89	RS	7×10^6			
	F2.1	7×10^6	1.33×10^6	81	81
	F2.2	1.33×10^6	5.07×10^5	61.88	92.757
	M2.1	5.07×10^5	1.6×10^5	68.44	97.714
	M2.2	1.6×10^5	3.47×10^4	78.31	99.504
4/5/89	RS	4×10^6			
	F2.1	4×10^6	2.27×10^6	43.25	43.25
	F2.2	2.27×10^6	4.63×10^5	79.6	88.425
	M2.1	4.63×10^5	4.67×10^4	89.91	98.82
	M2.2	4.67×10^4	2.73×10^4	41.54	99.317

DATE	LOCATION	INFLUENT FC Per 100ml	EFFLUENT FC Per 100ml	%REMOVAL	OVERALL % REMOVAL
9/5/89	RS	2.8×10^7			
	F2.1	2.8×10^7	1.35×10^6	95.18	95.18
	F2.2	1.35×10^6	3.95×10^5	70.41	98.589
	M2.1	3.95×10^5	1.45×10^5	63.29	99.482
	M2.2	1.45×10^5	1.85×10^4	87.24	99.934
16/5/89	RS	3.95×10^6			
	F2.1	3.95×10^6	1.3×10^6	67.1	67.1
	F2.2	1.3×10^6	3.9×10^5	70	90.13
	M2.1	3.9×10^5	1.41×10^5	63.85	96.43
	M2.2	1.41×10^5	4.75×10^4	66.31	98.80
18/5/89	RS	1.34×10^7			
	F2.1	1.34×10^7	2.15×10^6	93.96	83.955
	F2.2	2.15×10^6	9.75×10^5	54.65	92.724
	M2.1	9.75×10^5	3.1×10^5	68.21	97.687
	M2.2	3.1×10^5	1.41×10^5	54.52	98.948

CII MEASURED AND CALCULATED FC PER 100ml AT DANDORA WASTE STABILIZATION PONDS

LOCATION F2.1

	DATE	MEASURE FC PER 100ml	CALCULATED FC PER 100ml
1	4/1/89	1.25×10^7	4.6×10^6
2	13/1/89	4.6×10^6	1.91×10^6
3	17/1/89	7.00×10^5	3.17×10^5
4	26/1/89	2.83×10^6	3.71×10^6
5	2/2/89	4.60×10^6	3.57×10^6
6	8/2/89	1.73×10^6	5.18×10^6
7	13/2/89	5.76×10^5	9.47×10^5
8	15/2/89	8.50×10^5	5.93×10^5
9	17/2/89	1.05×10^6	5.05×10^6
10	20/2/89	1.40×10^6	3.13×10^6
11	22/2/89	4.03×10^6	2.08×10^6
12	24/2/89	2.97×10^6	1.79×10^6
13	28/2/89	6.25×10^5	6.25×10^5
14	2/3/89	1.27×10^6	1.84×10^6
15	6/3/89	1.70×10^6	2.89×10^6
16	10/3/89	8.00×10^5	4.57×10^6
17	13/3/89	8.15×10^5	1.62×10^7
18	17/3/89	5.7×10^6	5.87×10^6
19	22/3/89	1.86×10^6	1.66×10^6
20	29/3/89	1.40×10^6	1.53×10^6
21	3/4/89	2.30×10^6	2.42×10^6
22	7/4/89	1.15×10^6	5.70×10^6
23	11/4/89	4.80×10^6	3.40×10^6
24	13/4/89	5.40×10^6	2.30×10^6
25	17/4/89	6.00×10^5	1.10×10^6
26	21/4/89	1.50×10^6	3.14×10^6
27	25/4/89	4.65×10^5	1.49×10^6
28	27/4/89	4.25×10^5	5.40×10^6
29	3/5/89	1.33×10^6	5.95×10^5
30	4/5/89	2.27×10^6	3.4×10^5
31	9/5/89	1.35×10^6	2.38×10^6
32	16/5/89	1.30×10^6	3.36×10^5
33	18/5/89	3.15×10^6	1.14×10^6

LOCATION F2.2

DATE	MEASURE FC PER 100ml	CALCULATED FC PER 100ml
4/1/89	9.7×10^5	1.11×10^6
13/1/89	8.00×10^5	4.62×10^6
17/1/89	2.80×10^5	7.56×10^4
26/1/89	1.45×10^6	8.97×10^5
2/2/89	1.75×10^6	8.64×10^5
6/2/89	9.23×10^5	1.25×10^6
13/2/89	4.20×10^5	2.31×10^5
15/2/89	2.73×10^5	1.45×10^5
17/2/89	6.40×10^5	1.23×10^6
20/2/89	6.80×10^5	7.56×10^5
22/2/89	3.93×10^5	5.03×10^5
24/2/89	8.16×10^5	4.32×10^5
28/2/89	2.93×10^5	1.51×10^5
2/3/89	4.27×10^5	4.69×10^5
6/3/89	5.60×10^5	7.34×10^5
10/3/89	2.00×10^5	1.16×10^6
14/3/89	5.30×10^5	4.10×10^5
17/3/89	1.08×10^6	1.49×10^6
22/3/89	3.05×10^5	4.21×10^5
29/3/89	3.00×10^5	3.89×10^5
3/4/89	1.10×10^6	6.85×10^5
7/4/89	6.25×10^5	1.45×10^6
11/4/89	1.24×10^6	8.64×10^5
13/4/89	1.23×10^6	5.83×10^5
17/4/89	2.00×10^5	2.81×10^5
21/4/89	5.00×10^4	7.99×10^5
25/4/89	3.00×10^5	3.78×10^5
27/4/89	3.90×10^5	1.38×10^6
3/5/89	5.07×10^5	1.51×10^5
4/5/89	4.63×10^5	8.64×10^4
9/5/89	3.95×10^5	6.05×10^5
16/5/89	3.90×10^5	8.53×10^4
18/5/89	9.75×10^5	2.89×10^5

LOCATION M2.1

DATE	MEASURE FC PER 100ml	CALCULATED FC PER 100ml
4/1/89	1.00×10^5	3.24×10^5
13/1/89	4.65×10^5	1.35×10^5
17/1/89	9.00×10^4	2.20×10^4
26/1/89	4.15×10^5	2.61×10^5
2/2/89	2.90×10^5	2.52×10^5
6/2/89	4.14×10^5	3.65×10^5
13/2/89	2.50×10^5	6.74×10^4
15/2/89	4.64×10^4	4.22×10^4
17/2/89	1.10×10^5	3.59×10^5
20/2/89	2.50×10^5	2.20×10^5
22/2/89	2.73×10^5	1.47×10^5
24/2/89	1.63×10^5	1.26×10^5
28/2/89	1.30×10^5	4.41×10^4
2/3/89	2.00×10^4	1.37×10^5
6/3/89	1.40×10^5	2.14×10^5
10/3/89	7.67×10^4	3.38×10^5
13/3/89	7.90×10^4	1.20×10^5
17/3/89	1.70×10^5	4.36×10^5
22/3/89	2.65×10^5	1.23×10^5
29/3/89	3.00×10^4	1.13×10^5
3/4/89	4.05×10^5	1.79×10^5
7/4/89	2.25×10^5	4.22×10^5
11/4/89	5.80×10^5	3.52×10^5
13/4/89	1.95×10^5	1.7×10^5
17/4/89	1.00×10^5	8.19×10^4
21/4/89	1.25×10^4	2.33×10^5
25/4/89	9.00×10^4	1.10×10^5
27/4/89	6.83×10^4	4.03×10^5
3/5/89	1.60×10^5	4.41×10^4
4/5/89	4.67×10^5	2.52×10^4
9/5/89	1.45×10^5	1.76×10^5
16/5/89	1.41×0^5	2.49×10^4
18/5/89	3.10×10^5	8.44×10^4

LOCATION M2.2

DATE	MEASURED INFLUENT FC Per 100ml	CALCULATED INFLUENT FC Per 100ml
4/1/89	1.75×10^3	6.79×10^4
13/1/89	3.70×10^4	2.82×10^4
17/1/89	3.70×10^4	4.62×10^3
26/1/89	1.30×10^4	5.47×10^4
2/2/89	1.30×10^4	5.28×10^4
6/2/89	1.41×10^5	7.65×10^3
13/2/89	6.53×10^4	1.41×10^4
15/2/89	1.00×10^4	8.84×10^3
17/2/89	6.40×10^4	7.52×10^4
20/2/89	1.30×10^5	4.62×10^4
22/2/89	3.60×10^4	3.07×10^4
24/2/89	5.63×10^4	2.64×10^4
28/2/89	1.00×10^4	9.23×10^3
2/3/89	4.50×10^4	2.86×10^4
6/3/89	5.25×10^4	4.49×10^4
10/3/89	1.65×10^4	7.08×10^4
13/3/89	2.25×10^4	2.51×10^4
17/3/89	1.25×10^5	9.10×10^4
22/3/89	7.55×10^4	2.57×10^4
29/3/89	1.65×10^4	2.37×10^4
3/4/89	6.00×10^4	3.76×10^4
7/4/89	4.90×10^4	8.84×10^4
11/4/89	1.50×10^5	5.28×10^4
13/4/89	9.00×10^4	3.56×10^4
17/4/89	1.69×10^3	1.72×10^4
21/4/89	4.30×10^3	4.88×10^3
25/4/89	6.00×10^3	2.31×10^4
27/4/89	2.52×10^4	8.44×10^4
3/5/89	3.47×10^4	9.23×10^3
4/5/89	2.73×10^4	5.28×10^3
9/5/89	1.85×10^4	2.64×10^4
16/5/89	4.75×10^4	5.21×10^3
18/5/89	1.41×10^5	1.77×10^4

CIII REGRESSION THROUGH THE ORIGIN

For regression through the origin, the linear equation is given by

$$Y = bx_i$$

where

$$b = \frac{\Sigma X_i Y_i}{\Sigma X_i} = \text{Regression coefficient}$$

The sample correlation coefficient, r is given by (approx.)

$$r = 6\surd \frac{\Sigma X^2}{\Sigma Y^2}$$

The calculated and measured faecal coliform data was first transformed to logarithmic form. After analysis:

F2.1, $Y = 1.001x$, $r = 0.992$

F2.2 $Y = 1.001x$, $r = 1.0$

M2.1 $Y = 0.991x$, $r = 0.996$

M2.2 $Y = 0.976x$, $r = 0.99$.

(D) DANDORA WASTE STABILIZATION PONDS

AIR TEMPERATURE(^{o}C) 1989

DATE	JANUARY		FEBRUARY		MARCH		APRIL		MAY	
	MIN	MAX	MIN	MAX	MIN	MAX	MIN	MAX	MIN	MAX
1	15	29	15	26	15	30	18	34	22	33
2	14	30	14	26	16	33	16	30	22	30
3	14	29	16	30	17	34	17	34	23	34
4	15	30	15	32	16	35	18	36	22	33
5	22	32	15	28	18	33	18	36	22	30
6	16	35	15	33	17	34	18	36	21	31
7	16	34	15	30	16	36	17	35	21	30
8	17	32	16	32	15	35	18	36	23	34
9	16	33	16	30	14	38	18	36	20	32
10	16	34	16	32	16	36	24	33	21	31
11	15	29	14	28	16	36	24	33	21	30
12	15	30	16	30	16	35	23	34	20	29
13	15	35	15	31	14	32	17	21	21	33
14	15	35	17	29	16	32	14	30	16	29
15	15	34	18	28	17	34	17	34	21	33
16	15	33	14	31	17	33	22	30	22	30
17	16	31	16	32	16	30	22	33	23	29
18	16	31	16	30	18	36	15	35	23	34
19	16	31	15	28	15	33	13	32	23	30
20	16	31	14	29	16	33	17	21	22	33
21	17	31	17	31	16	33	22	32	24	32
22	16	30	17	30	16	33	21	30	23	34
23	15	35	16	28	17	30	23	28	21	32
24	15	33	13	29	16	32	23	33	21	32
25	15	33	15	34	18	30	23	33	22	34
26	15	32	17	33	17	33	22	32	20	30
27	15	32	18	32	17	28	23	35	23	32
28	16	35	21	31	18	30	17	21	23	33
29	15	36			16	30	22	32	21	30
30	16	32			18	34	23	33	23	34
31	12	33			16	30			20	33

Average minimum temperature=21.6^{o}C Standard deviation=1.52^{o}C

Average maximum temperature=31.7^{o}C Standard deviation=1.73^{o}C

E. pH VALUES

DATE	RAW SEWAGE	F2.1	F2.2	M2.1	M2.2
12/1/89	7.2	7.6	7.9	8.0	8.0
26/1/89	6.7	7.1	7.2	7.6	7.9
14/2/89	7.1	7.2	7.4	7.3	7.4
23/2/89	6.7	7.4	7.3	7.5	7.7
10/3/89	7.1	7.2	7.3	7.5	7.5
19/3/89	7.1	7.8	7.8	7.7	7.6
29/3/89	7.2	7.4	7.4	7.5	7.6
7/4/89	7.2	7.3	7.1	7.3	7.3
27/4/89	7.4	7.1	7.00	7.00	7.10
3/5/89	7.1	7.1	7.1	7.2	7.2
16/5/89	7.1	7.1	7.2	7.4	7.8

F2 EFFLUENT STANDARD FOR DIRECT DISCHARGE TO NATURAL WATER COURSE

C.O.D.....Not to exceed 30 mg/l

Total Nitrogen exclusive NO_3..... 1 mg/l

NH_3..... 1.5 mg/l

B.O.D. (5 days at 20°C).....Not to exceed 20 mg/l

pH.....In the range 6 to 9

Temperature.....Not to exceed 25°C

Suspended Solids.....Not to exceed 30 mg/l

Total Dissolved Solids.....Not to exceed 1,500 mg/l

4 hours oxygen absorption for permanganate N./80 at 27°C not to exceed 15 mg/l

Nitrate as NO_3.....Not to exceed 45 mg/l

All other standards mentioned in Standard "A" supra viz. greases, oils, toxic materials also apply.

FI EFFLUENT STANDARD FOR ACCEPTANCE INTO NCC'S SEWERAGE SYSTEM

B.O.D (5 days at 20°C).....Not to exceed 450 mg/l

pH.....To be in the range 6 to 9.

Temperature .....Not to exceed 55°C.

Suspended Solids.....Not to exceed 300 mg/l

4 hours oxygen absorption for permanganate N./80 at 27°C.

100 mg/l

Greases: The wastes should not contain more than 100 milligrammes per litre of greases that dissolve in Ethyl-ether.

Oil, Petrol, Kerosene or other combustible materials must be removed.

TOXITY: The wastes should not include any toxi materials.

In addition the waste should not contain materials that might damage pipes or treatment works.

The flow must not exceed the capacity of the sewerage system and a meter is to be provided with a log book to record flow which can be inspected by the General Manager or Medical Officer of Health.

Quarterly tests will be carried out, at the expenses of the industry concerned, in accordance with the "Standard Methods for the Examination of Water, Sewage and Industrial Wastes", issued by the American Public Health Association.