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EFFECTS OF POWER LEVEL, ORGANIC LOADING AND TEMPERATURE ON THE PERFORMANCE OF FACULTATIVE AERATED LAGOONS

By

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ABSTRACT

Facultative aerated lagoons are aerated lagoons operated at low power levels and are wastewater stabilization ponds with artificial aeration. These lagoons are used more commonly than aerobic aerated lagoons because a satisfactory effluent can be produced with a lower power input. The power level applied to facultative aerated lagoons is sufficient only to satisfy the oxygen demand but not adequate to keep all the solids in suspension and settled solids will decompose aerobically and anaerobically. In the study reported, laboratory-model facultative aerated lagoons of 81 litres volume, aerated with diffused air, were used to study the performance of such lagoons in the treatment of municipal wastewater. Different combinations of four power levels, 0.25, 0.5, 1 and 2.0 W/m^3, three different organic loadings, 20, 33 and 62 g $BOD_5/m^3.d$, and two temperature levels, 20°C and 30°C, were applied in twenty four experimental runs. Influent and effluent were sampled on a regular basis and their characteristics were determined.

The effluent from the model facultative aerated lagoons was always of reasonable quality, with respect to BOD_5, COD and suspended solids. This was achieved with no provision for effluent settling or additional treatment. Removals of 91 percent BOD_5 and 67 percent COD could be achieved for unfiltered samples. Effluent BOD_5 of 13 mg/l in the filtered samples and 31 mg/l in the unfiltered samples was attainable in these lagoons. Effluent suspended solids levels as low as 41 mg/l were also obtained. Thus facultative aerated lagoons will provide both biological and physical treatment operations in a single earthen tank. Because suspended solids in the effluent from facultative aerated lagoons are low, no sludge disposal or processing is needed on a continuous basis. Other performance criteria; nitrogen, phosphorus,

chlorophyll "a", *Escherichia. coli* and *faecal streptococci*, are reported on in the thesis.

It was observed that the level of power introduced into the facultative aerated lagoon had positive and significant effects on some performance parameters, including BOD_5 and COD filtered removal rate coefficients, removal efficiencies for BOD_5 and COD (except for COD removal in facultative aerated lagoons operated at high temperature, 30°C) and effluent suspended solids and negative and significant effects for others, such as suspended solids removal efficiency. The effect of power was insignificant for other parameters, especially BOD_5 and COD unfiltered removal rate coefficients. The effect of power level on filtered removal efficiencies was higher than on unfiltered ones.

BOD_5 and COD removal efficiencies were negatively affected by organic loading (or positively by retention time) and the effect was found to be significant. The organic loading effect was significant and positive on filtered BOD_5 and both filtered and unfiltered COD removal rate coefficients whereas it was negative on unfiltered BOD_5 removal rate coefficients.

Temperature had significant and positive effects on some parameters, including removal rate coefficients both filtered BOD_5 and COD as well as unfiltered BOD_5 and BOD_5 and COD removal efficiencies, and insignificant effects on others, such as unfiltered COD removal rate coefficient. The effect of temperature on the removal rate coefficients, except the unfiltered COD removal rate coefficient, was higher at higher organic loadings (shorter retention times) whereas its effect on BOD_5 and COD removals efficiencies was higher at lower power levels. The temperature correction coefficient for BOD_5 at low power levels was higher than at higher levels of power.

The effect of power level on the temperature correction coefficient was significant whereas the organic loading (or retention time) effect was insignificant.

Relationships between the individual operating parameters and performance parameters are presented in the form of empirical equations and the combined effects of these operating parameters and performance parameters were also modelled. High organic loading (short retention time) in facultative aerated lagoons operated at low power levels proved to remove more organic material per day per unit of power introduced into the lagoon. Therefore, the optimum conditions of organic loading and power level at which a single facultative aerated lagoon used as a sole treatment process for treating settled sewage should be operated are 0.25 W/m^3 for power level and 62 g BOD_5 $/m^3.d$ for organic loading (3 to 4 days retentiom time). The effect of mixing, represented by the parameters in a simulation model, on performance was also modelled in the form of empirical equations.

Nitrogen, phosphorus, *Escherichia coli* and *faecal streptococci* removals were considerable. Variations of their removal performance as well as algal concentration variations with the operating parameters were also studied and discussed. A mathematical equation was developed for the determination of the mean solids retention time (SRT) in facultative aerated lagoons. The relationship between SRT and power level and organic loading was determined and represented by an empirical equation. A power level of around 0.5 W/m^3 was the threshold for settleable solids suspension below which no significant decrease in effluent or mixed liquor suspended solids concentration would take place.

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SYMBOLS AND ABBREVIATIONS

A	surface area of the lagoon
C	constant or concentration of the tracer in the effluent
°C	degrees centigrade
f	proportionality factor
d	day or dispersion number
F/M	food-to-microorganism ratio
g	gram
ha	hectare
K_1	removal rate coefficient at T_1 temperature,
K_2	removal rate coefficient at T_2 temperature
K	overall removal rate coefficient
k	specific removal rate coefficient
K_{20}	removal rate coefficient at 20°C
K_{30}	removal rate coefficient at 30°C
K_d	specific decay rate, and called auto-oxidation rate coefficient or endogenous respiration rate
K_f	filtered removal rate coefficient
kg	kilogram
K_La	over-all oxygen transfer coefficient
K_s	standard removal coefficient
K_u	unfiltered removal rate coefficient
l	litre
L	organic loading
L_V	volumetric organic loading
m	meter
mg	milligram
N	number of tanks
P or P_V	power level
pH	the negative log of the concentration of hydrogen ion
Q	wastewater flow rate.
S	organic matter concentration
S_0	substrate, both soluble and insoluble in the influent
S_1	soluble substrate in the effluent
T	temperature

t	time or hydraulic retention time
T_a	air temperature
T_i	influent wastewater temperature
T_o	effluent wastewater temperature
T_w	liquid temperature in the lagoon
V	volume of reactor or lagoon
X	aeration volatile suspended solids
Y	growth yield, and called sludge yield coefficient; mg/l VSS produced per mg/l BOD stabilized

Greek Symbols

σ	variance of C curve
μ	microgram
β	feedback factor
θ	temperature correction coefficient.
β	feedback coefficient
θ	temperature correction coefficient
θc	mean cell residence time.

Abbreviations

BOD	biochemical oxygen demand
BOD_5	5-days biochemical oxygen demand
BOD_u	ultimate BOD
COD	chemical oxygen demand
EAD	Exit Age Distribution
FPR	filtered percentage removal
IAD	Internal Age Distribution
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
NCASI	National Council for Air and Stream Improvement
OTE	oxygen transfer efficiency
RTD	residence times distribution
SS	suspended solids

TKN	total Kjeldahl nitrogen
TS	total solids
TSS	total suspended solids
TVS	total volatile solids
TVSS	total volatile suspended solids
UPR	unfiltered percentage removal
VSS	volatile suspended solids

INTRODUCTION

It must be appreciated that the purpose of water pollution abatement by removing pollutants from used water is public health protection as well as the preservation and enhancement of water bodies and of the environment in general. The treatment of wastewater is, therefore, a means to satisfy the above requirements. So, wastewater treatment is an attempt to produce a stable effluent similar in composition to water existing naturally or of acceptable quality for discharge to the receiving water.

Wastewater treatment practice today relies heavily on high-maintenance, mechanically-complex systems. Such systems may be optimal for the treatment of wastewaters with medium to high discharge rates and those with special treatment requirements. For environmentalists and economists, the presence of various treatment processes has raised the issue of the selection of suitable, available, efficient and economical processes for particular areas and wastewater types. However, for wastewaters with relatively low discharges and moderate concentrations of biodegradable organics, such high-maintenance, mechanically-complex systems may prove to be less appropriate. The cost of these systems, both capital and operational costs, and the requirement for highly skilled operators may limit their feasibility for certain applications in many countries of the world. There are applications for which reliable, low-maintenance, mechanically-simple wastewater treatment systems are more appropriate. This, inevitably, leads to the possibility of adoption of wastewater treatment systems which are more land-intensive.

The need for such simpler treatment systems for domestic wastewater and industrial wastewaters has sometimes resulted in environmental engineers using aerated lagoons. Such systems, when properly designed, can meet the above mentioned needs. Aerated lagoons are a natural progression from wastewater stabilization lagoons, being pond systems which are provided with aeration facilities. They were used by the Chinese as long ago as a thousand years and have been in existence in North America for more than nine decades (Narasiah *et al.*, 1987 quoted from Allum and Carl, 1970).

Due to their relatively simple configuration, flexibility of operation and high treatment potential, aerated lagoons have been gaining widespread popularity in many parts of the world, particularly in warm climates, for the treatment of municipal wastewater and certain biodegradable industrial wastewaters. Essentially they are once-through aerated sludge systems with low mixed liquor solids levels and without sludge recycle.

The excessive power requirements of aerobic aerated lagoons has made this form of the treatment process relatively uneconomical and less popular. As an alternative, facultative aerated lagoons are more commonly used than aerobic aerated lagoons because a satisfactory effluent can be produced with lower power input. These facultative aerated lagoons are designed with power levels sufficient only to maintain dissolved oxygen throughout the liquid volume. Turbulence levels reached under these conditions are insufficient to maintain all the solids in suspension, and solids which settle out decompose anaerobically.

The facultative aerated lagoon is a partially suspending aerated lagoon, operating at higher retention times and lower loadings. This in turn provides for the stabilization of the organic matter, which is aerobically and anaerobically incorporated into the cellular mass, resulting in lower suspended solids concentration and hence lower sludge production and disposal.

The literature reveals some concern regarding aerated lagoons as a method of wastewater treatment. However, it appears there is not enough understanding or documentation of facultative aerated lagoons, their performance or the treatment mechanisms predominating under different operating conditions to enable optimum use to be made of this treatment method.

Nevertheless, the facultative aerated lagoon process lends itself well to the laboratory-scale study of the effects of various controllable factors on the performance of this treatment method, and conditions that could improve its use economically and efficiently.

Literature Review

2.1 Wastewater Treatment

The removal of wastewater pollutants', such as soluble and insoluble organics, various forms of nitrogen and phosphorus, and inert insoluble materials (Benefield and Randall, 1980) as well as other impurities is the objective of the use of wastewater treatment. These pollutants can be removed effectively by biological, chemical or physical treatment methods. Biological treatment depends on the activity of the microorganisms utilized for the removal of the wastewater pollutants. A mixed culture of biomass, suspended or attached to a fixed surface, with one or more groups of microorganisms predominating, depending upon the major organic comonent of the wastewater (Kantawala, 1986). Thus, providing a favourable environment for the living biomass is necessary for good performance. Good contact between the mixed culture of microorganisms and the organic matter to be removed biologically is essential. This organic matter is used as a food source, or substrate, for reproduction, synthesis, and energy.

Biological treatment methods are either aerobic, anaerobic or facultative. In the first type the presence of dissolved oxygen is essential for the microorganisms to perform the removal process, whereas its presence in the anaerobic method will be an inhibitory or toxic substance for these active microorganisms. Facultative biological treatment processes are a combination of both aerobic and anaerobic methods, in which a facultative zone exists between the aerobic environment in the top layer and anaerobic at the bottom. According to the biomass, the processes may also be either those employing suspended biomass or those employing biomass attached to a fixed surface. The mode of disposal, land requirement, capital, operating and maintenance costs of alternative processes are the main factors to be considered in the choice of a treatment process (Lal and Verma, 1989).

2.2 Aerated Lagoons

2.2.1 Introduction

An aerated lagoon is a simple retention tank through which raw wastewater is allowed to flow and in which the wastewater is mixed and aerated by mechanical or diffused aeration units. It has also been described as a "step up" from the more simple on-site treatment systems (McNeill and Bradley, 1988). It is basically an aerobic biological treatment process and as such is similar to an activated sludge system (Timpany *et al.*, 1971; Eckenfelder and Adams, 1972) however, it is the most economical means of aerobic treatment when additional oxygen, over the natural atmospheric aeration, is required (Bishop, 1975). It is popular in various parts of the world, especially in warm climates and developing countries as well as in semirural and rural communities (Thirumurthi, 1979) where inexpensive land is available and serves as pretreatment, a complete treatment method or as one unit process in a treatment train. Besides its use for the treatment of domestic wastewaters it is also used to treat certain biodegradable industrial wastes. When properly designed, aerated lagoons can meet the needs for treatment in suitable cases (Rich, 1982).

With the increase in population densities and the need for land for other uses or rising land costs, as well as ecological problems (Balash and Sperber, 1975) and the inadequacy of water resources in some countries, wastewater stabilization ponds can no longer be considered an adequate treatment process in most cases. Also in some situations existing stabilization ponds are overloaded, create nuisance problems (e.g., odours) and no longer work properly. Expansion is not always possible at the time when more stringent effluent quality standards need to be implemented. Therefore, the need for a treatment system which can hold a large wastewater volume with a smaller surface area and the relief of organically overloaded wastewater stabilization ponds by employing supplemental aeration lies behind the innovation of aerated lagoons. Larger depth and shorter time for pollutants' elimination are used in aerated lagoons compared with wastewater stabilization ponds; this means that more wastewater can be treated in a limited space. By contrast, in situations where conventional technologies are too expensive or too energy-intensive, or where technical expertise is not locally available, aerated lagoons are particularly well suited (Kouzell-Katsiri, 1987). Bearing the above points in mind, it can be said that aerated lagoons are usually preferred to wastewater stabilization ponds in densely populated areas where land is relatively expensive (Wong, 1990). Also it can be said that aerated lagoons stand between high-cost, energy-intensive conventional wastewater treatment processes and land-intensive stabilization pond (Easson *et al.* , 1988).

The artificial aeration in aerated lagoons replaces the natural oxygenation process (surface and photosynthetic aeration) in oxidation ponds, thus providing greater oxygenation rates and allowing greater organic loadings. Therefore, they are a simple and vital modification of stabilization ponds (Murphy and Wilson, 1974; Schroeder, 1977; Metcalf and Eddy, 1979; Gohil *et al.*, 1988), as well as a simple form of the activated sludge process in which sludge recycle is not employed (except in some special type of aerated lagoons) (Mara, 1976; Metcalf and Eddy, 1979; Lal and Verma, 1989). In the past, the use of aerators was almost exclusively to alleviate existing nuisance problems in heavily loaded stabilization ponds and to improve effluent qualities (Bartsch and Randall, 1971). In the spectrum of treatment processes, aerated lagoons fall between facultative stabilization ponds and the high rate activated sludge process (Fletcher, 1980; Gray, 1989). Also, the aerobic aerated lagoon was described by analogy as an activated sludge process with extended aeration, no sludge return and lower levels of power input (Bartsch and Randall, 1971). However, the aerobic lagoon is more sensitive to temperature than activated sludge because of the lack of sludge recycle.

The aerated lagoon approach to biological waste treatment provides a controlled, low level biooxidation environment in which organic matter is contacted with a biologically active mass (NCASI, 1971). The contact time, or solids retention time, is, on a once-through basis, theoretically equal to the hydraulic retention time if the lagoon is completely mixed. Solids recirculation is not usually practised with aerated lagoons because maintaining a set level of biological solids is not a design requisite. The basic difference between aerated lagoons and activated sludge processes is the degree of environmental control applied to them, whereas the microbiology in aerated lagoons is the same as that in activated sludge processes (Metcalf and Eddy, 1979).

Aerated lagoons are sometimes referred to in the literature as aeration lagoons (Gray, 1989), aerated stabilization basins (Eckenfelder and O'Connor, 1961; Sackellares, 1985; Gay, 1988; Hall and Randle, 1994), aerated stabilization ponds, aerated ponds (Griffith, 1968; Metcalf and Eddy, 1979), aerated ponds (Griffith, 1968; Metcalf and Eddy, 1991) and surface aerated basins (Soper *et al.*, 1975).

Removal of wastewater pollutants is the main objective in using an aerated lagoon. However, it offers a cost-effective means of flow equalization and BOD peak load reduction of industrial wastes, most of which are generated on a batch basis.

In addition to the treatment of domestic wastewater, these lagoons have been used for treatment of various wastes. For example, it was reported in the literature that they have been used for the treatment of effluents from felt roofing material plants (Loehr and McKinney, 1966), duck farms (Loehr and Schulte, 1970), textile-finishing plants (Bartsch and Randall, 1971), pulp and paper mills (Timpany, 1971; Stuthridge *et al.*, 1991), slaughterhouses (Heddle, 1982; Vallée *et al.*, 1989), forest products industries (Sackellares, 1985), leachate (Maris *et al.*, 1984; Robinson and Grantham, 1988), swine manure (Schulz and Barnes, 1990), dairy production (Pickett, 1988; Lal and Verma, 1989), laundry wastes (Mann, 1970), and the potato processing industry (Rusten *et al.*, 1991). It was reported that an aerated lagoon removes from one-third to one-half of organic halide compounds present in Kraft mill wastewater (Amy *et al.*, 1988).

2.2.2 Advantages of Aerated Lagoons

Aerated lagoons have several advantages over other processes, which make such methods more uniquely valuable than other treatment methods. The main advantages, as compiled from the literature (Mancini and Barnhart, 1968; Timpany *et al.*, 1971; Barnhart, 1972; Rich and White, 1977; Benefield and Randall, 1980; Golueke and Diaz, 1989), that all types of aerated lagoons have are:

1. Construction features are simple;
2. Comparatively low in both requirements and costs of construction, operation and maintenance;
3. Require much less skilled manpower than conventional treatment methods such as the activated sludge process;

4. High effluent quality obtained if solids separation is provided;
5. Relatively stable biological solids at low loading operations;
6. Potential for resistance to shock toxic load;
7. Ability to treat high strength wastes;
8. Buffering capacity when pH is a problem;
9. Permits a considerable amount of flexibility in design;
10. The relative ease with which existing lagoons' volume or aeration capacity or both can be added to as populations increase or as better efficiency is desired;
11. Relatively easily relocated (aerators only) to another site if necessary;
12. Wastewater equalization potential;
13. High capacity for heat dissipation when high temperature wastes are to be treated;
14. Minimal sludge production.

In addition to the above, aerated lagoons are able to respond rapidly to changes in load, provided that sufficient oxygen is available to maintain aerobic conditions, unlike the activated sludge process which is sensitive to shock loading (Fletcher, 1980). This is justified on the basis that aerated lagoons operate with a low sludge age and do not rely on a high level of return sludge to provide microorganisms. This is considered a useful attribute of a lagoon system which adds to its value as a pretreatment or a treatment method.

The relative cost effectiveness (McNeill and Bradley, 1988) has made this treatment method preferable, conditions permitting, compared with other activated sludge treatment systems. Capital cost of aerated lagoons is approximately two-third that for activated sludge (Eckenfelder, 1961). It can be said, therefore, that the nature of aerated lagoons' design is why this system can be considered a low cost alternative technology. From the list above, it is clear that aerated lagoons have some advantages over the activated sludge process. It is worth mentioning that aerated lagoons remove organo-Cl compounds concurrently with soluble oxygen demand. This occurs under anaerobic conditions within the benthal zone (Bryant *et al.*, 1987; Bryant *et al.*, 1988), whereas, it was reported that most of the soluble adsorbable organic halogen occurs in the aerobic portion of the lagoon (Collins and Allen, 1991).

2.2.3 Disadvantages of Aerated Lagoons

Just as aerated lagoons have advantages, like all treatment processes, they also have disadvantages. These include:

1. Larger land area requirement than conventional activated sludge systems;
2. Difficulty of process modification;
3. High effluent suspended solids concentration if no final clarification (Barnhart, 1972);
4. Sensitivity of process efficiency to variations in ambient air temperature (Bartsch and Randall, 1971).
5. Low effluent quality without solids separation;
6. Solids separation at high loadings may prove a significant problem (Mancini and Barnhart, 1968);
7. Complete recovery after toxicity upset requires extended operating periods;
8. Severe decrease in efficiency results from large inputs of biodegradable or toxic waste (Gray, 1989).

As a comparison of various treatment methods, Table 2.1 lists important criteria that can be used to describe the characteristics and efficiencies of removal of pollutants and pathogens of any wastewater treatment process, as well as their advantages and disadvantages. Thus, according to this table it will be useful to correctly rank this process among alternative treatment processes for the conditions prevailing in any given situation.

2.2.4 Construction Features of Aerated lagoons

The materials used in the building of aerated lagoons are mainly those parent materials at the site where the lagoon is excavated and thus low cost and fast construction are usual. Often these lagoons are lined with an impermeable plastic membrane. Disparate depths have been used, from 1.2 m to as high as 5.0 m (Bartsch and Randall, 1971; Lal and Verma, 1989; Horan, 1990). Even higher depths, 6.1 (Dawson and Grainge, 1969; Gunning, 1979) and 6.7 m (Murphy and Wilson, 1974) have been reported. Deep lagoons are used to ensure adequate oxygen transfer and to allow a sedimenntation zone at the base of the lagoon for anaerobic decomposition (Eckenfelder, 1982). Various sizes of lagoons have been utilized, depending on the flow rate of the wastewater to be treated and the degree of treatment required. Varying retention times are employed in aerated lagoons, ranging from less than two days to more than thirty days (Dawson and Grainge, 1969; Malina *et al.*, 1971a; Murphy and Wilson, 1974). High retention times (100 days) in temperate climates have also been cited (Horan, 1990). Schulz and Barnes (1990) reported that in deep treatment lagoons (5.5 m deep) with shallow aeration, producing a stratified lagoon, 75 percent removal of organic material was achieved when treating piggery wastewaters, without the generation of nuisance odours, and using only one-third of the power required for fully aerobic treatment.

Table 2.1 Comparison of Different Wastewater Treatment Processes (After Arthur, 1983)

	Package Plant	Activated Sludge Plant	Trickling Filter	Extended Aeration Plant	Oxidation Ditch	Aerated Lagoon System	Waste Stabilization Pond System (including anaerobic units)	Waste Stabilization Pond System (excluding anaerobic units)
Criteria								
BOD_5 Removal	++	++	++	++	+++	+++	+++	+++
FC Removal	+	+	+	++	++	+++	+++	+++
SS Removal	++	+++	+++	+++	+++	++	++	++
Helminth Removal	+	++	+	+	++	++	+++	+++
Virus Removal	+	++	+	++	++	+++	+++	+++
Ancillary Use Possibilities	+	+	+	+	+	+++	+++	+++
Effluent Reuse Possibilities	+[a]	+[a]	+[a]	++	++	+++	+++	+++
Simple and Cheap Construction	+	+	+	+	++	++	+++	+++
Simple Operation	+	+	++	+	++	+	+++	+++
Land Requirement	+++	+++	+++	+++	+++	++	++	+
Maintenance Costs	+	+	++	+	+	+	+++	+++
Energy Demand	+	+	++	+	+	+	+++	+++
Minimization of sludge for removal	+	++[b]	++[b]	++[b]	+	++	+++	+++

Key +++ good ++ fair + poor

a The effluents from activated sludge, trickling filter and package plants frequently have high ammonia levels (> 5 mg/l) and fecal bacterial concentrations, and are usually not suitable for irrigation or fish farming without tertiary treatment.

b Assume provision of sludge digesters.

The choice of depth, as stated by Arceivala (1981), depends on the following factors:

1. Type of the soil and construction problems likely,
2. Type of aeration equipment used,
3. The minimum depth above the anaerobic layer needed to oxidize any gases produced in the anaerobic layer (for facultative aerated lagoons only).

The use of artificial aeration processes in aerated lagoons reduces the land area required, comparative to stabilization ponds. Because of the larger depth used and lower retention time needed, than for natural algal ponds, the land area required for an aerated lagoon is about 10 percent to 20 percent of that required for facultative wastewater stabilization ponds.

In some designs, aerated lagoons are preceded by flow measurement (Griffith, 1968), grease removal (McNeill and Bradley, 1988), fine screening or comminution (Feachem *et al.*, 1977) and followed by secondary and tertiary stabilization ponds or surface settling units (Griffith, 1968), to comply with certain discharge standards. Grit removal is usually advisable, whereas primary sedimentation is not required (Feachem *et al.*, 1977).

Aeration equipment types usually used in aerated lagoons are mechanical surface aerators, perforated tubing diffused air and aeration gun diffused air (Townshend *et al.*,1969). However, the general classification of these aerators is mechanical or diffused aeration systems. Mechanical surface aerators are installed on floats or on a permanent base to aerate the liquid contents. The diffused air aeration system is most successful in maintaining aerobic conditions in areas where the surface of the pond is frozen (Gray, 1989) and the cooling effect will be less than with surface aerators (Argaman and Adams, 1977). Therfore in very cold areas, during winter months, compressed air aeration is preferable to mechanical aeration (Dawson and Grainge, 1969), however, mechanical aeration systems afford a more economical initial installation and a reduced maintenance cost (Eckenfelder, 1961).

2.2.5 Types of Aerated Lagoon

Depending on power input per unit lagoon volume (power level), and the provision or otherwise of recirculation arrangements, the solids in the system will either flow through, build up or settle. Thus, based on the way solids are handled, three types of aerated lagoons are distinguished:

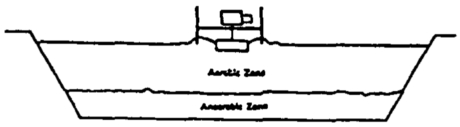
i) Aerobic flow through;

ii) Aerobic with solids recycling;

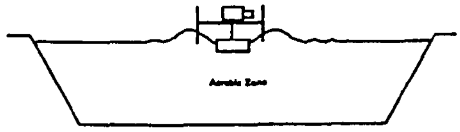
iii) Facultative.

Most authors (e.g., Eckenfelder, 1970; Bartsch and Randall, 1971; Kormanik, 1972; Rich, 1977; Thirumurthi, 1979 and 1980; Benefield and Randall, 1980; Golueke and Diaz, 1989) refer to (i) and (ii) as the most common types of aerated lagoons and do not mention (iii). However, other authors (Metcalf and Eddy, 1979; Arceivala, 1981; Metcalf and Eddy, 1991) include aerated lagoons with solids recycling as the third member of the aerated lagoons family. The common types of aerated lagoons are shown in Figure 2.1.

The different ways of handling the solids have a substantial effect on efficiency, power requirement, retention time, etc., and design methods must take these differences into account, although the basic principles of biological treatment apply equally well to all three types.



Facultative Aerated Lagoon



Aerobic Aerated Lagoon

Figure 2.1 Types of Aerated Lagoons

2.2.5.1 Aerobic Flow-Through Lagoons

Aerobic flow-through lagoons are also known as aerobic aerated lagoons, completely-mixed aerated lagoons or completely-mixed aerobic aerated lagoons. It is noted that if the aerated lagoon term is used this usually means an aerobic aerated lagoon. The power level in this type of aerated lagoon is high enough not only to diffuse sufficient oxygen into the liquid but also to keep all solids in suspension, in which case no settlement of solids occurs.

Completely-mixed aerated lagoons are extremely effective in removing organic substrate at high loadings and converting it to cellular mass. It was reported by Robinson and Grantham (1988) that more than 99.5% BOD_5 removal of leachate of strength higher than 10,000 mg BOD_5/l (flow weighted mean BOD_5 of 3700 mg/l) was obtained when treated in a 10-day aerated lagoon and the effluent BOD_5 rarely exceeded 50 mg/l. However, Fletcher (1980) concluded that effluent quality from a completely-mixed aerated lagoon treating domestic wastewater operated with two days retention time was not good enough for the receiving water standards, since the average BOD_5 and SS at two days retention were 116 and 130 mg/l, respectively; even when retention time was 5.5 days the corresponding figures were 37 and 53 mg/l, respectively. It should be pointed out that without sludge recycle, organic matter stabilization is not significant with short retention times.

i) Organic Matter Removal in Aerobic Aerated Lagoons

For design purposes, it is usually assumed that an aerated lagoon behaves like a completely-mixed reactor (Murphy and Wilson, 1974). The overall substrate removal rate, K, can be determined either from laboratory studies or from field measurements of substrate removal under known conditions.

The removal of organic matter in completely-mixed aerated lagoons is generally considered by environmental engineers to be a first order reaction within continuously stirred tank reactors therefore, removal rate is proportional to the concentration remaining:

$$\frac{dS}{dt} = -KS \qquad (2.1)$$

where; K = overall removal rate coefficient,

S = organic matter concentration, and

t = time.

A material balance around the lagoon yields the following:

$$V(dS/dt)_{net} = QS_0 - QS_1 + V(dS/dt)_{reaction} \qquad (2.2)$$

where; V = lagoon volume,

Q = wastewater flow rate.

At steady state $[(dS/dt)_{net} = 0]$, Equation 2.1 can be rearranged to yield

$$\frac{S_1}{S_0} = \frac{1}{1 + Kt} \tag{2.3}$$

where; S_1 = soluble substrate in the effluent; mg/l,

S_0 = substrate, both soluble and insoluble in the influent; mg/l, and

t = hydraulic retention time (V/Q); day.

In some of the literature, effluent total BOD_5 is reported to have been used as the value of S_1 in the above equation, whereas S_0 usually represents the unfiltered BOD_5 (Narasiah *et al.*, 1987). Removal rate coefficients for domestic wastewater are expected to be larger than those for a mixture of domestic and industrial wastewaters (Bennet, undated).

The overall removal rate coefficient has been related to the aeration volatile suspended solids (Eckenfelder, 1970) and Equation 2.3 can be expressed as:

$$\frac{S_1}{S_0} = \frac{1}{1 + kXt} \tag{2.4}$$

where, X = aeration volatile suspended solids; mg/l, and

k = specific substrate removal rate coefficient; l/mg.day.

Variable values have been reported for the overall removal rate coefficient, K, for completely suspended aerated lagoons. Table 2.2 summarizes the reported values of the overall removal rate coefficient.

Removal rate coefficients are affected by temperature and biomass level in the reactor, as well as organic loading (Thirumurthi, 1979) and the influent BOD concentration (Kouzell-Katsiri, 1987). The latter is justified in that, with the increase in influent BOD_5 concentration, the proportion of easily biodegradable components in the waste also increases. This means that retention time also affects the value of the removal rate coefficient because it (retention time) represents the ratio between the influent concentration and organic loading. However, it has been

Table 2.2 BOD_5 Removal Rate Coefficients (overall and specific) Reported for Completely-Mixed Aerated Lagoons without Solids Recycling.

Coefficient value	Remarks	Reference
8.1 d^{-1}	Aerobic aerated lagoon, municipal wastewater, filtered removal rate coefficient (i.e., for filterable BOD_5)	Fleckseder and Malina (1970)
5.1 - 6.7 d^{-1} and 0.023-0.037 l/mg.d	The lagoon have a character close to fully mixed, power level of 2.7 W/m^3, full scale aerated lagoon treating domestic sewage.	Balasha and Sperber (1975)
0.031 l/mg.d 6.7 d^{-1}	Specific, at 20°C, the lagoon is close to fully fixed aerobic conditions, power level in the lagoon is 2.7 W/m^3 Overall, at 20°C, the lagoon is close to fully fixed aerobic conditions, power level in the lagoon is 2.7 W/m^3	Balasha and Sperber (1975)
0.42 - 0.51 d^{-1}	Overall, at 20°C, for BOD_5, mixture of domestic wastewater (90%) and synthetic sewage (10%),	Thirumurthi (1979)
0.01-0.03 l/mg.d 1 - 1.5 d^{-1}	Specific, at 20°C, for BOD_5, municipal wastewater Overall, at 20°C, domestic waste-water, filtered samples	Arceivala (1981)
2.7 to 4.8 d^{-1} 0.4 to 0.5 d^{-1}	Overall, at 20°C, for BOD_5 , first lagoon in series of two. Overall, at 20°C, for BOD_5 , second lagoon in series of two	Kouzell-Katsiri (1987)

observed that the effect of retention time on BOD_5 removal is less than the effect of influent BOD_5 concentration (Kouzell-Katsiri, 1987).

Kouzell-Katsiri (1987) describe the effect of influent BOD_5 concentration, S_0, on the removal rate coefficient according to the following equation:

$$K_{20} = 0.0556\,(S_0)^{0.8} \tag{2.5}$$

The effect of organic loading on the removal rate coefficient in completely mixed aerated lagoons was studied by Thirumurthi (1979). He described the retardant effect of organic loading in two ways:

a) If first-order removal rate coefficient is assumed in the design then a standard removal coefficient, K_s, of proposed magnitude 0.5 d^{-1} is to be used in the following equation:

$$\log 20 - \log L_l = 12\,(K_s - K_{20}) \tag{2.6}$$

where, K_{20} = removal rate coefficient at 20°C, and

L_l = organic loading; g BOD_5 /m^3.d.

The value of Ks he proposed was at 20°C liquid temperature, organic loading of 20 g BOD_5 /m^3.d and retention time of approximately 10 days.

b) The retardant effect of the organic loading is quantified by assuming an *n*th-order BOD removal rate coefficient. He suggested the following design criteria:

$$\frac{S_0}{S_1} = 1 + KtS_1^{\,0.1} \tag{2.7}$$

He obtained a value of 0.38 d^{-1} for K in the above equation. Kouzell-Katsiri (1987) pointed out that the rate of BOD_5 removal in completely mixed aerated lagoons is first order provided the rate coefficient is corrected for temperature and input BOD_5 concentration.

ii) Solids Concentration in Aerobic Aerated Lagoons

The concentration of biomass in a completely suspended aerated lagoon can be predicted using a relationship developed from a steady-state mass balance across the lagoon (Balasha and Sperber, 1975; Tikhe, 1975; White and Rich, 1976b; Arceivala, 1981):

$$X_1 = \frac{Y(S_0 - S_1)}{1 + K_d(V/Q)} \tag{2.8}$$

where, Y = growth yield, and called sludge yield coefficient; mg/l VSS produced per mg/l BOD stabilized, and

K_d = specific decay rate, and called auto-oxidation rate coefficient or endogenous respiration rate; d^{-1}.

Coefficient Y represents the accumulation of volatile suspended solids in the aerated system and expresses the synthesis of biological solids (Eckenfelder, 1967). The value of Y is characteristic of the nature of the wastewater. For domestic wastewater the reported values are between 0.5 and 0.8. The concentration of aeration volatile solids can be approximated to be one-half the concentration of BOD_5 removed (Eckenfelder and Adams, 1972). Coefficient K_d expresses the cellular auto-oxidation rate and its values cited in the literature vary between 0.05 and 0.08 d^{-1}. The suspended solids present in the lagoon consist of inert solids, which are organic and inorganic, in the influent and in the biomass created.

The greater part of BOD_5 in aerated lagoon effluent is in the form of suspended solids or biomass (McKinney and Edde, 1961; Malina *et al.*, 1971a and b; Balasha and Sperber, 1975; Benefield and Randall, 1980; Lewandowski and Bradley, 1981; Rich, 1982c), and total BOD_5 is a function of the soluble BOD_5 and total suspended solids concentration in the effluent. Therefore, a relationship between total BOD_5 in the mixed liquor and the concentration of suspended solids (total or volatile) or the active biomass is given by the equation:

$$\text{Effluent total BOD}_5 = (\text{effluent soluble BOD}_5) + C\,(\text{effluent SS}) \qquad (2.9)$$

where, C = constant, and

SS = suspended solids.

Different constant values in the above equation have been reported for aerated lagoons, as summarized in Table 2.3. These constants represent BOD_5 resulting from the suspended solids.

Combining Equations 2.4 and 2.8 results in the following equation:

$$S_1 = \frac{1 + K_d t}{Ykt} \qquad (2.10)$$

As can be seen from Equation 2.10, the final effluent quality is not a function of the influent organic matter concentration. An increase in influent concentration (within a small range) will result in an increase in microbial solids concentration and, hence, an increase in the overall removal rate coefficient and a corresponding decrease in effluent organic matter concentration (Arceivala, 1981).

Table 2.3 BOD_5 Associated with Effluent Suspended Solids (Total or Volatile) Reported for Aerated Lagoons.

Coefficient value	Remarks	Reference
0.61 0.24	SS, aerobic aerated lagoon SS, aerobic-anaerobic aerated lagoon	Eckenfelder (1967)
0.84	Of the active biomass	Eckenfelder *et al.* (1972)
0.5-0.57	VSS, lagoon is operated at 2.7 W/m^3 the lagoon is close to fully fixed aerobic conditions	Balasha and Sperber (1975)
0.85	Of biomass fraction in the VSS	Rich (1978)
0.5	VSS	Arceivala (1981)
0.3	Of SS, in winter, lagoon system consisting of settling basin, completely mixed aerated lagoons, facultative aerated lagoon and another settling basin connected in series, treating Kraft pulp and paper mill effluent	Cook and Chandrasekaran (1986)
0.84		Kouzell-Katsiri (1987)
0.22 - 0.43	SS, aerated lagoon operated at 3.5 W/m^3	Narasiah *et al.* (1987)
0.36 - 0.54	Of VSS, partially mixed aerated lagoon following a completely mixed lagoon.	Kouzell-Katsiri (1987)
0.4	Total suspended solids	Rich (1989)

BOD removal efficiency in aerobic aerated lagoons is not very high (about 50 to 60%) mainly because of the solids level in the effluent. Therefore, additional treatment is necessary if better BOD and solids removal is desired. These lagoons are generally followed by facultative stabilization ponds or facultative aerated lagoons, or are designed with the intention of converting them eventually to facultative aerobic lagoons with solids recycling. The power requirement of aerobic lagoons is somewhat high when seen in relation to the BOD removal.

Malina *et al.*, (1971a) asserted that the degree of mixing in aerated lagoons has the greatest influence on overall performance. Gohil *et al.* (1988) reported that the efficiency of an aerated lagoon is a function of speed and depth of the rotor through which diffusion of dissolved oxygen is accomplished by mechanical means. A sharp decrease has been observed in the effluent suspended solids, which also means a decrease in insoluble BOD_5, as power level decreases (Malina *et al.*, 1971a).

Two days retention time and a volumetric organic loading of 116 g $BOD_5/m^3.d$ was sufficient to remove 38 percent of unfiltered BOD_5, as was observed by Fletcher (1980) when treating domestic wastewater in a completely-mixed aerated lagoon aerated with diffused air at a rate of 3.6 $l/min.m^3$. With increase in the air flow rate to 7.3 $l/min.m^3$ of lagoon volume he found that, at the same retention time and organic loading, 50 percent was removed. With two hours subsequent settling, the above figures increased to 55% and 68%, respectively. However, retention times of 4 days (57.5 g BOD_5 $/m^3.d$ organic loading) and 5.5 days (41.8 g BOD_5 $/m^3.d$ organic loading) achieved 60% and 84%, respectively, before settling and 84% and 88% following two hours settling. In an aerobic aerated lagoon operated at 5 W/m^3 power level treating wastewater from a shopping centre and domestic sewage, with a 24-hr average BOD_5 of the influent of 240 mg/l, McKinney and Edde (1961) observed that a retention time of 6.4 days gave an effluent of 58 mg BOD_5/l, which represents 76 percent reduction. Four days retention time in an aerated lagoon can readily achieve 85% to 90% BOD removal (Dhallah, 1971, cited by Feachem *et al.*, 1977).

The concentration of suspended solids in the effluent may be the same as that in the lagoon, or somewhat less if the outlet is baffled. About 50-80% of these solids may be assumed to be volatile (BOD_5 corresponding to the volatile solids may be assumed to be about 0.5 mg/mg VSS in the effluent).

Arceivala (1976) postulated that the K values derived from observed data using models which do not take into account the actual dispersion conditions in the reactor are specific for the reactor studied and can neither be used to compare different configurations nor to make full-scale designs in which different dispersion conditions may prevail.

The effect of temperature on the performance of aerobic aerated lagoons is more than that in activated sludge processes, mainly because of the low solids levels in the system. A decrease in temperature from 20°C to 10°C caused the BOD_5 removal efficiency to decrease from 85 percent to 65 percent in an aerobic aerated lagoon treating domestic wastewater at a retention time of 5 days (Gray, 1989).

For a retention time of 5 days or greater, Kouzell-Katsiri (1987) observed that almost complete nitrification was achieved in completely-mixed aerated lagoons treating domestic wastewaters at temperatures from 17° to 20°C whereas, Middlebrooks and Pano (1983) concluded that a minimum theoretical hydraulic retention time of approximately 45 days appears to be required to accomplish any ammonia nitrogen or total Kjeldahl nitrogen (TKN) removal in aerated lagoons with diffused air aeration. Oleszkiewicz (1986b) observed that the decrease in

TKN concentration in aerated lagoons is much larger for highly loaded lagoons than in relatively low loaded lagoons.

2.2.5.2 Aerobic Lagoons with Solids Recycling

Aerobic lagoons with solids recycling are sometimes called extended aeration aerated lagoons. If an extended aeration treatment design is to be used, completely-mixed aerated lagoons, in practice, are usually utilized as an intermediate biological treatment process to which solids are recycled from the clarifier (Kormanik, 1972). This type of aerated lagoon is favoured for industrial or suburban developments where land is expensive or where better quality effluent is desired. Larger municipalities may also wish to consider them up to a certain population size, beyond which conventional activated sludge process may prove less costly.

Aerobic aerated lagoons with solids recycling are not commonly used because of the requirements for solids settling and recycling. However, they give the best quality effluent (BOD removal can be as high as 95%) and they have an ability for nitrogen removal through nitrification and denitrifiction. Their consistency of performance over varying temperatures and the requirement for much less land than the other two types of aerated lagoon are two of its main advantages. Power required for this treatment process is higher than either aerobic flow-through aerated lagoons or facultative aerated lagoons. They are similar to activated sludge or extended aeration plants. The power input level is sufficient to meet oxygen requirements and keep all solids in suspension. The solids concentration in the lagoon is quite high, since the system is designed to prevent solids from escaping with the effluent by the incorporation of some form of solids settling and recycling.

i) Organic Matter Removal and Solids Concentration in Aerobic Lagoons with Solids Recycling

The equations used to determine the organic matter removal in this type of aerated lagoons are similar to those used for completely-mixed aerated lagoons without solids recycling (Arceivala, 1981). However, microbial solids present in the lagoon at equilibrium conditions would be determined according to the following equation:

$$X_1 = \frac{Y(S_0 - S_1)}{1 + K_d\theta_c} \tag{2.11}$$

where, θ_c = mean cell residence time.

2.2.5.3 Facultative Aerated Lagoons

This type of aerated lagoon is referred to in the literature as a facultative aerated lagoon, incompletely mixed aerated lagoon and aerobic-anaerobic aerated lagoon (Thirumurthi, 1980). Sackellares (1985) used the term aerated stabilization basin for an aerated lagoon of this type. In the remainder of this literature review and in other chapters, the term facultative aerated lagoon will be used most frequently to refer to such type of aerated lagoons. Facultative aerated lagoons have demonstrated their ability to produce relatively good effluents with low power input and a saving in land area (Bartsch and Randall, 1971).

The power input per unit volume is sufficient to diffuse the required amount of oxygen into the liquid, but not sufficient to maintain all the solids in suspension. Depending on the power level at which these types of lagoons are operated, the major portion of inert suspended solids and nonoxidized biological solids settle to the bottom of the basin where part of them undergo aerobic/anaerobic decomposition and the remainder will compact on the lagoon bottom. An aerobic decomposition takes place in the lower layers of the sludge while aerobic solids decomposition is in the upper layers. Accordingly, in facultative aerated lagoons two distinct biological communities are found, aerobic bacteria and higher order eukaryotes in the upper fluid and anaerobic bacteria in the bottom sludge (Sackellares, 1985). The products of hydrolyzation of complex organics in the sludge layer are released back into the solution. Therefore, low effluent total and volatile suspended solids from facultative aerated lagoons are expected (Kormanik, 1972).

The facultative aerated lagoon design usually consists of a single lagoon of large size or a series of smaller lagoons. These lagoons are operated at higher retention times and lower loadings than completely mixed aerated lagoons. This provides for stabilization of the organic matter incorporated into the cellular mass, which results in lower solids concentration. These facultative aerated lagoons have a definite advantage in that they approach the total oxidation concept (Sawyer, 1968).

This type of aerated lagoon may be favoured for large number of situations under which the stabilization pond may not be acceptable owing to its high land requirement, and other methods such as activated sludge may not be as desirable either owing to their technological requirements or simply because a higher quality effluent is not essential. They are being used successfully in the treatment of sewage and many different types of industrial wastes. However, land requirement is larger than for aerobic aerated lagoons.

Design equations and performance as well as requirements of facultative aerated lagoons from experimental studies and experience have been reviewed in this chapter. There are certain factors, as documented in the literature, affecting the performance of facultative aerated lagoons in particular and aerated lagoons in general. These are either natural (e.g., temperature) or artificial (e.g. power level) and are discussed briefly below.

i) Organic Matter Removal in Facultative Aerated Lagoons

The equations used for determination of the removal of organic matter in completely-mixed aerated lagoons are frequently used for facultative aerated lagoons (Eckenfelder, 1970). However, the solids that settle to the bottom of the lagoon decompose during the summer, releasing soluble organic materials to the liquid above (White and Rich, 1976a and b; Rich 1982c). Such materials, called seasonal feedback, add to the substrate load in the lagoon. As a result, the effective influent substrate concentration to the lagoon, S_0, during this period may be as much as 50% greater than that actually found in the influent. It was observed by Balasha and Sperber (1975) that, in aerated lagoons, most of the soluble BOD is removed in a short period. For facultative aerated lagoons:

$$S_1 = \frac{\beta\ S_0}{1 + Kt} \tag{2.12}$$

or:

$$S_1 = \frac{\beta\ S_0}{1 + kXt} \tag{2.13}$$

where, β = factor related to the BOD feedback (≈1-1.5).

In some publications (e.g. Bartsch and Randall, 1971) the coefficient β was not included in the equation. Tikhe (1975) stated that the value of the specific removal rate coefficient, k, would not change with change in lagoon type, completely-mixed or partially-mixed aerated lagoons, because it is independent of the population of microorganisms. The difference in overall BOD reduction rate for facultative aerated lagoons is the result of the level of volatile suspended solids present in the lagoon (Kormanik, 1972), also the rate at which the settleable solids in the waste are liquefied (Bartsch and Randall, 1971). At optimum retention time, maximum active mixed liquor suspended solids concentration and BOD_5 removal occur (Malina *et al.*, 1971a and b); also the activity of the bacterial population will be highest (Rudd, 1972).

For a particular waste and a particular lagoon design, there is a K value determined using one of the above equations. Because of the low levels of the active biological solids in the facultative aerated lagoons, BOD removal will primarily be a function of retention time in the lagoon, temperature, and the nature of the waste treated (Bartsch and Randall, 1971). In partially-mixed aerated lagoons overall removal rate coefficient values are smaller than for completely-mixed aerated lagoons.

Arceivala (1981) pointed out that the part of the BOD_5 associated with the incoming solids and new solids produced, as a result of substrate removal, that settle down and entered the anaerobic zone in the lagoon will go through either liquefaction, liquefaction and gasification or neither. Thus the removal in the lagoon bottom, conversion to gases and feedback to the upper liquid volume can be described by:

a. No removal in the bottom if no liquefaction or gasification, and the value of β S_0 will be assumed to be 70% to 100% of the total influent ultimate BOD. This only occurs in very cold climates.
b. Only soluble organic feedback to the upper layers by liquefaction process without gasification process. The value of the coefficient β S_0 will be assumed to be 100% of the total influent ultimate BOD. This occurs in relatively lightly loaded, well aerated , or shallow lagoons at favourable temperatures.

c. Soluble feedback (liquefaction process) and gasification of some of these solids (conversion to carbon dioxide and methane). The value of the coefficient $\beta\ S_0$ will be assumed to be 40% to 70% of the total influent ultimate BOD. This occurs in heavily loaded, deep lagoons operated at low power levels in warm climates.

Some researchers (e.g. Thirumurthi, 1980) recommend the use of the dispersed flow model given by Wehner and Wilhem for first-order kinetics. Middlebrooks (1987) pointed out that he found that the first-order, plug flow, and complete-mix models and variations thereof do not describe the performance of facultative or aerated pond systems. Arceivala (1981) considered this method to be more appropriate if lagoon configuration is not likely to promote well-mixed conditions. This equation is as follows:

$$\frac{S_0}{S_1} = \frac{4a\exp(1/2d)}{[(1+a)^2\exp(a/2d)]-[(1-a)^2\exp(-a/2d)]} \qquad (2.14)$$

where, $a = (1+4Ktd)^{0.5}$, and

d = dispersion number; dimensionless.

Different typical values for the removal rate coefficient in facultative aerated lagoons have been reported, some of which are summarised in Table 2.4. There can be a wide variation of K values for similar wastes. The reason for this wide variation in reported K values could result from the assumptions made in the derivation of the equation, which are for steady-state conditions, first-order BOD removal, and a completely-mixed flow regime (Bartsch and Randall, 1971).

Table 2.4 BOD_5 Removal Rate Coefficients (overall and specific) Reported for Facultative Aerated Lagoons.

Coefficient value	Remarks	Reference
0.232 - 0.287 d^{-1}	Overall, at 20°C, for BOD_5, mixture of domestic wastewater (90%) and synthetic sewage (10%), using Eq. 2.14	Thirumurthi (1980)
0.01-0.03 l/mg.d	Specific, at 20°C, for BOD_5, municipal wastewater	Arceivala, 1981
0.6-0.8 d^{-1}	Overall, at 20°C, domestic wastewater, filtered samples	
0.3-0.5 d^{-1}	Overall, at 20°C, domestic wastewater, unfiltered samples	
0.19-0.45d^{-1}	Overall, at 20°C, for BOD_5, domestic wastewater, proceeded by aerobic aerated lagoon, using Eq. 2.12 with β value of 1 and using soluble influent and effluent samples.	Kouzell-Katsiri, 1987

As in aerobic aerated lagoons, removal rate coefficients are affected by organic loading (Thirumurthi, 1980; Kouzell-Katsiri, 1987). Thirumurthi (1980) proposed a relationship to correct the value of K as a result of organic loading increase or decrease in terms of the following equation:

$$\log 20 - \log L_l = 8.5\,(K_s - K_{20}) \qquad (2.15)$$

where, K_{20} = removal rate coefficient at 20°C, and

L_l = organic loading; g BOD_5 /m^3.d.

The value of the standard design coefficient, K_s, he proposed was 0.29 d^{-1}. The standard conditions he used were 20°C liquid temperature and 20 g BOD_5 /m^3.d organic loading. The K values he used in developing Equation 2.15 were determined by using Equation 2.14.

Kouzell-Katsiri (1987) found a relationship between removal rate coefficient and volumetric organic loading for partially-mixed aerated lagoons. The relationships he found were developed from his study of a laboratory scale facultative aerated lagoon preceded by a completely-mixed aerated lagoon. Both the influent and effluent BOD_5 values he used for the determination of K coefficients (using Equation 2.12 with β value of 1) were for filtered samples. The relationship is as follows:

$$K_{20} = 0.17 + 0.22 \log L \qquad (2.16)$$

ii) Solids Concentration in Facultative Aerated Lagoons

An estimate of the solids concentration in the mixed liquor is most important, partly because of effluent suspended solids and discharge standards and partly because of its use in the equation used for the determination of organic matter removal efficiency. However, it should be noted that part of the effluent BOD_5 and COD are associated with particulate solids. The determination of settled solids, quantity and rate of sedimentation, have rendered determination of the true mean solids (or sludge) retention time impossible.

Biomass concentrations maintained and the degree of deposition in partially suspended aerated lagoons are a function of the aeration power levels maintained in the system (Malina *et al.*, 1971a and b; Rich and White, 1976a; Bennet, undated; Rich, 1982c), size and nature of the solids (Arceivala, 1981), BOD loading and the suspended solids present in the influent (Malina *et al.*, 1971a and b; Eckenfelder *et al.*, 1972; Rich, 1982c), and retention time (Malina *et al.*, 1971a; Bennet, undated). Retention time of the solids and specific decay rate of the biomass solids have also been reported to influence average biomass concentration in facultative aerated lagoons (Rich, 1982c). For this reason, it is difficult to make a prior estimate of the value of X in Equation 2.13.

Different solids levels have been reported from field studies. At 0.75 W/m^3 power input the concentration of solids in suspension may range between 30 and 150 mg/l and for domestic sewage this concentration may be around 40 to 60 mg/l (Arceivala, 1981). Adams and Eckenfelder (1974), based on observations in existing aerated lagoons treating domestic wastewater, observed a concentration of 50 mg/l at 0.75 W/m^3, 175 mg/l at 1.75 W/m^3, and 300 mg/l at 2.75 W/m^3 power level. White and Rich (1976b) reported from field studies on partially suspended lagoons treating domestic wastewaters for power levels of 1 W/m^3, that a biomass concentration of no more than 20 mg/l can be expected to be maintained in suspension. Rich (1978) reported that in facultative aerated lagoons operated at 1.2 W/m^3 the concentration of nonsettleable suspended solids will be 25 mg/l. He considered 1.6 W/m^3 to represent the threshold power level below which no settleable solids are maintained in suspension and presented the following relationship between the concentration of settleable solids and the power level (P):

$$\text{Settleable SS (mg/l)} = 62.5\ P(W/m^3) - 100 \qquad (2.17)$$

In a later study, Rich (1982c), based on the extrapolation of data of Fleckseder and Malina (1970), also made the assumption that a power level of 2 W/m^3 is the threshold power level and the nonsettleable suspended solids will be as that stated in his former (1978) study. Malina *et al.* (1971a) reported a value of 55 mg/l of mixed liquor suspended solids at power levels between 2 and 2.6 W/m^3. At low power level (1.6 W/m^3), sufficient only to disperse oxygen, Eckenfelder (1967) reported that the suspended solids rarely exceeded 50 to 100 mg/l. Kouzell-Katsiri (1987), in his studies on laboratory scale aerated lagoons observed that for a facultative aerated lagoon operated at 1 W/m^3, the concentration of the mixed liquor suspended solids will be 25 mg/l. It was also stated that finely dispersed biological growth will cause suspension of all of the solids even at low power levels (Arceivala, 1981). This usually occurs at low organic loadings.

iii) Oxygen Requirements in Facultative Aerated Lagoons

Oxygen is required for the breakdown of carbonaceous substrate and the predation and endogenous wasting of the microbial cells formed with a further demand for nitrification (Eckenfelder *et al.*, 1972; Heddle, 1982; Frey, 1992). The latter greatly affects the rate of oxygen utilization (Billings and Smallhorst, 1971). The dissolved oxygen concentration in the liquid body should be maintained above approximately 1 mg/l (Eckenfelder and O'Connor, 1961).

Because of the aerobic/anaerobic activity in the lagoon bottom, soluble feedback and anaerobic by-products are released to the upper layers and in turn oxidized in this upper aerobic layer of the basin, oxygen requirements will be more than required for the removal of influent soluble BOD_5. Therefore, depending on the extent of the anaerobic activity occurring, oxygen requirements will vary. Various investigators (Bartsch and Randall, 1971; Arceivala, 1981; Bennett, undated) consider that the required oxygen is equivalent to the soluble ultimate BOD_5 removed aerobically. Bartsch and Randall (1971) gave a relationship for determination of the normal biological oxygen requirement. This relationship is related to the ultimate BOD (BOD_u) removal as follows:

$$\text{Oxygen required} = BOD_u \text{ removed} \qquad (2.18)$$

WHO (1992) adopted 1 to 2 kg O_2/kg BOD_5 removed as design criteria for aerated facultative pond, whereas Arceivala (1981) considered the direct oxygen demand at the sludge-water interface exerted by decomposition of the biodegradable solids that settle to the bottom. The feedback from the bottom layers varies with temperature level and loading condition, as discussed earlier in this chapter. Accordingly, depending on the temperature and loading conditions, oxygen requirements can be estimated. Soluble BOD and nutrients benthal feedback from settled deposits of sludge have a major impact on the performance of heavily loaded lagoons, and the effect usually becomes greatest as the wastewater approaches the discharge pipe (Bryant, 1993). Dissolved oxygen concentration in the overlying water and the surface loading rate of settled solids affects feedback levels. Bryant (1993) estimated from laboratory results that 0.43 mg COD and 0.3 mg BOD_5 are the soluble feedback per mg TSS settled, whereas Eckenfelder *et al.* (1972) quoted from Marais and Capri (1970), who indicated that feedback may be 0.4 mg BOD_5/mg volatile suspended solids. Sawyer (1968) stated that the aerators in facultative aerated lagoons must be adequate to supply oxygen sufficient to satisfy the total oxygen demand of the waste, which may range from 125 to 200 percent of the BOD_5 value, depending upon the BOD reaction rate constant (Sawyer, 1968). It should be considered that an increase in mixed liquor suspended solids will, relatively, reduce the oxygen transfer capacity of the aeration system (Benjes and McKinney, 1967).

In facultative aerated lagoons treating domestic wastewaters the minimum power required for running aerators is calculated from the oxygenation requirements. However, for high strength wastewaters (e.g., industrial wastes) the power levels required to satisfy oxygen needs might be higher than that for solids suspension, therefore it is necessary to increase retention time or lagoon volume. Generally, the oxygenation achieved under field conditions is only 60-80 percent of that under standard test conditions.

Considering only influent and effluent BOD_5 in the calculations, Eckenfelder *et al.*(1972) and White and Rich (1976b) included the feedback coefficient, β, in the estimation of the rate of oxygen required in facultative aerated lagoons. β values range from 0.8 in winter to 1.5 in summer were obtained from the following equation:

$$\text{Oxygen required per unit time} = \beta \times \text{flow rate} \times BOD_5 \text{ removed} \qquad (2.19)$$

Accordingly, power levels required to satisfy oxygen requirements should be designed for summer operation, due to the higher rates of BOD removal, benthal feedback and lower rates of oxygen dissolution. To ensure the availability of dissolved oxygen in a lagoon treating municipal wastewater, Arceivala (1981) recommended that the minimum power level required is 0.75 W/m^3.

iv) Organic Loading in Facultative Aerated Lagoons

Sawyer (1968) pointed out that facultative aerated lagoons are, in general, applicable where BOD_5 loadings do not exceed 80 g BOD_5 /m^3.d. He justified this by stating that at higher loadings the high mixing required to satisfy the oxygen requirements will not permit sedimentation of the solids within the lagoon. However Townshend *et al.* (1969) pointed out that successful operation of facultative aerated lagoons has been demonstrated with applied organic loadings up to 32 g BOD_5 /m^3.d. McKinney and Edde (1961) described the aerated lagoon with organic loading of 32 g BOD_5 /m^3.d to be a very heavily loaded oxidation pond or a lightly loaded activated sludge system. Oleszkiewicz and Sparling (1987) applied a range of loads (from 20 to 500 g BOD_5 /m^3.d) on the completely mixed aerated lagoons in the Manitoba, Canada, which had temperature extremes of -45°C in winter and +40°C in summer.

Thirumurthi (1980) used an organic loading of 6.7 to 19.8 g BOD_5 /m^3.d in his laboratory-scale facultative aerated lagoons for the treatment of municipal wastewater. The full scale facultative aerated lagoons in Canada that were studied by Lewandowski and Bradley (1981) were loaded at 1.6 to 46.5 g BOD_5 /m^3.d and they achieved BOD_5 removals of 41 to 71%.

v) Power Levels in Facultative Aerated Lagoons

The degree of mixing in aerated lagoons depends on type of aeration system used (surface aerator or diffused air aerators), lagoon size and geometry and wastewater characteristics (e.g. settleable solids concentration) (Fleckseder and Malina, 1970; Rich, 1982c). The degree of mixing is widely measured in terms of power level expressed as watts per unit volume (m^3) (Fleckseder and Malina, 1970). This power level is required to maintain a given turbulence level only to distribute dissolved oxygen uniformly in the liquid volume or with part or all of the settleable solids in suspension. The latter is that required for aerobic aerated lagoons. In practice, it has been found that the power level required for solids suspension is fivefold that required for oxygen dispersion (Narasiah *et al.*, 1987). However, different power levels were reported for both types aerobic and facultative aerated lagoons. Table 2.5 lists power levels reported for the different types of aerated lagoon. Aeration power level will have an effect on the rate of oxygen transfer (Benefield and Randall, 1980).

Table 2.5 Power Levels and Type of Aerated Lagoons Reported.

Power Level (W/m^3)	Lagoon Condition and Description	Reference
1.6	Uniform distribution of dissolved oxygen	Eckenfelder (1967)
6 - 12	Maintain solids uniformly in suspension	
6	Aerobic lagoons	Fleckseder and Malina (1970)
20	Complete solids suspension in $500m^3$ lagoon	
15	Complete solids suspension in 1000 m^3 lagoon	
10	Complete solids suspension in $2000m^3$ lagoon	
1	Uniform BOD_5 in lagoon, mechanical surface aerators, L/W of 1:2	NCASI (1971)
2	Uniform BOD_5 in lagoon, mechanical surface aerators, L/W of 1:4	
2.8	Complete solids suspension	
4	All solids were in suspension, MLSS was 130 mg/l	Malina *et al.*, (1971a)
6	Completely mixed tank (from tracer studies and relationship between MLSS and power level).	
1.8 - 2.6	Only equilibrium solids were in suspension	
4 - 6	Practically all of the solids will be suspended, for surface aerators	Rich (1978)
1.3, 1.6, 1.8	Full scale facultative aerated lagoon, surface aerators	Lewandowski and Bradley (1981)
2.6, 3.9	Full scale complete mixed lagoons with surface aerators	
2.4	Completely mixed full scale aerated lagoon treating Kraft pulp and paper mill effluent.	Cook and Chandrasekaran (1986)
1.6	Partially mixed full scale aerated lagoon treating Kraft pulp and paper mill effluent.	
1	Threshold power level for complete suspension	Kouzell-Katsiri (1987)
5	Threshold power level for partial suspension	

Rich (1982c) quoted from Associated Water and Air Resources (personal communications, 1976) a relationship of power level required for maintaining settleable solids in suspension for low-speed mechanical surface aerators. This relationship is a function of the concentration of the suspended solids (mg/l) only and limited (restricted) to a maximum of suspended solids of 2000 mg/l, as follows:

$$\text{Power level} = (0.004 \times SS) + 5 \tag{2.20}$$

At low retention times the power levels required for satisfying the oxygen requirements will be higher than those required for solids suspension. This critical retention time was found by some investigators to be 0.6 day (White and Rich, 1976b) and 1.2 day (Malina *et al.*, 1971a).

vi) Nutrient Additions in Facultative Aerated Lagoons

In facultative aerated lagoons treating domestic wastewater, nutrient addition is not required, whereas it may be necessary in the case of certain industrial wastes to ensure optimal operation. Nitrogen and phosphorus are released back into solution during anaerobic degradation of bottom deposits, hence low nutrients are required. However, there may be a need for some addition of nitrogen and phosphorus but this need is less than for aerobic aerated lagoons. It was observed that the need for nutrients for treating some industrial wastes will be less in summer than in winter (Amberg and Bachman, 1981). If these two elements are insufficient for biological operations, the removal rate coefficient will decrease, causing a decrease in the performance of the facultative aerated lagoon.

Arceivala (1981) pointed out that retention time could be increased to compensate for nutrients deficiency. With this practice the decrease in removal efficiency could be avoided but this required larger land area. If there is a need for addition of nutrients, then ammonia and dibasic sodium phosphate can be added to the influent wastewater (Mancini and Barnhart, 1967).

vii) Algae and Effluent Quality from Facultative Aerated Lagoons

The use of facultative aerated lagoons is much more common than the use of aerobic lagoons, mainly because a good quality effluent can be produced with a lower power input. For readily degradable wastes such as domestic sewage it is expected that a removal of 70 to 90% BOD can be achieved, provided adequate retention time is allowed. However, the effluent from facultative aerated lagoons is of lower quality than that from activated sludge systems (Townshend *et al.*, 1969). Oxidation, clarification and aerobic/anaerobic sludge digestion should occur in any aerated lagoon operating alone (Bryant, 1993)

Malina *et al.* (1971b) and Bennet (undated) listed the factors that affect the effluent quality from a facultative aerated lagoon. These include power level, retention time, the distance of the aerators from the effluent, as well as type, size and construction of the effluent weirs. Malina *et al.* (1971b) also reported that a full-scale aerated lagoon operated at 6.4 W/m^3 cannot be applied as a single treatment unit because there is no removal of suspended solids and the total effluent substrate concentrations are very high. In another study, Malina *et al.*, (1971a) added more factors including the location of the effluent and the construction of the effluent structure as baffles and settling basins. On the other hand, with increase in retention time, efficiency per unit retention time will decrease, as stated by Rudd (1972), who explained this by the senescence of the bacterial population causing a reduction in its physiological activity.

Normal algal growth in oxidation ponds is retarded or eliminated if aerators are added (Bartsch and Randall, 1971) due to turbidity, turbulence and other factors. Therefore, in aerobic aerated lagoons growth of algae is not expected. Mara (1976) observed that algae disappeared and the microbial flora resembled that of activated sludge soon after aerators were put into operation in wastewater stabilization ponds. In facultative aerated lagoons the retention times are longer than those for aerobic aerated lagoons, whereas the power level is lower and this may allow the growth of algae. Malina *et al.* (1971a) claimed that, with the development of algae, the effluent suspended solids concentration may not change but the composition will be different. However, it is cited in the literature that this will influence the quality of the effluents from such treatment processes. It has been found that the concentration of suspended solids is quite significant during the summer season because of increased algal production in such lagoons (Narasiah *et al.*, 1987). Algae will exert a high BOD in a standard laboratory BOD test involving darkroom incubation and will show a high SS value (Khandual, 1987). Water Pollution Research Laboratory (1973) reported that if a lagoon produces a large amount of phytoplankton, this will exert an oxygen demand equivalent to about 1 mg BOD_5 /mg algal suspended solids in the traditional BOD test. It was also reported that algal samples yielded 10 g COD/ 100 mg chlorophyll "a" (Naehle, 1987, cited by Bryant, 1988). However, if the BOD test is carried out under optimum lighting conditions, the algae will produce, on average, about ten times as much oxygen as they consume in the dark. Therefore, the development of algae in the lagoon should be considered in design.

At high temperatures and loading conditions it is expected that effluent from facultative aerated lagoons will be high in BOD_5 and low in suspended solids if algae in the lagoon are at a low condition. This is because of the release of soluble materials from the bottom deposits to the water column above. Rich (1982c) stated that the effluent total BOD_5 and TSS of aerated lagoon systems, if algal biomass had been excluded, would drop from 45 and 85 mg/l, respectively, to 25 mg/l. Bacterial floc settles readily in lagoons, whereas algae do not settle readily and end up primarily in the effluent (Rich, 1989) with no alteration in algal settling rates between aerobic and anaerobic conditions (George, 1980). The removal of algal cells from aerated lagoon effluent was studied by Abernathy *et al.* (1990) who observed that 6 μm microscreen media is preferable to 1 μm media for the removal of these algal cells. Each gram of chlorophyll "a" was estimated to contribute 143 gram to the suspended solids (White and Rich, 1976a; Rich, 1978). Chemical and physical processes (e.g. coagulation, flocculation, filtration and dissolved-air flotation) and biological processes, including grass plots, rock filters, floating macrophytes, fish, micro-invertebrates, are the processes used for removing algae from water (Mara *et al.*, 1992b). High turbulence will increase the BOD_5 and SS of the effluent, due to inhibition of the bacterial floc sedimentation, and at the same time will result in low algal growth. The presence of algal and bacterial floc in the effluent will cause the BOD_5 to be high through endogenous respiration. More disadvantages of the presence of algae in effluent are discussed elsewhere (Mara, *et al.*, 1992b).

White and Rich (1976a) studied the performance of six aerated lagoon systems in South Carolina, each consisting of one aerated lagoon and followed by a polishing pond. They observed that in some instances the effluent from the polishing pond had larger BOD_5 and suspended solids concentrations than did the influent. They attributed this to high algal concentration in these effluents. However, they concluded that the non-algal fraction of the total suspended solids in the effluents from the aeration cells operated at low power levels appeared to meet the 30 mg/l standard. In addition, they showed graphically the relationship between the power levels into the aeration cells and the chlorophyll "a" concentration in the effluents from these aeration cells. According to their study, the concentration of chlorophyll "a" from the aerated lagoon will be very high (200 - 1000 μg/l) at power levels of about 1.2 W/m^3 and this decreases with increase in power level, reaching zero at about 7.5 W/m^3. Thus, to obtain better effluent quality it is important to have effective control of lagoon effluent solids. For this the system should be designed in such a way that the growth of algae will be reduced or eliminated and bacterial biomass removal facilitated by improving sedimentation characteristics (Rich, 1978). Lewandowski and Bradley (1981) concluded that aerated facultative cells usually produce effluents with considerably lower effluent suspended solids concentrations.

viii) Temperature Effects on Facultative Aerated Lagoons

It is generally accepted that temperature increases microbial respiration rates (Rudd, 1972) and activity (Narasiah *et al.*, 1987) improve substrate removal efficiencies, decrease sludge production, and encourage nitrification. Thus, temperature plays a significant role in the overall efficiency of facultative aerated lagoons. With decrease in temperature, the rate of sludge oxidation will decrease causing an increase in effluent insoluble BOD. To ensure optimum treatment efficiency in a biological treatment process an appropriate environment must be provided. Bacteria utilized in treatment processes are classified as either psychrophilic, mesophilic, or thermophilic, which classification depends on the temperature level at which they can grow well. The optimum temperature (for mesophylic bacteria) for biological oxidation in an aerobic system is 37°C (Timpany *et al.*, 1971; Barnhart, 1972).

Oxygen transfer rate and solubility and rate of biodegradation in the biological process will be affected by temperature (O'Connor and Eckenfelder, 1960; Rudd, 1972; Narasiah *et al.*, 1987). Additionally growth processes are dependent on chemical reactions and the rate of such reactions are affected by temperature levels. Also, nutrients requirements for certain wastes increase as temperature decreases (Cook and Chandrasekaran, 1986).

Under certain operational conditions, BOD removal efficiency in facultative aerated lagoons is affected by mixed liquor temperature (Balasha and Sperber, 1975). It was observed that total BOD in the effluent from a full scale facultative aerated lagoon decreased with decrease in temperature (Malina *et al.*, 1971b). Malina *et al.* (1971a) pointed out that temperature has very little effect on effluent BOD at long retention times, while Sawyer (1968) provided data showing that temperature exerts no influence on BOD removal efficiency of aerated lagoons when the retention period is longer than 4.5 days. It is likely that higher retention periods allow higher stability to be established within the lagoon, which renders it less sensitive to temperature changes.

Because of the low solids levels maintained in suspension, 25 mg/l (Rich, 1978), 50 to 100 mg/l (Kormanik, 1972), 50 to 500 mg/l (Lal and Verma, 1989), facultative aerated lagoons have been found to be quite sensitive to temperature and other environmental changes (Sawyer, 1968; Malina *et al.*, 1971a). Therefore, facultative aerated lagoons are more sensitive to temperature than are aerobic aerated lagoons and other treatment processes such as activated sludge. Caution must also be exercised in the choice of the various design criteria relative to temperature variations at a particular locality. Other investigators (Bartsch and Randall, 1971) doubt the use of facultative aerated lagoons for the production of high quality effluents in winter months.

Mixed liquor temperature is affected by the temperature of influent wastewater and the ambient air temperature.

The variation of K with respect to temperature can be determined through the relationship:

$$K_2 / K_1 = \theta^{(T_2 - T_1)}. \qquad (2.21)$$

where, K_1 = removal rate coefficient at T_1 temperature,

K_2 = removal rate coefficient at T_2 temperature, and

θ = temperature correction coefficient.

High values of θ represent higher changes in removal rate coefficients and means higher temperature effects. Different values of θ have been reported in the literature. Table 2.6 lists some of these values. The above equation can be used for temperatures in the range of 5° to 35°C (Barnhart, 1972). However, Horan (1990) asserted that this equation should not be used outside the range 5 to 25°C, justifying this by citing the significant changes that occur in microbial populations at temperatures outside this range and which in turn invalidate the equation. Horan (1990) also pointed out that at temperatures above 20°C, BOD removal rate seems not to be significantly affected by temperature, as observed by several researchers.

Retention time required for a given BOD removal efficiency under winter conditions controls the design retention (Bartsch and Randall, 1971) because at the coldest time of the year the retention time required for a given removal efficiency will be higher than in warmer months. Bartsch and Randall (1971) concluded that the efficiency of the aerated lagoon system was strongly affected by temperature changes at temperatures below 14.4°C. At temperatures of 14.4°C and above they observed that there is no significant change in efficiency occurring with change in temperature and they considered that the system had a temperature threshold value

Table 2.6 Temperature Correction Coefficients Reported for Aerated Lagoons.

Coefficient value	Remarks	Reference
1.06 - 1.09	For temperature range of 10 to 30°C	Eckenfelder (1966)
1.065 - 1.085 1.035	Aerobic facultative lagoon Aerobic lagoon.	Eckenfelder (1967)
1.016	Over the range 2 to 30°C, for aerated lagoons treating wastes from pulp and paper mills.	Vamvakias and Miller (1969)
1.097 1.11 - 1.33	Four aerated lagoons in series, wastewater from textile-finishing plant. In the lagoons above but at temperatures less than 10°C	Bartsch and Randall (1971)
1.035	Over the range 14 to 34°C, for aerated lagoons treating wastes from pulp and paper mills.	Timpany *et al.* (1971)
1.08	Considered to be generally used for aerated lagoons	Malina *et al.*, (1971a)
1.03 - 1.04	The lagoon have a character close to fully mixed, power level of 2.7 W/m^3, full scale aerated lagoon treating domestic sewage.	Balasha and Sperber (1975)
1.035	Aerobic aerated lagoon, laboratory scale, domestic wastewater (90%) and synthetic sewage (10%)	Thirumurthi (1979)
1.035	Completely mixed aerated lagoons and facultative aerated lagoons.	Arceivala (1981)
1.035	Aerated lagoons	Horan (1990)
1.06	Adopted as design criteria for aerated facultative pond	WHO (1992)

of 14.4°C. Interestingly, Hurwitz (1963) did notice a well defined difference between the summer and winter performance of a 0.9 acre lagoon in the Chicago area aerated by a diffused aeration system and stated that the biological action was not slowed materially during cold weather and ice coverage. In addition, it was reported (Olson, 1966) that no odours or operational difficulties were observed in a lagoon aerated with this type of aeration system during winter months.

Narasiah (1983) stated that aerated lagoons, even in cold weather, will function satisfactorily providing that proper care is given to their design. Victorio *et al.* (1991) reported that the number of total and viable bacteria in facultative aerated lagoons treating pulp effluent, decreased and the proportions of different groups of bacteria shifted when lagoon basin temperatures fell below 16°C .

The liquid temperature in a facultative aerated lagoon can be determined by the following equation suggested by Eckenfelder (1966):

$$T_w = f\, A\, (T_w - T_a)\, (T_i - T_o)\, Q \tag{2.22}$$

where, T_w = liquid temperature in the lagoon; °F,

f = proportionality factor,

A = surface area of the lagoon,

T_a = air temperature; °F,

T_i = influent wastewater temperature; °F,

T_o = effluent wastewater temperature; °F, and

Q = wastewater flow rate.

The proportionality factor is a factor containing the heat transfer coefficients, the surface area from aeration equipment, wind, and humidity effects.

As is clear from the above equation, the temperature of the facultative aerated lagoon contents is proportional to the area of the lagoon.

ix) Nutrients Removals and Nitrification in Facultative Aerated Lagoons

In facultative aerated lagoons, nitrification generally does not occur; however, as a result of solids settlement there may be some nitrogen removal. Also, some nitrogen is removed in the lower layers of the lagoon under certain conditions, though mechanisms for this removal are as yet unexplained (Arceivala, 1981). Kouzell-Katsiri (1987) noticed that a high degree of nitrification was achieved in a partially-mixed lagoon, which is probably caused by the accumulation of nitrifying organisms in the sludge settled out in the bottom of the lagoon. Degree of mixing has a negative effect on nitrification, as was observed in bench-scale activated sludge reactors (Azimi and Horan, 1991).

Metcalf and Eddy (1991) pointed out that seasonal and continuous nitrification may be achieved in aerated lagoon systems. The design and operating conditions within the system affect the degree of nitrification. At higher temperatures and lower loadings (increased sludge retention time) a higher degree of nitrification can be achieved. Middlebrooks and Pano (1983) found a poor relationship between the nitrogen reaction rate and temperature using different models, which indicates that the relationships predicted by the models were not affected by temperature. They justified this by stating that other parameters which may include dispersion, retention time, light and species of organisms, "overshadowed" the influence of temperature.

Narasiah (1983) in his studies on two full-scale aerated lagoons in Quebec, Canada, operated at 3.5 W/m^3 operating in series observed that phosphate removal was 43% in summer (19°C wastewater temperature) and 16% in winter (3°C wastewater temperature). This difference is due to increased bacterial activity and also the up-take of phosphorus by algae. Narasiah *et al.* (1987) in a later study on the same aerated lagoons found that these lagoons produced an effluent having 2-3 mg/l of total phosphates when treating domestic wastewater in summer at a retention time of about 8 days, without coagulant addition. In the former study he had concluded that removal of phosphates was very low without coagulant addition.

However, some researchers (Loehr and Schulte, 1970) assert that the bacterial synthesis of phosphates in aerobic processes is very poor and that the best way to reduce phosphate levels in treatment plants is to resort to chemical coagulation and flocculation using alum or ferrous sulphate. Chemical addition can be to (a) raw or settled wastewater followed by an aerated lagoon, (b) raw wastewater followed by settling and an aerated lagoon, or (c) the effluent from the aerated lagoon. The sludges of these salts are more chemical than biological in nature and their disposal is expensive.

x) Pathogen Removal in Facultative Aerated Lagoons

It is important to reduce the number of pathogenic microorganisms from wastewater that is to be treated and discharged to a receiving water used for potable supply, contact recreation, or shell fisheries (Water Pollution Research Laboratory, 1973) or for irrigation of crops. Faecal indicator organisms are used as indicaters of the presence of pathogenic microorganisms. Of these, faecal coliforms and faecal streptococci are usually used as a faecal contamination indicators. The natural factors that are expected to be responsible for pathogenic organisms removal include sedimentation, lack of food and nutrients, solar ultraviolet radiation, high water temperatures, high pH value, predators (bacteriophages, microcrustaceans, protozoa and rotifers), the toxins and antibiotics exerted by some organisms, as well as natural die-off.

Coliform removal in facultative aerated lagoons has been reported to range from 90 to 99 % in summer and 60% or less in winter, in single-celled lagoons (Arceivala, 1981). A facultative aerated lagoon with 6 day retention time followed by a polishing pond (2 day retention time) gave a removal of 99.5 percent (Thomas and Phelps, 1987). Other researchers stated that aerated lagoons are not effective in removal of faecal bacteria and this removal is in the range 90 to 95 percent (Mara, 1976), or 85 to 95 percent removal of faecal coliforms in aerated lagoons with four days retention time (Feachem *et al.*, 1977). Lower removals were also reported, 52 % reduction in total coliform and 20 % of faecal coliform in a facultative aerated lagoon with 2.5 days retention time (El-Gohary *et al.*, 1993),

Effluents from facultative aerated lagoons do not comply with normal discharge standards. Panicker and Krishnamoorthi (1981) concluded that the removal efficiency of parasite eggs and cysts in an aerated lagoon was less than in an oxidation ditch. To improve the removal of coliforms in aerated lagoons it is recommended that these lagoons are designed to be two or more lagoons in series, otherwise further treatment may be necessary. If the effluent from the aerated lagoon is allowed to settle in a settling tank the quality will improve, with respect to pathogenic microorganisms. Maturation ponds were also recommended and in some literature (e.g. Feachem *et al.*, 1977) reported to be necesary.

xi) Solids Sedimentation and Sludge Accumulation in Facultative Aerated Lagoons

Growth of new bacterial cells occurs in facultative aerated lagoons as a result of the bacterial oxidation of organic material. Some of these solids, as a result of the low power levels, are deposited and accumulate in the bottom of the lagoon or in relatively quiescent areas. Therefore, in high turbulence areas near the aerators it is not expected to observe any settlement of these solids (Bartsch and Randall, 1971). The undegraded organics, inert residue, and inorganic settled materials are the constituants of these setled solids, which are known as sludge. The rate of this accumulation was reported to be dependent on the organic loading (NCASI, 1971; Barnhart, 1972; Eckenfelder *et al.*, 1972), the retention time employed (Barnhart, 1972), and the suspended solids present in the influent wastewater (Eckenfelder *et al.*, 1972). At organic loading levels of less than 2242-3363 kg BOD_5 /ha.d (2000-3000 lb BOD_5 /acre.d) it was observed that there was no settleable solids production (NCASI, 1971).

The deposition of settlable solids by gravitational force is affected by the turbulence level in the lagoon and the surface overflow rate from the cell, as well as the fraction of suspended solids that is nonsettleable (Rich, 1982c). The concentration of the solids in the effluent from an aerated lagoon operated at 2.7 W/m^3 deposited in a polishing pond after two years of operation was 4 to 5% (Balasha and Sperber, 1975). Some suspended solids in the wastewater from industrial plants may pose a problem, depending on their biodegradability, because they might undergo slow anaerobic decomposition (Loehr and McKinney, 1966).

Bryant (1993) estimated that aerobic oxidation yields 0.5 mg biomass per mg BOD_5 utilized, using $C_5H_7O_2NP_{0.5}$ as a common formula for the chemical composition of bacterial biomass. Lower figures were also reported by Amberg and Bachman (1981) and Cook and Chandrasekaran (1986), who were investigating the performance of a lagoon system treating Kraft pulp and paper mill effluent consisting of a settling basin, completely-mixed aerated lagoons, facultative aerated lagoon and another settling basin connected in series. They reported that 0.3 and 0.2 kg suspended solids were discharged from the lagoon system per kg BOD_5 removed in winter and summer, respectively.

Deposited solids undergo aerobic/anaerobic decomposition and the rate of this decomposition is higher at the higher temperature in summer months and relatively low during the winter months (Malina *et al.*, 1971a). In lagoons with bottom sludge temperatures higher than 20°C, the breakdown of this sludge by digestion will take place (Horan, 1990), whereas Narasiah *et al.* (1990) from his past experience pointed out that at temperatures of less than 18°C anaerobic digestion is considerably slowed down. It was also reported that 40 to 60 percent degradation rate per year would take place at temperatures of 20-30°C (Adams and Eckenfelder, 1974, cited by Arceivala, 1981). Decrease in this sludge digestion rate would cause an accumulation of the solids in the lagoon bottom. Biodegradable solids loading of 76 $g/m^2.d$ (Rich, 1981) and 80 $g/m^2.d$ (Rich, 1982c) will be totally removed over an annual cycle in temperate climates. In another study, Rich (1978) stated that a loading of 110 $g/m^2.d$ appears to be a conservative design. Therefore, it was considered that a facultative aerated lagoon acts as a sedimentation tank , to some extent, because it is able to remove some suspended solids in it (Tikhe, 1975). However, it was expected that there will be a net increase in sludge volume (Arceivala, 1981). Sludge accumulation in treating domestic sewage can be estimated at 0.03 to 0.08 m^3/person.year for a cleaning interval of 5 years. And with continuing accumulation of settled solids, the liquid volume will decrease. As a consequence, power per unit volume (power level) will increase, causing the resuspension of some of these solids.

In facultative aerated lagoons, these bottom deposits will release soluble material during warm temperatures which will create high BOD_5, low suspended solids and low chlorophyll "a" in the lagoon. At suitable temperature levels, methane gas will be produced as a result of anaerobic decomposition even though high oxygen levels (5-6 mg/l) are present in the water volume above (Rich, 1982c).

Settled solids will impact the overlying liquid volume as a result of the stabilization of these solids. Oxygen will be consumed for the oxidation of the reduced inorganic compounds (e.g. disulphide) and soluble organic substances and nutrients released from the benthal zone. In addition, oxygen in the liquid volume will be consumed due to bacterial respiration in a thin, aerobic zone at the sludge/wastewater interface (Sackellares *et al.*, 1987). Hung *et al.* (1986) found that with the addition of bacterial cluture products "LLMO" (Liquid Live Micro-Organisms) the bio-conversion of accumulated sludge in aerated lagoons will be enhanced without deterioration in the final effluent quality in terms of BOD_5 or SS or other pollution parameters.

Odour nuisance caused by odour causing agents which are the products of anaerobic digestion (e.g., CH_4, H_2S) at the sludge-water interface, is not expected from facultative aerated lagoons; this is because of the absence of obnoxious gases which are prevented from being released at the air-water interface due to the presence of the aerobic layer. Loehr and McKinney (1966) observed no conventional anaerobic odours around the facultative aerated lagoon system treating wastewater from a felt roofing material plant. Because the system was not entirely aerobic, the odour around the lagoon system was that of the felt-making operation.

2.2.5.4 Layout of Aerated Lagoons

Different combinations of aerated lagoons have been used in practical applications. Malina *et al.* (1971a) recommended the use of baffled or unmixed ponds following an aerated lagoon in order to reduce the effluent suspended solids. Murphy and Wilson (1974) studied a full-scale wastewater treatment plant in Carolina consisting of one completely-mixed aerated lagoon followed by two partially-mixed lagoons, connected in series, and a non-aerated lagoon at the end of this four-cells lagoon treatment system. Oleszkiewicz and Sparling (1987) also studied a system consisting of one completely-mixed cell followed by two facultative ponds used for the treatment of domestic wastewater.

Aerated lagoons are also used in conjunction with a polishing pond. The lagoon systems studied by White and Rich (1976a) consisted of an aerated lagoon followed by a polishing pond. A two days retention polishing pond (1 m depth) following a facultative aerated lagoon (0.5 day retention time) was also reported by Thomas and Phelps (1987).

Fletcher (1980) pointed out that aerated lagoons may prove particularly suitable if applied as a stage of pretreatment to relieve the load from an overloaded oxidation pond. In a different layout, Vallée *et al.* (1989) described a treatment system in which two aerated lagoons were used for polishing effluent from a high rate aerobic treatment process treating liquid swine manure. These aerated lagoons removed 97% BOD_5, 70% COD and 88% suspended solids.

Tikhe (1975) used the term "Aerofac aerated lagoons" for those lagoon systems consisting of aerobic (completely-mixed) as the first stage and facultative aerated lagoons as the second stage. This type of processe is known as dual-power level multicellular system. The concept of these types of treatment systems has been followed in the upgrading of existing facultative lagoon systems (Rich, 1982a; 1983b; 1988). The use of two stage aerated lagoon systems have some advantages over a single facultative aerated lagoon or two or more facultative aerated lagoon in series. These advatages include:

1. Reducing the retention time required for certain degree of treatment,
2. Improving the soluble biochemical oxygen demand removal,
3. Producing a better effluent quality, and
4. Reducing the algal growth potential.

Power consumption per m^3 of wastewater was observed to be higher in a single-stage facultative aerated lagoon than in a three-stage facultative aerated lagoons system (Lovell-Smith, 1984). However, these multicellular aerated lagoon systems have complex configurations and are operated in a such a way as to make their design difficult (Rich, 1982b and c; Rich, 1983a). Rich (1988) claimed that that the use of these systems is restricted to wastewaters with BOD_5 less than 300 mg/l.

2.3 Mixing Studies

Operations conducive to the reduction of nonuniformities or gradients in composition, properties, or temperature of material in bulk are called mixing (Uhl and Gray, 1966). Such mixing is accomplished by movement of material between various parts of the whole mass. Solids suspension is probably the most common application in mixing technology (Oldshue, 1983). Ideal reactor types are batch reactors, plug flow reactors and mixed flow reactors. Knowing the retention time (Summer, 1976) and an understanding of the system mixing characteristics (Summer, 1976; Eischen and Keenan, 1985) are necessary to evaluate the treatment performance characteristics. Correct interpretation of monitoring data for process performance measurement can be obtained with knowledge of actual mixing patterns. Hydraulic bahaviour of the system can also be improved through the informations gained from mixing studies (Slade, 1991).

Idealized flow patterns, plug flow and completely-mixed, in real world process reactors operated on a continuous-flow basis are not attainable (Levenspiel, 1972; Barton *et al.*, 1983) and full-scale experience indicates that the hydrulic performance of many full-scale treatment units is highly non-ideal and numerous opportunities exist to optimize the performance of full-scale treatment units (Daigger *et al.*, 1992). Real reactors may be hydraulically characterized by comparison with one of the ideal reactors; however, real reactors such as aerated lagoons, are more complex (Barton *et al.*, 1983). Most designs in wastewater treatment approximate to ideal plug flow or ideal completely-mixed flow, though deviations from ideality are considerable. Fluid channelling or recycling as well as the occurrence of stagnant regions causes nonideal flows in reactors, which adversely affect process performance (Levenspiel, 1972). Information about flow regimes is useful for designing reactors.

Maintaining the biomass in suspension as well as distributing oxygen and substrate throughout the liquid volume in a biological wastewater treatment can be achieved by good mixing. With this, also, dilution of toxic materials occurs and the build up of inhibitory concentrations of substrate in the reactor will not take place. However, the degree of mixing required for oxygen dispersion is considerably less than that required for complete mixing. The zone of complete mixing is smaller than the zone of oxygen dispersion at similar power levels (Benefield and Randall, 1980). In contrast, the degree of mixing affects the degree of pollutants removal negatively. This is demonstrated by the Wehner-Wilhem equation for first-order removal kinetics. High mixing conditions will cause wash-out of bacteria and substrate out of the system before completing substrate conversion. In plug-flow systems the substrate remains in the reactor for one hydraulic retention time during which the bacteria and substrate are effectively contacted together achieving high removal efficiency.

Facultative aerated lagoons are characterized by their hydraulic flow patterns, which are normally neither plug-flow nor completely-mixed flow and are described as non-ideal flow reactors. The low power levels at which facultative aerated lagoons operate, to permit solids settling and promote anaerobic decomposition, will reduce the removal potential of the liquid volume due to the low suspended biomass at low mixing intensities.

Narasiah *et al.* (1987) stated that the maximum depth of the lagoon is about 4.0 meters to maintain completely-mixed conditions by surface aerators whereas with diffused aeration depths up to 6.0 meters can be reached (Narasiah *et al.*, 1987).

2.3.1 Residence Time Distribution of Fluids in Reactors

The theory of residence time deals with particles that enter, flow through, and leave a system (Nauman and Buffham, 1983). The time taken by every single element of the fluid in the reactor is the rationale behind residence times distribution (RTD) of the flowing fluid. Therefore, residence time distribution analysis was developed to indicate how long a fluid element remains in the mixing vessel. The difference in the time taken to exit from a reactor by the elements of fluid which entered together cause the spread of residence time of the flow elements and this is described by an age distribution function. Accordingly, residence time distribution is a technique used to describe mixing conditions in a reactor and indicates how long a fluid element remains in the mixing vessel.

Stimulus-response experiments are an easy and direct method for the determination of the residence time distribution of the fluid. In this method, a known concentration of a tracer (stimulus) is introduced with the influent stream and the concentration of this tracer in the outlet (response) is measured. Thus the system is disturbed and its response is observed and the desired information about the system can be obtained.

2.3.2 Tracer Tests

Hydraulic characterization can be achieved by performing tracer tests. Energy conservation and efficient and cost-effective operation of wastewater treatment systems depends on the availability of information obtained from mixing studies in a reactor (Barton *et al.*, 1983).

Hydraulic characteristics as well as material and heat balances of any process can be determined by performing a tracer test. It can be said that any conservative material that can be detected and which does not disturb the flow pattern in the vessel can be used as a tracer. Tracers used may be dyes, chemicals, radioactive isotopes or bacteriophage. The most common tracer types used in wastewater treatment systems are fluorescent dyes and lithium compounds (Barton *et al.*, 1983). Fluorescent dyes can be measured by fluorometry whereas lithium compounds are measured by flame atomic absorption spectrophotometry.

Tracers are injected at the inlet end of the reactor with one of the signal types, random signal, periodic signal, step (continuous) signal, or a pulse (or impulse or slug) signal (Levenspiel, 1972). The last two are considered to be simplest to treat. There must be no tracer material in the reactor prior to the test. Ideally, input of the tracer, for pulse signal, should be instantaneous (not drown out). However, in practice this is difficult to attain, especially for large reactors that require large amounts of tracer. Therefore, the injection of the tracer should not take more than ten percent of the hydraulic retention time (Barton *et al.*, 1983).

Levenspiel (1972) and Barton *et al.* (1983) listed the primary concerns that characterize the selection of conservative tracers:

a. The tracer must be truly conservative (inert) in the system of interest;
b. The tracer must be measurable by some accurate and sensitive means;
c. The tracer must not be toxic; and
d. The tracer must not be overly costly.

An inert tracer means that it should not biodegrade during its stay in the reactor or react with any material present in the vessel. The tracer to be used should not have any unusual activities such as adsorption at walls, disappearance by reaction (Levenspiel, 1972). Barton *et al.* (1983) pointed out that tracer adsorption capacity characteristics should be determined and should be acceptable. The possibility to detect very low concentrations of the tracer in the effluent stream is important. The relative cost of the various tracer options should be considered in addition to the other concerns when selecting a tracer material (Barton *et al.*, 1983). Tracer and fluid densities should be similar in the reactor. The change in magnitude of the stimulus should result in a corresponding proportional change in the magnitude of the response. Invisible tracers are favourable (Horan, 1990).

2.3.3 Internal and Exit Age Distributions

Residence time distributions (RTD) are a measure of the distribution of ages of the fluid either in the reactor or leaving the reactor. The former is the Internal Age Distribution (IAD) and the latter the Exit Age Distribution (EAD). **I** and **E** are the normalised forms of both distribution, respectively. The mathematical equation defining these functions and the relationship between them are presented below:

$$\int_0^\infty I\,dt = 1 \tag{2.23}$$

Idt is the fraction of the tracer with age between t and t+dt.

$$\int_0^\infty E\,dt = 1 \tag{2.24}$$

Edt is the fraction of tracer material in the effluent of age between t and t+dt.

$$\frac{E}{I} = 1 \tag{2.25}$$

The normalized Exit Age Distribution for a pulse signal with a tracer is called a C curve or **C**. Therefore, both the **E** and **C** distributions are the result of a delta function or pulse tracer introduction into a reactor system; they only differ in the way the independent variable, time, is defined. For a step signal the exit age distribution curve is called the **F** curve. For pulse tracer input:

$$C = E \tag{2.26}$$

$$F = \int_0^t E\,dt \qquad \text{or} \qquad \frac{dF}{dt} = E \tag{2.27}$$

$$F = \int_0^t C\,dt \qquad \text{or} \qquad \frac{dF}{dt} = C \tag{2.28}$$

The **C** diagram is commonly used in interpreting tracer experiments because most tracer experiments involve the injection of an instantaneous tracer pulse (Barton *et al.*, 1983).

For a completely-mixed flow the concentration of the tracer, C, at any time, t, in the effluent can be determined by the equation developed from the mass balance of the tracer. For pulse tracer input the following equation is obtained:

$$C(t) = e^{-t} \tag{2.29}$$

and for step tracer input the following equation is obtained:

$$F(t) = 1 - e^{-t} \tag{2.30}$$

Equation 2.30 can also be derived by solving Equations 2.28 and 2.29 simultaneously.

2.3.4 Nonideal Flow Models

Real world mixing vessels do not provide ideal examples of complete mixing or plug-flow. This type of flow within reactors is known as non-ideal flow and the reactor itself is called a non-ideal rector. Various models have been used to characterize these non-ideal flows including point indices, dispersion model, tanks-in-series model and combined models which are discussed below.

i) Point Indices

This is a simple and quick method used to provide an initial estimate of the types of mixing present in the reactor. These are determined from the Exit Age distribution curve. Some of them are presented below:

t_{10} = 10th percentile rank in the RTD, which is the time required for 10% of the tracer to pass through the system,

t_p = peak residence time (time of peak concentration) of the RTD corresponds to the model concentration in the distribution,

t_{90} = 90th percentile rank in the RTD, which is the time required for 90% of the tracer to pass through the system,

t_{min} = time to the initial appearance of the tracer in the system outlet,

t_{max} = time to the last measurable appearance of the tracer in the system outlet,

t_{90}/t_{10} = Morrill index.

The Morrill index is a percentile index used as a measure of dispersion or deviation from plug flow. High Morrill index (>1, the value for ideal plug flow) means that more dispersion is taking place. Much higher values of the Morrill index (i.e. >10) indicate mixing is approaching a complete mixed reactor. The Morrill index is not a very accurate indicator of the hydraulic character of treatment systems and may not appropriately represent non-ideal mixing conditions in real systems (Barton *et al.*, 1983). However, it is useful for initial characterization of a treatment system (Barton *et al.*, 1983).

This central-tendency analysis uses only a few properties of the residence time distribution to determine the hydraulic character of the system and hence does not completely characterize non-ideal hydraulics. However, such analysis is still useful in hydraulic characterization. Therefore, involving the full RTD curve in a more complete modelling technique is generally recommended to provide a clearer understanding of mixing conditions (Thirumurthi, 1969; Barton *et al.*, 1983).

ii) Dispersion Model (Dispersed Plug Flow)

The dispersed plug flow model or dispersion model assumes complete mixing in the horizontal and vertical directions with some backmixing along the longitudinal direction of flow (Sackellares, 1985).This condition implies that there exist no stagnant pockets and no gross bypassing or short-circuiting of fluid in the vessel (Levenspiel, 1972). The degree of dispersion depends upon factors such as power level and reactor geometry. The dispersion model usually represents flows that do not greatly deviate from plug flow. Dispersion number, D/uL, in the dispersion model can be determined from the following equation which incorporates the variance of C curve (σ):

$$\sigma_t^{\,2} = \frac{\sigma^2}{\bar{t}^2} = 2\frac{\mathrm{D}}{\mathrm{uL}} - 2\left(\frac{\mathrm{D}}{\mathrm{uL}}\right)^2\left(1 - e^{-\frac{\mathrm{uL}}{\mathrm{D}}}\right) \tag{2.31}$$

where, D = longitudinal or axial dispersion coefficient,

u = fluid velocity,

L = characteristic length of travel path of a typical particle in the reactor, and

(D/uL) = vessel dispersion number, and is the parameter which measures the extent of axial dispersion.

$$\sigma^2 = \frac{\int_0^\infty (t-\bar{t})^2 C\,dt}{\int_0^\infty C\,dt} = \frac{\int_0^\infty t^2 C\,dt}{\int_0^\infty C\,dt} - \bar{t}^2 \tag{2.32}$$

By using Equations 2.31 and 2.32, the dispersion number can be estimated by trial and error. This dimensionless number varies from zero to infinity. Zero and infinity are for the two extremes; plug flow and completely-mixed flow, respectively.

The existence of stagnant pockets and gross bypassing or short-circuiting of fluid in the vessel limits the use of the dispersion model. Therefore, with the presence of several distinct flow regimes which have different degrees of dispersion, a separate tracer response curve for each zone is needed to obtain an accurate evaluation of the dispersion characteristics of the system.

iii) Tanks-in-Series Model

The tanks-in-series model is another single-parameter model widely used to represent non-ideal flow. In tanks-in-series modelling, the hydraulics of a partially-mixed reactor or treatment system may be simulated by a number of ideal completely-mixed flow reactors connected in series with a total volume equal to the volume of the system being simulated. These tanks are called the model segments which are not separated physically. An ideal completely-mixed flow reactor is represented by one reactor, while low mixing is represented by a large number of these tanks. The response of a pulse tracer input into a reactor with only one tank is represented by:

$$C(\theta) = e^{-\theta} \tag{2.33}$$

The tracer concentration in the effluent from the *i*th tank is:

$$C_i(\theta) = \frac{C_0}{(N-1)!}(N\theta)^{i-1} e^{-N\theta} \tag{2.34}$$

where, $\theta = t / \bar{t}_N$,

$\bar{t}_N$ = hydraulic retention time in each tank, and

N = number of tanks.

In this model only one parameter is used, which is the number of tanks in the chain. A high number of complete-mixed tanks connected in series means less mixing takes place in the entire system. The extension of this concept to a large number (i.e. ten or more) tanks connected in series results in an equivalent plug-flow reactor. The number of tanks can be calculated from the variance of the RTD curve according to:

$$N = \frac{1}{\sigma_t^{\,2}} = \frac{\bar{t}^2}{\sigma^2} \qquad (2.35)$$

In a tanks-in-series model the liquid cannot move upstream; this is not true for a dispersion model. For large N, the RTD curve becomes increasingly symmetrical and approaches the normal curve of the dispersion model, and a comparison of these two curves allows us to relate the tanks in series and dispersion models (Levenspiel, 1972). However, it is clear that the dispersion model is more accurate for flows patterns not greatly different from plug flow, whereas the tanks-in-series model is more accurate for flow patterns not greatly different from a completely-mixed reactor.

iv) Combined Models

Nonideal flows have been described by several models. However, these models are simple and only consider the deviation from ideal plug-flow or mixed-flow. In real reactors, these nonideal flows may arise from the presence of different regions which are either dead volume, plug-flow, mixed-flow or dispersed plug-flow or a combination of some or all of them. These regions are interconnected by bypass, recycle or crossflow. Therefore, simple processes are not suitable to describe such actual reactor flows. The models described above, which are based on only one parameter (e.g., simple tanks-in-series and axial dispersion modelling approaches to hydraulic characterization), are unable to describe these flow patterns. Therefore, these simple model limitations require a more complicated models that take into account such presence and the complications of nonideal flow in real reactors.

Models that simulate real reactor hydraulics are known as combined, mixed, or multiparameter models. Different combined models have been developed which vary from each other with respect to accuracy and complicity. The development of combined models is important for chemical reactors in terms of hydraulic characterization (Barton *et al.,* 1983). In these mixed models it is visualized that various flow regions are connected in series or in parallel.

Rapid observation of influent material in the effluent indicates short-circuiting. This short-circuiting could be attributed either to a very high degree of mixing or to the presence of dead regions. Dead regions are portions of the reactor in which the flow through it is very slow, and ideally is considered to be completely stagnant.

Different forms of combined models have been discussed elsewhere (Levenspiel, 1972). The simplicity of a combined model is an important criterion. Complicated models described by increase in the number of the parameters affect their usefulness. Backmixing between segments has also been incorporated into some models to allow the simulation of backwards dispersion. Models that utilize backmix flows are found to be of significant use in the characterization of wastewater treatment plant hydraulics (Barton *et al.*, 1983).

RESEARCH AIMS AND METHODOLOGY

Many studies reviewed in the literature survey (Chapter 2) have investigated the contribution of facultative aerated lagoons as part of a dual-power multicellular aerated lagoon system or other system, and not simply as a single unit secondary treatment process. The facultative aerated lagoon treatment process can reduce, and perhaps eliminate, the use of high-maintenance, mechanically complex secondary treatment systems. This type of aerated lagoon, which is an economical alternative method of treating wastewater to produce an effluent of a reliable quality that complies with discharge standards, may well have important implications for the future in many countries.

Although many facultative aerated lagoons have been constructed for the purpose of low cost efficient treatment of wastewater, the design of these lagoons is still not well understood and is not based on a through analysis of the relationship between operating variables and lagoon performance. Accordingly, laboratory evaluation of the effects of combinations of controllable parameters on a facultative aerated lagoon's function was the main objective of this research.

Performance and efficiency assessments of facultative aerated lagoons when subjected to the effects of controllable external factors, such as organic loading rate and power level under different climatic conditions, i.e.; different levels of temperature, were considered in this study. Three levels of organic loading, 20, 33 and 62 g $BOD_5/m^3.d$ and four input power levels, 0.25, 0.5, 1 and 2 W/m^3, for two levels of temperature, 20° and 30°C, were the main parameters selected in this research and various combinations of these variables constituted the different experimental runs in the experimental work. An 81 liter laboratory-scale reactor with diffused aeration system was used as facultative aerated lagoon.

In order to achieve the stated objective, the following tasks were undertaken:

- The aeration system was described in terms of the rate of oxygen transfer capacity, by performing the reoxygenation capacity test for different air flow rates assigned, and related to the power level input into the reactor

- Hydraulic characteristics of the reactor for the various combinations of power level and organic loading were identified at each flow condition by performing the appropriate tracer tests, using a suitable tracer. These included:
  - mean hydraulic residence time of the process;
  - dispersion number;
  - number of tanks in the unequal-tanks-in-series model; and
  - volume of the deadspaces present in the reactor.

- A simulation model was developed to simulate the reactor hydraulics and the relationship between the parameters of this simulation model and the performance parameters of the model facultative aerated lagoon was determined.

- BOD_5 and COD of influent and effluent were tested on several samples collected during each test run.

- Change of solids concentration between influent and effluent for each experimental run was considered, to provide a design guide for a sedimentation clarifier or polishing pond, if required, following facultative aerated lagoons. The solids contents were tested for the following:

- total solids, fixed and volatile.
- suspended solids, fixed and volatile.

- Mathematical relationship for the determination of the mean solids retention time (SRT) was developed and the mean retention time of the solids in the lagoon for each run was determined. An empirical relationship between solids retention time:hydraulic retention time ratio (SRT/HRT) and both power level and organic loading rate was developed.

- Other physico-chemical and biological parameters were examined for each test run by conducting the following laboratory analyses:
  - nitrogens (TKN, ammonical-nitrogen);
  - phosphorus;
  - pathogens removal (*E. coli*, and *faecal streptococci*);
  - chlorophyll "a" concentrations; and
  - pH and alkalinity.

- Identification of optimum combinations of controllable parameters used in the research.

- Data from the experimental runs were analysed to determine the effects of controllable parameters on the process performance mathematically.

- Effluent quality from such treatment process was assessed with respect to reuse and disposal standards and the need for further treatment was checked.

EXPERIMENTAL METHODS AND DESIGN

A description of the experimental work is presented in the following sections, encompassing a description of the laboratory model and of the procedures followed and concluding with the data required to achieve the objectives of this study.

The process operating parameters provided the necessary interrelationships for assessment of the performance and functional efficiency of the operation. For two levels of temperatures (20°C and 30°C), the laboratory-scale facultative aerated lagoon was investigated for its performance under the combined effects of different power levels (0.25, 0.5, 1 and 2 W/m^3) and different organic loadings (20, 33 and 62 $g/m^3.d$).

Two FORTRAN programs were developed for simulation and calculations for determination of hydraulic characteristics of the reactor under the conditions of each run. More details of the these two programs are presented in Section 4.3.2.

Aerators, in any aerobic wastewater stabilization system, perform two basic functions: (i) they must provide for transfer of oxygen into the mixed liquor in sufficient quantity to satisfy the oxygen demand rate of the microorganisms responsible for decomposition of organic matter and, (ii) produce sufficient mixing of the liquid contents for the contact of dissolved oxygen, organic substrate, and microorganisms to take place (NCASI, 1971).

In this research, the aerators were tested for oxygen transfer rate (at 20°C and 30°C), at the various power levels utilised in this research, and to ascertain the degree of mixing provided in the reactor, for each set of power level and organic loading. These two tests are described in Sections 4.2 and 4.3, respectively.

4.1 The Laboratory Model

The aerated lagoons used in this research were aerated lagoons of a laboratory-scale, shown in Plate 4.1. A flowchart of the process is presented in Figure 4.1, whereas Figure 4.2 shows the schematic diagram.

a) Feed Tank: The feed tank used was an open top circular tank of approximately 0.2 m^3 capacity. The contents of the feed tank were gently mixed by means of a stirrer to prevent settlement of solids in the tank and to give a homogeneous feed. The feed wastewater was settled municipal wastewater brought to the laboratory from a local wastewater treatment plant.

b) Feeding Pump: variable flow peristaltic pumps (Model 302S/RL, Watson-Marlow Ltd. England) were used to transfer the wastewater from the feed tank to the reaction vessels through soft rubber tubes.

c) Reaction Vessels: Three 0.6 m by 0.45 m rectangular tank reaction vessels with sloping trapezoidal sides were used in this research. The side walls' slope of these inverted truncated pyrmid shape reaction vessels was 15:1 (vertical:horizontal), therefore, the base area was smaller than the surface area. The effective volume of each of the reaction vessels was 0.081 m^3 and a liquid water depth of 0.3 m. The average cross-sectional area was 0.27 m^2. Six artificial daylight fluorescent tubes (1.75 m, 75/85 W manufactured by Thorn) were

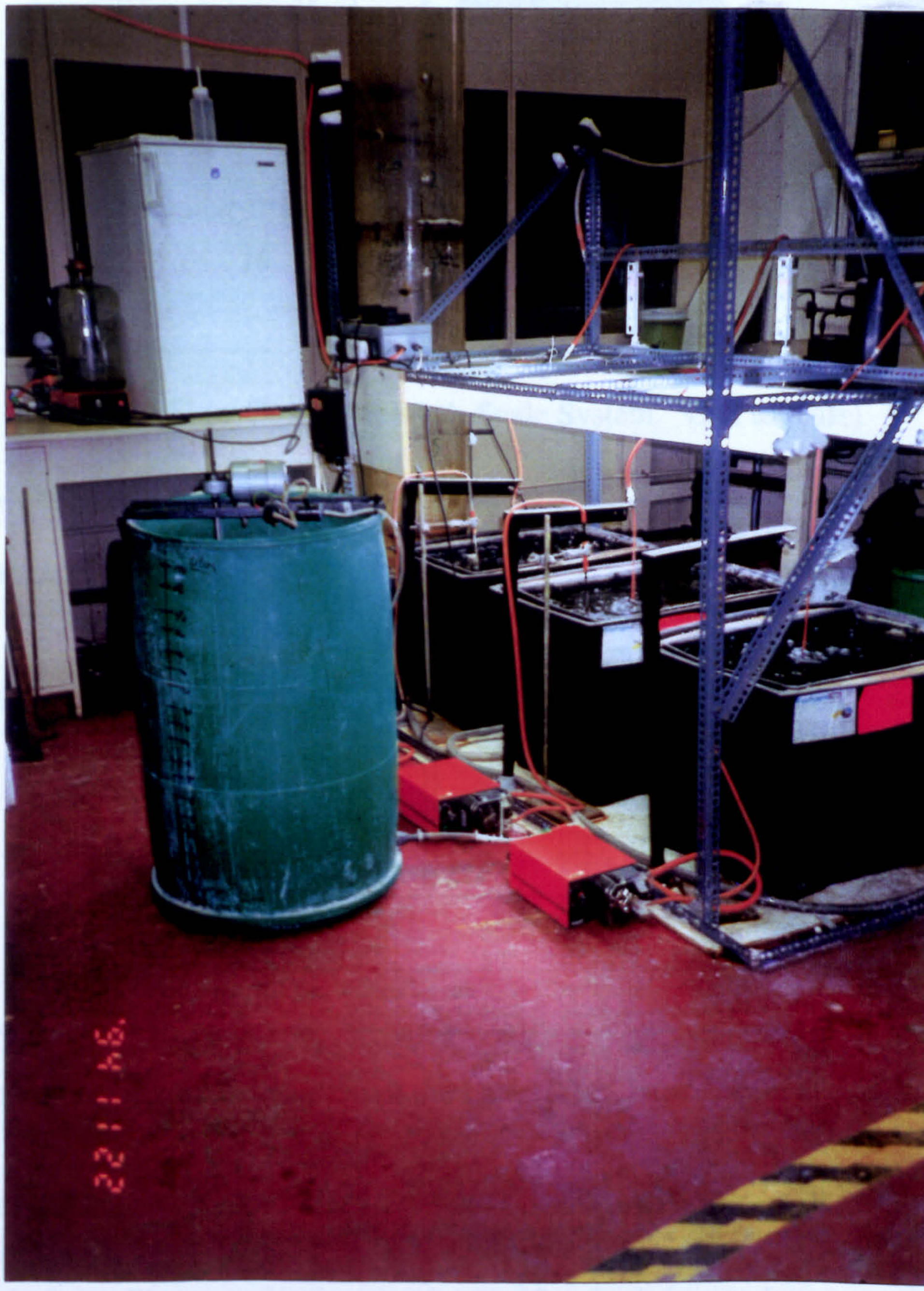


Plate 4.1 Aerated lagoons used in Study

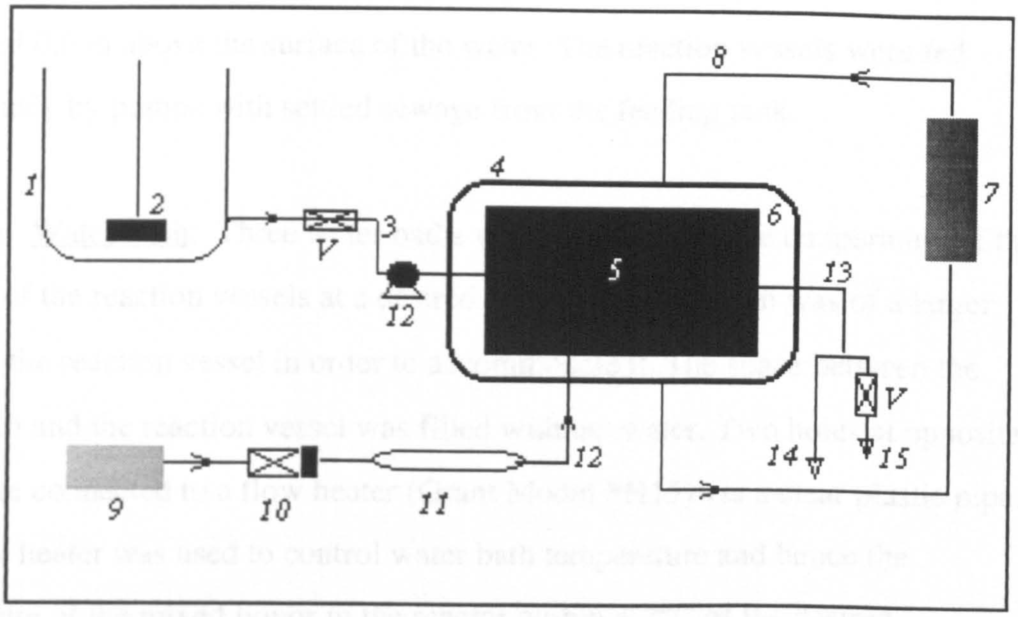


Figure 4.1: Flow Chart of the Lab-Scale System (not to scale)

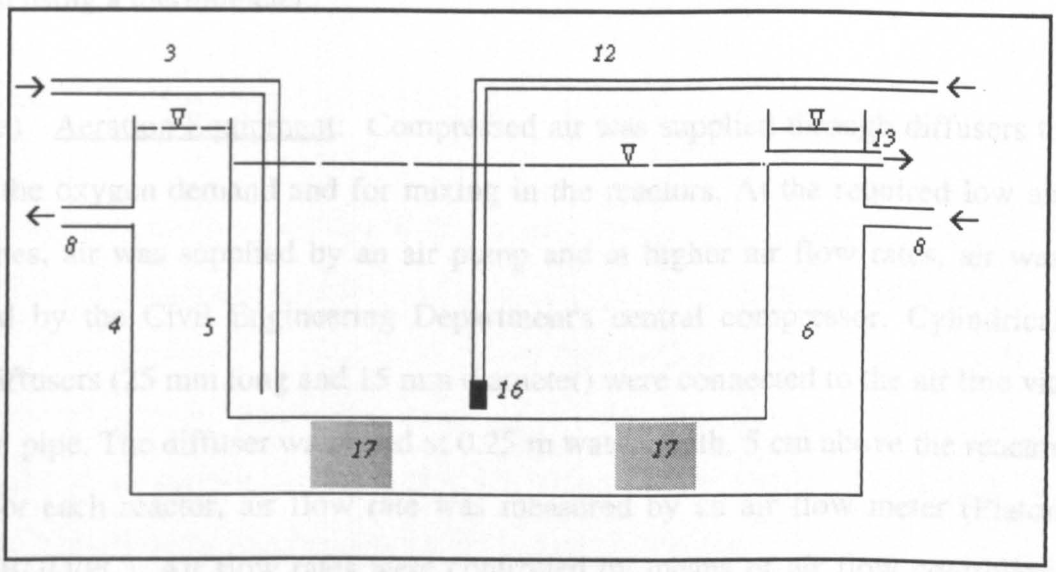


Figure 4.2: Schematic Diagram of the Lab-Scale System (not to scale).

1 = Feed Tank	10 = Air Flow Meter
2 = Stirrer	11 = Air Flostat
3 = Influent Pipe	12 = Air Injection Line
4 = Water Bath	13 = Effluent Pipe
5 = Reaction Vessel	14 = Drainage
6 = Water	15 = Effluent Sampling Point
7 = Flow Heater	16 = Air Diffuser
8 = Hot Water Line	17 = Concrete Cube
9 = Compressed Air Source	

suspended 0.6 m above the surface of the water. The reaction vessels were fed continuously by pumps with settled sewage from the feeding tank.

d) Water Bath: Three water baths were used to keep the temperature of the contents of the reaction vessels at a desired level. The water bath was of a larger size than the reaction vessel in order to accommodate it. The space between the water bath and the reaction vessel was filled with tap water. Two holes at opposite sides were connected to a flow heater (Grant Model FH15) via a clear plastic pipe. This flow heater was used to control water bath temperature and hence the temperature of the mixed liquor in the reactor within ± 2°C of the desired temperature. The temperature of the contents of the reaction vessels was regularly checked using a thermometer.

e) Aeration Equipment: Compressed air was supplied through diffusers to satisfy the oxygen demand and for mixing in the reactors. At the required low air flow rates, air was supplied by an air pump and at higher air flow rates, air was supplied by the Civil Engineering Department's central compressor. Cylindrical stone diffusers (25 mm long and 15 mm diameter) were connected to the air line via a plastic pipe. The diffuser was fixed at 0.25 m water depth, 5 cm above the reactor base. For each reactor, air flow rate was measured by an air flow meter (Platon Model BGD/PC). Air flow rates were controlled by means of air flow controllers (Platon Flostat Model MN/AL) which were designed to give a constant air flow rate whenever the outlet pressure changed, due to clogging of the diffusers.

A summarised description of the various parts of the rig used in this research is given in Table 4.1

Table 4.1: Summarised Sizes and Descriptions of the Laboratory-Scale system

Part	Description
Feed Tank	0.2 m^3 capacity, with gentle mixing
Feeding Pumps	peristaltic pumps
Reaction Vessels (three vessels)	L=0.6 m, W=0.45 m, D=0.37 m, liquid depth=0.3 m, liquid volume=0.081 m^3.
Aeration Equipment	cylindrical diffuser (L=25 mm, Dia.=15 mm), compressed air and air flow controllers and meters
Temperature controlling system	water paths (L=0.75 m, W=0.5 m, D=0.5 m) and flow heater

4.2 Rates of Oxygen Supplied and Oxygen Transfer

Oxygen-transfer capacities of the aeration system are an essential parameter in the rational design and operation of an aerated biological process (Conway and Kumke, 1966; Ford, 1972). This is to ensure that sufficient oxygen is supplied to the microorganisms to satisfy their demands in oxidizing the organic matter in the wastewater to be treated. For this reason the aeration system used was tested for its ability to transfer sufficient oxygen for the treatment process.

The non-steady state reoxygenation of tap water approach was used, with the dissolved oxygen being measured. This method involves the deoxygenation of clean water with sodium sulfite (Na_2SO_3), 117 g/m^3 of reactor volume if initial dissolved oxygen concentration in the water is 10 mg/l. 8.05 g/l cobaltous chloride (Co $Cl_2.6H_2O$), cobalt-ion concentration of 2 mg Co^{++}/l, in solution form was used as a catalyst in the deoxygenation process. The oxygen-free water was then reoxygenated using the aeration system described, and increase in dissolved oxygen residual with time measured using an oxygen probe and meter and automatic recorder. The measured rate was then interpreted and used in the determination of aerator performance. For chemical reaction equations describing the reactions involved and the mathematical equations used for this test see Appendix A.

4.3 Hydraulic Characterization of Reactor

The hydraulic characterization of the reactor is dealt with in the following two sections. The first describes the tracer test used and the second the FORTRAN computer programs developed. Twelve tracer tests were applied in this research for each experimental run formed by the different sets of the two relevant parameters: organic loading rate and power level.

4.3.1 Tracer Tests

The objective of the tracer tests was to obtain data for use in the determination of the hydraulic characterization of the basin. Because the effectiveness of an Aerated stabilization basin is measured by its ability to remove pollutants from the waste stream, the correlation between mixing regime and treatment performance must be determined (Barton *et al.*, 1983).

A pulse tracer injection method was used and lithium, as lithium chloride (LiCl), was chosen as a tracer in these series of tracer studies. The ease of use and measurement, availability and non-toxicity to biomass, are among the advantages of using lithium among other tracers. For the characterization of high retention reaction vessels, the length of time taken by this test was a disadvantage.

Sampling was continued for more than two retention times, with more frequent sampling at the beginning and less at the later stages of testing. In this research, sampling periods were in the range of 7 to 25 days. The "HYDCH" FORTRAN program, described in the following section, was used in analysis of tracer test data.

4.3.2 FORTRAN Programs

Two FORTRAN computer programmes were developed. The first was "HYDCH" (Appendix 2a), while "TRSIM" (Appendix 2b), was the second.

"HYDCH" was used to analyse tracer test data and the outputs are the parameters describing the hydraulics of the reactor. The hydraulic characteristics generated from using "HYDCH" were:

i) Mean hydraulic retention time, $\bar{t}$.

ii) Dispersion number, d.

iii) Dead spaces volume, DV.

iv) Number of tanks in the tanks-in-series model, N.

The mathematical equations used for the determination of the above descriptive parameters were as follows:

$$\bar{t} = \frac{\int_0^\infty tcdt}{\int_0^\infty cdt} \tag{4.1}$$

$$\sigma^2 = 2d - 2d^2(1 - e^{-\frac{1}{d}}) \tag{4.2}$$

$$\sigma^2 = \frac{\int_0^\infty (t-\bar{t})^2 C dt}{\int_0^\infty C dt} = \bar{t} = \frac{\int_0^\infty t^2 C dt}{\int_0^\infty c dt} - \bar{t}^2 \tag{4.3}$$

$$DV = (1 - \frac{\bar{t}}{t_C})V \tag{4.4}$$

$$t_C = \frac{V}{q} \tag{4.5}$$

$$N = \frac{1}{\sigma^2} \tag{4.6}$$

Where, σ^2 = Variance of the C-curve,

V = Volume of the reaction vessel; m^3, and

q = Wastewater flow rate; l/hr.

Due to the use of tracer test data collected up to two hydraulic retention times from the start, the values from Equation 4.1 were corrected.

FORTRAN program "TRSIM" was used to determine the number and sizes of tanks as an imaginary series of segments in the reactor, in the non-equal-tanks-in-series model with backmixing between the segments. The number and sizes of these segments were determined by matching the predicted tracer output response curve with the profile of the simulation model. Description of the basic equations and the procedure used in the analysis of the data is presented in Chapter 5.

4.4 Experimental Procedure

Evaluation of a wastewater treatment process under certain working conditions involves an intensive programme of sampling and analysis of influents, unit process contents and process effluents.

4.4.1 Experimental Design

To study the performance of facultative aerated lagoons, three laboratory-scale facultative aerated lagoons were used and the performance was studied for the organic loadings selected (20, 33 and 62 g BOD_5 /m^3.d). Tests were carried out at the different power levels (0.25, 0.5, 1.0 and 2.0 W/m^3) that may be applied in facultative aerated lagoons. For two levels of temperatures, all possible combinations of these three parameters were factorially designed resulting in the need for twenty four experimental runs. A run number was assigned for each set of these parameters and the experimental runs were randomly planned. Table 4.2 shows the assigned levels of the parameters selected in this research, for each test run.

4.4.2 Reactor Startup

For the first three runs, the reactors were seeded with activated sludge from a domestic wastewater treatment plant. For the succeeding runs, the starting contents of the reactors were those of the preceding run. According to this practice, no seed was used for these runs, since the biological mass had already developed in the reactor.

Table 4.2 Combinations of the Independent Parameters for the Experimental Runs

Run	Temperature (°C)		Power Level (W/m^3)				OrganicLoading(g/m^3.d)		
No.	20	30	0.25	0.5	1.0	2.0	20	33	62
1	X		X						X
2	X		X					X	
3	X		X				X		
4	X			X					X
5	X			X				X	
6	X			X			X		
7	X				X				X
8	X				X			X	
9	X				X		X		
10	X					X			X
11	X					X		X	
12	X					X	X		
13		X	X						X
14		X	X					X	
15		X	X				X		
16		X		X					X
17		X		X				X	
18		X		X			X		
19		X			X				X
20		X			X			X	
21		X			X		X		
22		X				X			X
23		X				X		X	
24		X				X	X		

Each time, the biomass was allowed to grow and acclimatize to the new conditions of the run. This usually took between ten and fifteen days. Each test run was then continued for at least ten days after steady-state conditions were established.

The steady-state condition was considered to be reached when the difference between consecutive effluent BOD_5 and the mixed liquor suspended solids, MLSS concentrations were within ± 10-15 %. During this period a systematic monitoring of the process was carried out.

The biological reactor was gradually adjusted to the new conditions for a new run whenever temperature, loading rate or power level were changed.

4.4.3 Operation Monitoring and Sampling Techniques

The three levels of organic loading with the four levels of power input for each of the two levels of temperature were all arranged in a full factorial design. Twenty-four experimental runs were planned to fulfil the research objectives.

For each test run, the set of controllable operation parameters was assigned, applied and monitored. Liquid temperature was checked using a thermometer at least three times a day.

The air flow rate was set for each test run to give the desired power level in the reactor, using the following equation:

$$Q_a = \frac{P \times V}{3.9 \times Log[(h + 10.366)/10.366]} \tag{4.7}$$

Where, Q_a = Air flow rate; l/ m,

P = Power level; W / m^3,

V = Volume of the reactor; m^3, and

h = Depth of diffuser; m.

Air flow rate was checked daily by reading the air flow meter.

Organic loading rates, volumetric (L_V) or areal (surface) (L_A), can be controlled by changing the reactor volume, influent concentration or by changing the retention time. In this research, retention times, and hence influent flow rates were varied to obtain the anticipated organic loadings for each test run. The flow rate was determined for this particular research according to the following equations:

$$q = \frac{41.667 \times V \times L_V}{S_0} \quad \text{or} \tag{4.8}$$

$$q = \frac{138.889 \times V \times L_A \times 10^{-4}}{S_0} \tag{4.9}$$

Where, q = Liquid flow rate; l/hr,

V = Volume of the reactor; m^3,

L_V = Volumetric organic loading rate; g/m^3.d,

S_0 = Influent wastewater BOD_5 concentration; mg/l, and

L_A = Areal organic loading rate; g/ha.d.

According to this practice, as the reactors were operated at a constant volume, different retention times were assigned to runs of different loading rates. The liquid flow rate, q, was monitored twice a day and was measured by establishing the time required to fill the 250 millilitre cylinder, where:

$$q = \text{Volume/time.}$$

Grab samples, taken at regular time intervals, were collected simultaneously from both the influent and effluent streams, as well as from the mixed liquor contents. Samples collected were immediately split into a filtered and unfiltered fraction. Whenever composite samples were needed, grab samples were taken and stored in a cold room (4°C) until the final sample of the period was taken.

4.4.4 Performance Examination

Settled sewage was used as an influent for all test runs. The influent wastewater was a primary treated wastewater brought in 20 litre containers from Morpeth Treatment Works, Northumberland, every second working day (three days a week). The required volume for one day flow was taken from the containers brought on that day and the rest of the containers stored in a cold room for use on the following day.

Storage effects on the change in influent BOD_5 and COD concentrations between the fresh sample and just before refilling the feeding tank with the next fresh sample were evaluated by analyzing three samples. The results of these three tests are presented in Table 4.3 and reveal an average reduction of 4 percent for both BOD_5 and COD over two days.

Table 4.3: Two Days' Storage Effect on BOD_5 and COD

Parameter	Sample Age	Test Number		
		1	2	3
BOD_5 (mg/l)	Fresh	188	230	255
	2-days	180	221	242
COD (mg/l)	Fresh	466	607	663
	2-days	452	581	638

Pollutant concentrations in the influent varied from run to run, though no extreme variations were observed. Variations were due to seasonal differences, depending on the time the influent samples were brought in. Influent constituent concentrations are set out Table 4.4

Effluent samples were obtained by opening a valve in the outlet line.

Physico-chemical analysis and biological examination of sample quality were carried out in accordance with the procedures outlined in Standard Methods for the Examination of Water and Wastewater (APHA *et al.*, 1989).

During each run, several samples were collected and tested at the time of collection. All samples collected were analysed for the following physico-chemical parameters:

- Biochemical oxygen demand (BOD_5).
- Chemical oxygen demand (COD).
- Nitrogen, ammonia (NH_3) and total Kjeldahl nitrogen (TKN).
- Phosphorus.
- Total solids, total and volatile.
- Suspended solids, total and volatile.

Table 4.4: Influent Wastewater Characteristics Used During the Project Period.(all values are expressed in mg/l, except pH)

Constituent	Concentration
BOD_5 (filtered)	88-160
BOD_5 (unfiltered)	188-265
COD (filtered)	256-325
COD (unfiltered)	466-670
TS	660-894
SS	103-140
TVS	251-286
VSS	79-127
TKN	38-54
Ammonia	27-43
P (filterable orthophosphate)	6.4-9.5
P (total orthophosphate)	7.1-10.5
pH	7-7.5

- pH and alkalinity.
- Chlorophyll "a" concentration.
- *Escherichia. coli (E. coli)* and *faecal streptococci* densities.

4.4.4.1 Biochemical Oxygen Demand (BOD_5)

The 5-day biochemical oxygen demand (BOD_5) test is the most widely used parameter of organic pollution applied to both wastewater and surface water. It provides a measure of the dissolved oxygen utilized by microorganisms during stabilization of biodegradable organic matter under specified conditions. It is assumed that most biodegradable organic matter contained in a municipal wastewater sample will be stabilized biologically. The BOD_5 test is, therefore, a direct measure of the oxygen requirements to stabilize the wastewater and is an indirect measure of the biodegradable organic matter contained in the sample. However, the BOD_5 is a time-consuming test which required 5 days for completeion. Gough *et al.* (1990) developed a prediction equation for BOD_5 in dairy plant processing wastewater treated in aerated lagoons, including COD, TSS and TS.

The 5-day biochemical oxygen demand (BOD_5) tests were carried out on both filtered and unfiltered samples to give soluble and total BOD_5, respectively.

4.4.4.2 Chemical Oxygen Demand (COD)

The chemical oxygen demand (COD) test is also used to measure the content of organic matter of both wastewater and natural water. This test is based on the principle that most organic compounds can be oxidized to carbon dioxide (CO_2) and water (H_2O) by strong chemical oxidizing agents in an acidic medium at elevated temperatures. The measurement represents the oxygen that would be needed for oxidation of biodegradable organics by aerobic microorganisms to carbon dioxide and water.

The closed reflux method was adopted, using sulphuric acid and an excess of standardized potassium dichromate. During the reflux period, the chemically oxidizable organic material reduces a stoichiometrically equivalent amount of dichromate, the remainder of which is measured by titration with standard ferrous ammonium sulfate. The amount of dichromate reduced is a measure of the amount of organic material oxidized.

To ensure accuracy, each sample was tested three times for COD and an average of these three results was used.

4.4.4.3 Nitrogen and Phosphorus

Nitrogen and phosphorus are in most cases the important nutrients necessary for biological growth. Nitrogen and phosphorus are required for wastewater treatment. On the other hand, removal or reduction of nitrogen and phosphorus in wastewaters may be desirable prior to discharge to the receiving water, in order to control the growth of algae.

Ammonia (NH_3), nitrite (NO^-_2) and nitrate (NO^-_3) are various forms of nitrogen. The major forms of nitrogen in municipal wastewater are ammonia and organic nitrogen. Bacteria can oxidize the ammonia nitrogen to nitrites and nitrates in an aerobic environment. The predominance of nitrate nitrogen in wastewater indicates that the waste has been stabilized with respect to oxygen demand.

Ammonia was determined, raising the pH by adding sodium hydroxide (NaOH), followed by distillation and condensation of the ammonia and then colorimetrically measuring it using sulphuric acid (H_2SO_4).

Total Kjeldahl Nitrogen (TKN) analysis is employed to determine the combined concentration of ammonia and organic nitrogen in wastewater, and the result is expressed as mg/l of nitrogen. It is determined in the same manner as organic nitrogen, except that the ammonia is not driven off before the digestion step, using a digestion acid and boiling.

Orthophosphates ($PO^-_4{}^3$) is one of the phosphorus forms in municipal wastewaters and is available for biological metabolism without further breakdown. Bacteria assimilate orthophosphate during their growth process in biological wastewater treatment. This can be determined colorimetrically by directly adding combined reagent of chemical substances, sulphuric acid, potassium antimonyl tartrate solution, ammonium molybdate solution and ascorbic acid solution, which will form a coloured complex with the phosphate.

4.4.4.4 Solids Content

The total solids content is one of the most important physical characteristics of wastewater. Total solids include both suspended and dissolved solids were determined by separating the solids and liquid by means of evaporation. The suspended solids were determined by filtering a certain volume of the wastewater using Whatman GF/C glass fibre filters with an average pore size of 0.45 μm.

Volatile solids were measured by ignition at 600°C, at which temperature the organic fraction will oxidize and be driven off as gas. Volatile suspended solids (VSS) represent biomass concentration. Solids content analysis was used for assessment of this treatment process in terms of sludge production and disposal requirements.

4.4.4.5 pH and Alkalinity

Hydrogen-ion concentration is an important quality parameter of both natural waters and wastewaters. pH is a common means of expressing the hydrogen-ion concentration. For most bacteria, and thus for most wastewater treatment processes, the extremes of the pH range for growth falls somewhere between 4 and 9, with an optimum value for growth generally lying between 6.5 and 7.5. A pH meter (Kent Model 7020) was used for determination of the pH values of samples collected electrometrically.

Alkalinity in wastewater results from the presence of hydroxides,, carbonates, and bicarbonates of elements such as calcium, magnesium, sodium, potassium or ammonia, of these, calcium and magnesium bicarbonates are most common. The alkalinity of samples was determined by titrating measured volume against sulphuric acid and the results expressed in terms of calcium carbonate, $CaCO_3$.

4.4.4.6 Dissolved Oxygen and Temperature

Dissolved oxygen and temperature were measured using a dissolved oxygen meter (Clanden Model 54 ARC/230) in conjunction with an oxygen-sensitive membrane probe (YSI Model 5720A) and thermometer and recorded.

4.4.4.7 Chlorophyll "a" Concentration

The concentration of chlorophyll "a" was used to represent the concentration of algae in the test samples. The methanol extract technique was used (Appendix B). This technique involves boiling off the retained solids on a filter paper of a measured volume of the sample in 10 ml of 90% (V/V) methanol, followed by centrifuging and absorbency measuring using a spectrophotometer.

4.4.4.8 Microbiological Examination

Escherichia coli (E. coli) and *faecal streptococci* were the two microbiological examinations utilized in this research. These were used to check the effectiveness of this treatment method in the removal of such microorganisms. The samples were collected in sterile bottles and, if necessary, preserved in a cold room and examined the same day.

The plate count method was used for both bacterial enumeration tests. Eosin methylene blue was used as the agar media for examination of *E. coli.* 37°C was the incubation temperature used for a 24 to 48 hours incubation period, whereas the Slanetz and Burtley technique (APHA *et al.*, 1989) was used for *faecal streptococci* enumeration using an incubation temperature of 44°C and an incubation period of 48 hours.

Mixing and Aeration Studies

In facultative aerated lagoons the power level input will have an important effect on the mixing intensity in the reactor and hence on the treatment process. Due to many factors, such as short circuiting and eddies, wastewater treatment unit operation represents neither a plug flow nor a completely mixed system but an intermediate flow condition. The deviation from ideal flow extremes and the characteristics of mixing characteristics can normally be determined by tracer tests.

Tracer tests involve the addition of an inert tracer to a process unit and measurement of tracer concentration in the process effluent. Subsequent analysis of the tracer data allows identification of system flow-through hydraulics. Chapter Two reviewed literature concerned with mixing hydraulics in treatment reactors.

5.1 Reactor Hydraulic Characterisation

Actual hydraulic retention time, dead zones volume, active volume, dispersion number (d) and identification of other non-ideal flow behaviour, such as short-circuiting, can be determined from tracer test results. Treatment system performance is affected by hydraulic loading and this is represented by the value of the actual (mean) residence or detention time. Short-circuiting in tanks and exit and entrance hydraulic mixing characteristics can be represented by a value of "d" (Thirumurthi, 1969). The value of "d" is zero and infinity, respectively, for plug flow and completely mixed systems. Chapter Four presented equations used to determine the hydraulic characteristics of a system using tracer test data. Following tracer tests, an analysis of the data obtained and the results of these tests are presented in this chapter.

5.2 Simulation Model

5.2.1 Introduction

Single value descriptions such as dispersion number (d), of the Exit Age Distribution (EAD) curves have been used in this research. Many conventional methods of analyzing tracer test data utilize these single value methods for the hydraulic characterisation of a system. Although these methods are not complicated, their shortcomings (Chapter 2) make them unsuitable to describe actual reactors.

For more description of the EAD curves, a mathematical simulation model should be developed from which an EAD curve could be used to simulate the different shapes of these actual EAD curves. A hydraulic description of the actual system is described by the parameters of the mathematical model used for the simulation process.

5.2.2 Simulation Model Development

The model selected and developed in this research was a modification of a mathematical model for wastewater treatment in aerated stabilization basins, as described by the National Council for Air and Stream Improvement (NCASI, 1982b). The developed simulation model considers only streams entering and exiting the reactor and the reactor itself. The reactor is mathematically segmented into one to four compartments in series, each segment being isolated from the upstream segment by imaginary mathematical boundaries. Figure 5.1 illustrates a basin segmentation into four compartments. The segments are operated in series and need not be of equal volume

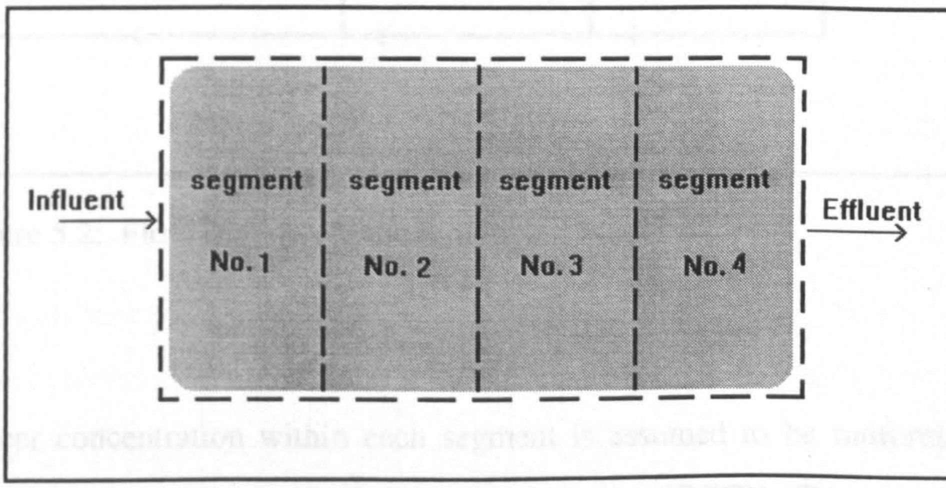


Figure 5.1: Reactor Imaginary Segments used in the Simulation Model.

Between each two adjacent segments a backmixing flow is incorporated into the model to provide two-dimensional flow across the artificial segment boundaries. Backmixing flows allow for transport across segment boundaries in both upstream and downstream directions. A recycle flow was also incorporated in the model. Recycle flow is defined as a flow diverted from a model segment and returned to the raw influent in the basin. Recycle flow is usually considered to alleviate plug flow conditions. Figure 5.2 contains a diagram illustrating backmixing and recycle flow throughout the system. Both recycle flow and backmixing flow are expressed as a percentage of the influent flow.

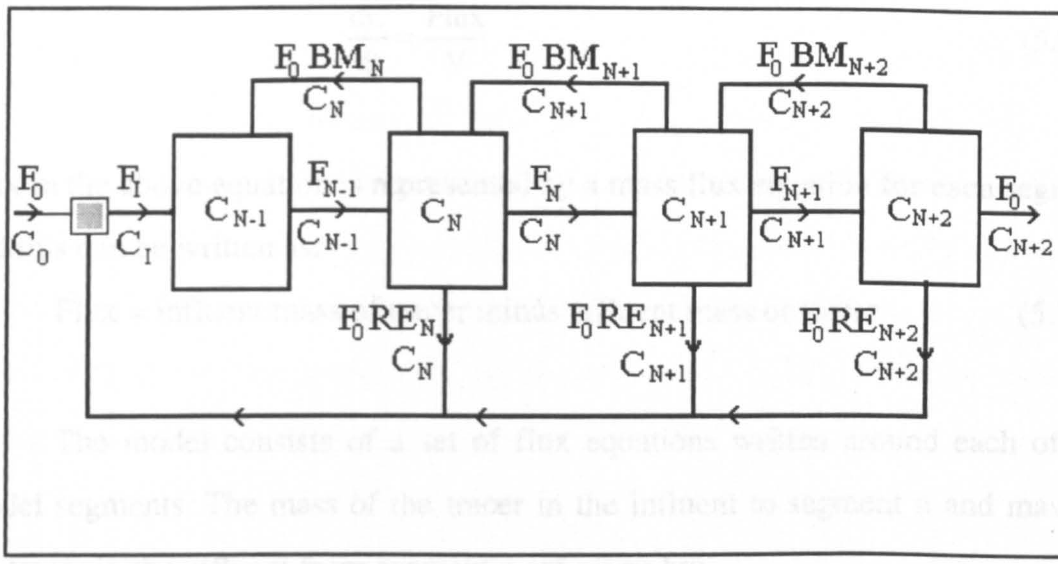


Figure 5.2: Flow Diagram in the Simulation Model

Tracer concentration within each segment is assumed to be uniform, i.e. each segment represents a completely stirred tank reactor (CSTR). This means that the effluent tracer concentration from each segment, when a pulse input of tracer is injected into that segment, will decrease with time according to the following differential equation:

$$\frac{dC}{dt} = \frac{Flux}{V} + r_C \qquad (5.1)$$

where, C = effluent tracer concentration,

t = time,

Flux = mass flux across segment (change of tracer mass between influent and effluent),

V = segment volume, and

r_C = reaction rate of tracer.

Since the tracer is an inert (conservative) material, r_C will be zero and the above equation reduces to:

$$\frac{dC}{dt} = \frac{Flux}{V} \tag{5.2}$$

Flux in the above equation is represented by a mass flux equation for each segment and this can be written as:

Flux = influent mass of tracer minus effluent mass of tracer (5.3)

The model consists of a set of flux equations written around each of the model segments. The mass of the tracer in the influent to segment n and mass of the tracer in the effluent from segment n are given by:

Influent mass of tracer = $(F_{N-1} \times C_{N-1}) + (F_0BM_{N+1} \times C_{N+1})$ (5.4)

Effluent mass of tracer = $(F_N \times C_N) + (F_0BM_N \times C_N) + (\ (F_0RE_N \times C_N)$ (5.5)

where, $F_{N-1} = F_I + F_0BM_N$

$F_N = F_{N-1} + F_0\ (BM_{N+1} - BM_N - RE_N)$

$F_{N+1} = F_N + F_0(BM_{N+2} - BM_{N+1} - RE_{N+1})$

$F_I = F_0\ (1+\Sigma RE)$

$C_I = F_0\ (C_0 + \Sigma(RE\times C))$

C_0 = mass of tracer injected divided by the total volume of segments.

Substituting the above two Equations 5.4 and 5.5 into Equation 5.2 and solving for segment n, Equation 5.2 would be written in the following form:

$$\frac{dC_N}{dt} = \frac{Flux_N}{V_N} \tag{5.6}$$

For each segment, Equation 5.6 should be solved separately; this means that a similar number of partial differential equations to the number of segments should be solved.

Two methods are used to solve any partial differential equation; these are namely analytical and numerical solutions. The analytical solution of these differential equations is an accurate solution. Many partial differential equations cannot be solved analytically because of variable coefficients, boundary conditions or complexity. For the above differential equation the complexity of using this method led to the conclusion that the analytical solution could not be used. A numerical solution would be easier but less accurate.

To improve the accuracy of the results, the fourth-order Rung-Kutta method described elsewhere (Elliott *et al.*, 1984), which is classified as a single-step method, has been adopted in research for solving this type of differential equation. An iterative procedure was adopted, using a time step, dt, as the increment in the time parameter. Depending on the value of the time step, the accuracy of the solution will be affected; the accuracy of the solution will increase with decrease in value of this time step. For each iteration, all the differential equations should be solved in the order of first segment to last segment. The effluent tracer was the influent to the succeeding segment and the concentration of the tracer in the effluent from the last segment will be the effluent concentration of the simulation model which will be used for drawing the simulation C-curve. Such a model can be used for any type of input signals used as methods of tracer injection in the influent stream, such as a pulse or step function. This can reduce any inaccuracy due to the effect of non-ideal pulse signals used in some models. The above model was written on computer as a FORTRAN program since a large number of iterations are required to obtain a simulation curve. The program was named "TRISM" and is set out in Appendix C.

5.2.3 Simulation Model Testing

For the purpose of validation the model developed in this research should be tested for the simulation of output tracer concentration, C-curves, of some relatively simple systems from which the change in tracer concentration with time in the effluent can be determined by using one-shot exact equations.

The simple system used to test the model comprised a series of continuous-flow stirred-tank reactors. The EAD curve of this system was due to a slug, pulse, of a tracer injected into the first reactor of a series of equally sized reactors. Metcalf and Eddy (1979) and Levenspiel (1972) describe the C-curve of this system. The equation can be written in the following form:

$$E_\theta = \frac{N(N\theta)^{N-1}}{(N-1)!} e^{-N\theta} \qquad (5.7)$$

or

$$E_\theta = \frac{\theta^{N-1}}{(N-1)!} e^{-\theta} \qquad (5.8)$$

$$E_\theta = \frac{C_t}{C_0} \qquad (5.9)$$

$$\theta = \frac{t \times Q}{V} \qquad (5.10)$$

where, C_t = Effluent tracer concentration; mg/l,

C_0 = Mass of tracer injected per the total volume of all the reactors which is the initial tracer concentration in the first tank; mg/l),

N = Number of tanks,

t = Time; h,

Q = Flow rate; l/h, and

V = Volume of tank (l).

Three systems of tanks-in-series were used in testing the simulation model developed, to form three tests and these were:

1 - Four tanks in series,

2 - Three tanks in series, and

3 - Two tanks in series.

For simulation model testing, Equation 5.7 was used for a system of 81 litres size, i.e., total volume of all the reactors, a flow rate of 0.3375 l/h, a total retention time of 10 days, and 1.0 mg/l as the value of C_0. In the developed model the values of backmixing (BM) and recycle (RE) flow are set to zero to represent actual partitions between the tanks in the series. Values of other parameters were:

$C_0 = 1$ at the first time step, dt.

$F_0 = 0.3375$ l/h

dt = 0.01 h

N = 50000

V1,V2, V3 and V4 were of equal values and were calculated by dividing the total system volume, 81 litres, by the number of tanks. V3 and V4 were set to zero when the number of tanks in series was only two, whereas only V4 was set to zero when three tanks in series were used.

The calculations were carried out for more than two retention times, which is twice the value of θ. The results of both the simulation model and Equation 5.7 were drawn for the three systems and the graphs are shown in Figures 5.3 - 5.5, respectively.

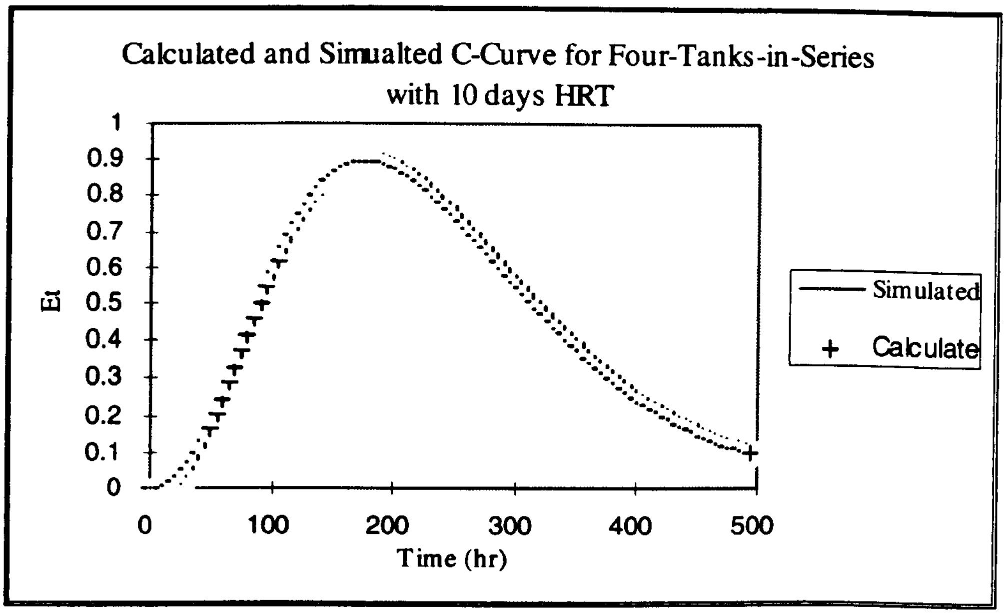


Figure 5.3: Calculated and Simulation C-curves for Four-Tanks-in-Series System.with 10 Days Hydraulic Retention Time

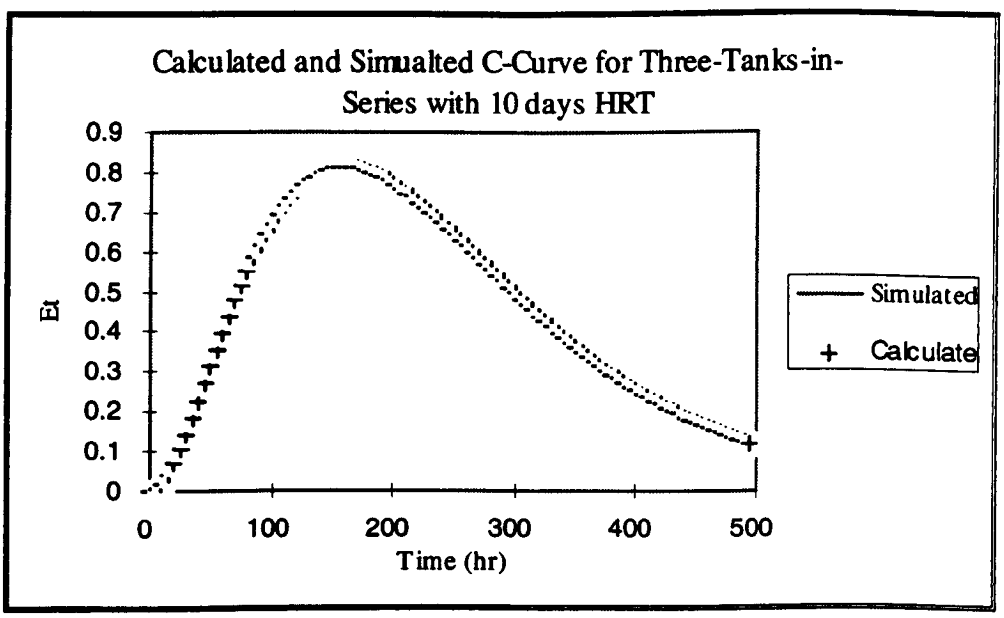


Figure 5.4: Calculated and Simulation C-curves for Three-Tanks-in-Series System.with 10 Days Hydraulic Retention Time

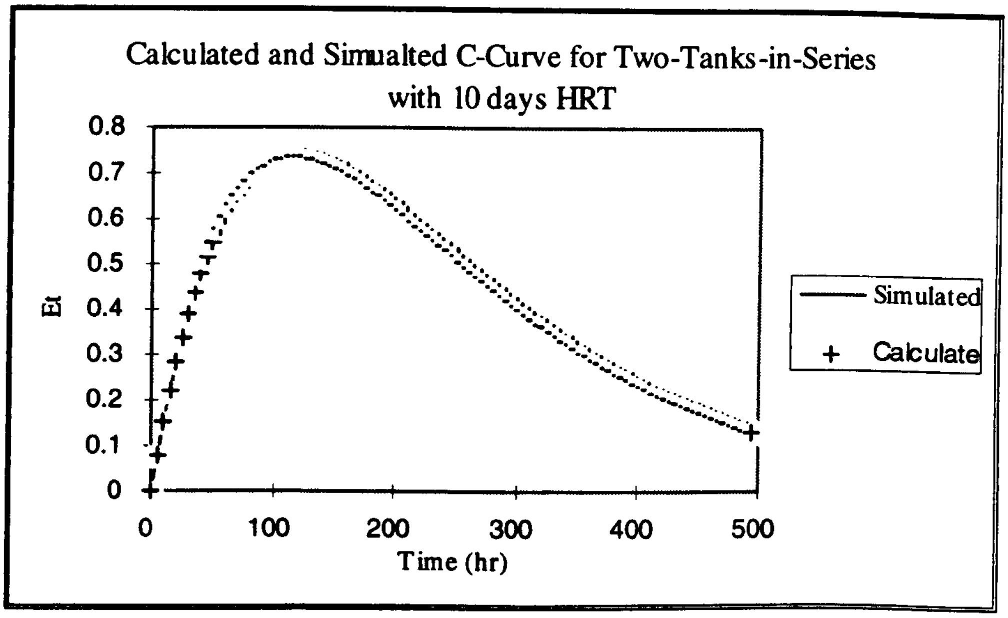


Figure 5.5: Calculated and Simulation C-curves for Two-Tanks-in-Series System.with 10 Days Hydraulic Retention Time

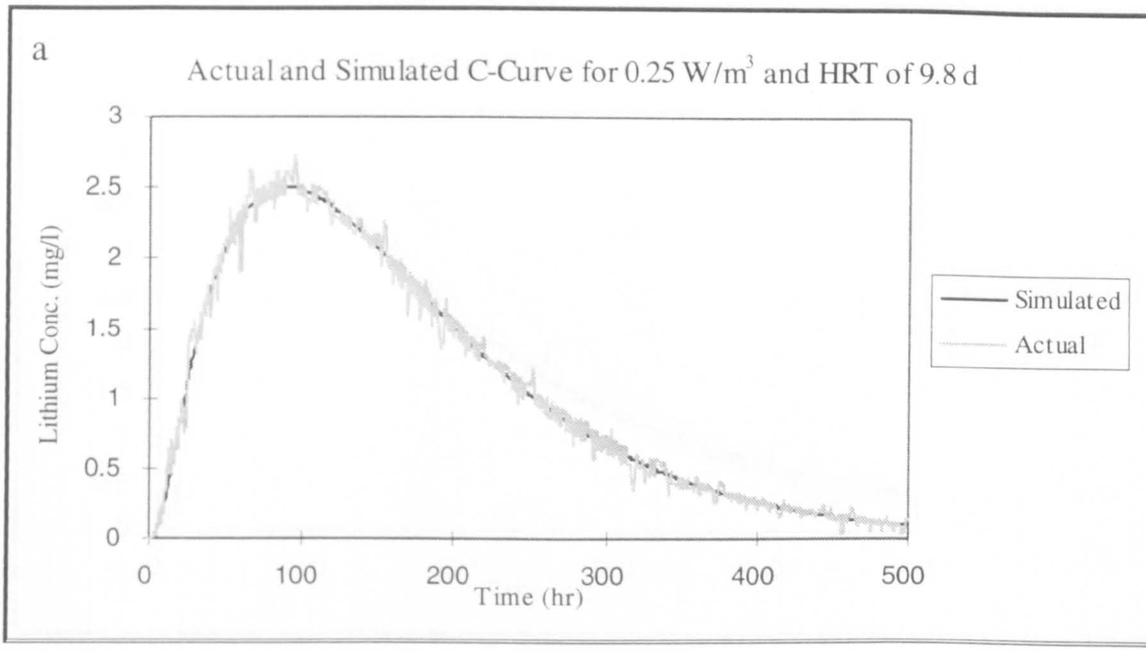
It is clear from these figures that the calculated and simulated C-curves are almost identical for each of the three systems, proving the capability and potential of the model.

5.3 Mixing Studies Results

Mixing studies were carried out at different power levels and flow rates; the latter was changed according to the organic loading required. For the 20°C temperature experimental runs, a total of twelve tracer tests were carried out at the end of the operational test run. Ratios of actual to designed hydraulic retention time were used to determine the actual hydraulic retention times for the corresponding runs operated at 30°C temperature.

The mixing characteristics were determined utilizing the age distribution of the exit stream. Lithium chloride (Li Cl) was injected as a pulse and used as a tracer in all mixing studies. 2.4736 gm Li Cl dissolved in warm water was injected with the influent into the reactor which gave a uniform initial concentration of 5 mg/l Li^+ in the reactor if initial complete mixing took place in the reactor. The sampling process was performed for more than two theoretical hydraulic retention times.

Tracer concentration in the effluents during the different runs was measured with time during the mixing test period and a graph of effluent tracer concentration versus time, C-diagram or C-curve, was drawn. The C-curve, which is known as the exit age diagram, for each of the twelve tests is presented in Figures 5.6 to 5.8. The data of the tracer were analysed for determination of the hydraulic characteristics of the flow regime taking place in the reactor. A FORTRAN program "HYDCH" developed in this research (Appendix C) was then used to



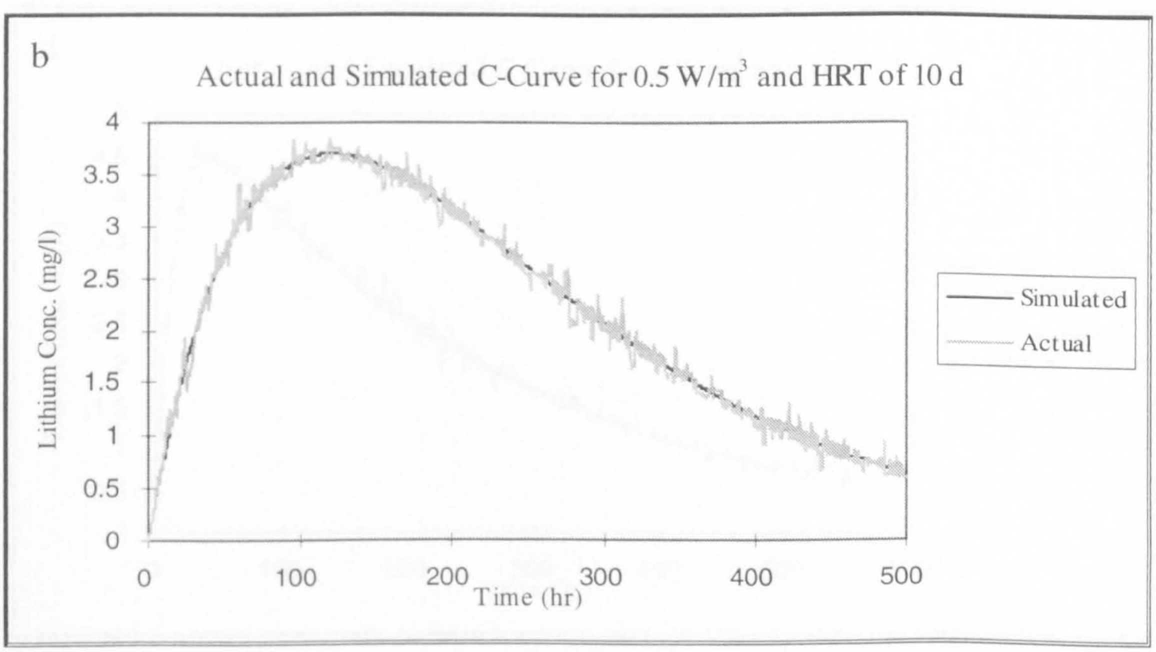
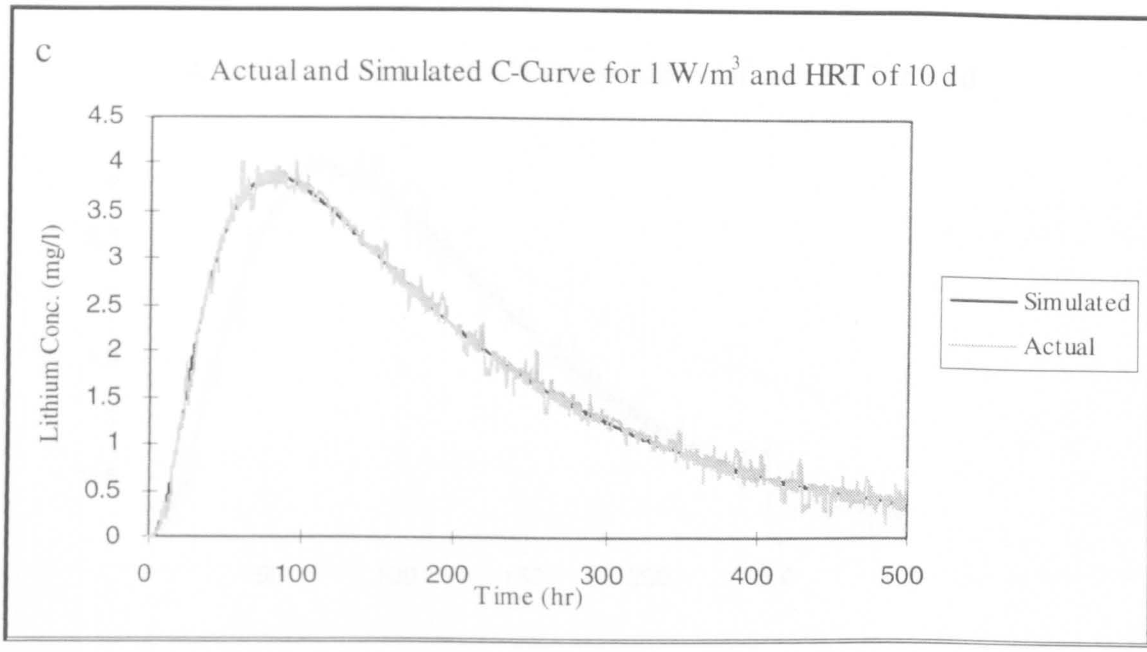


Figure 5.6 Actual and Simulated C-curves for Runs with 20 g BOD_5/m^3.d Organic Loading



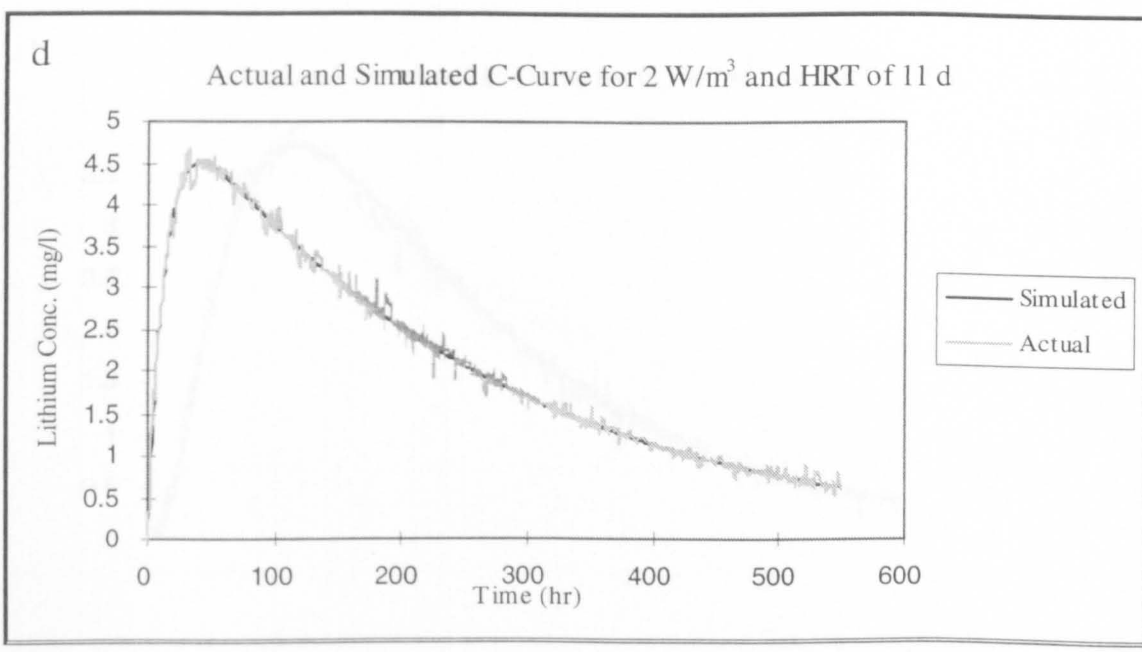
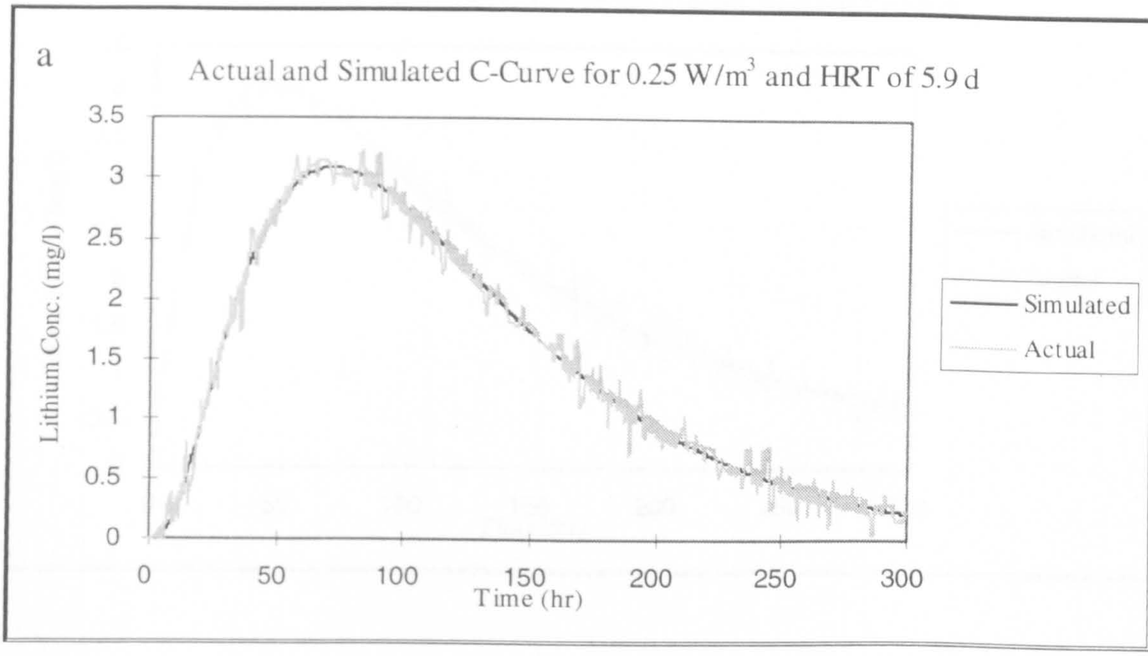


Figure 5.6 (contd) Actual and Simulated C-curves for Runs with 20 g $BOD_5/m^3.d$ Organic Loading



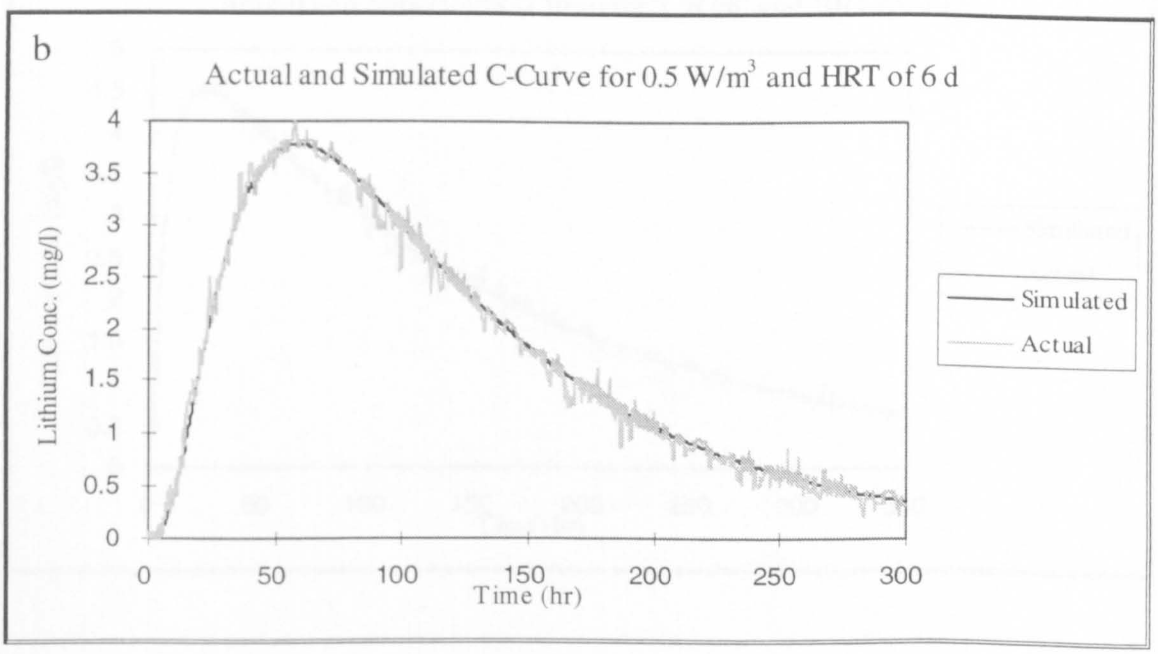
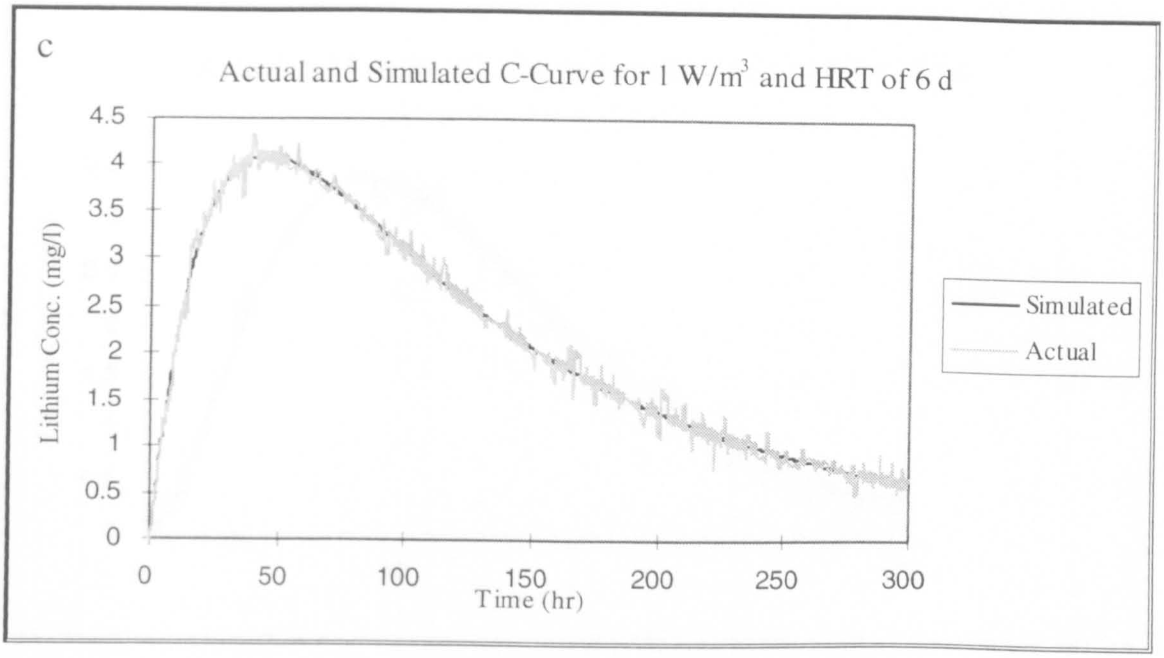


Figure 5.7 Actual and Simulated C-curves for Runs with 33 g BOD_5/m^3.d Organic Loading



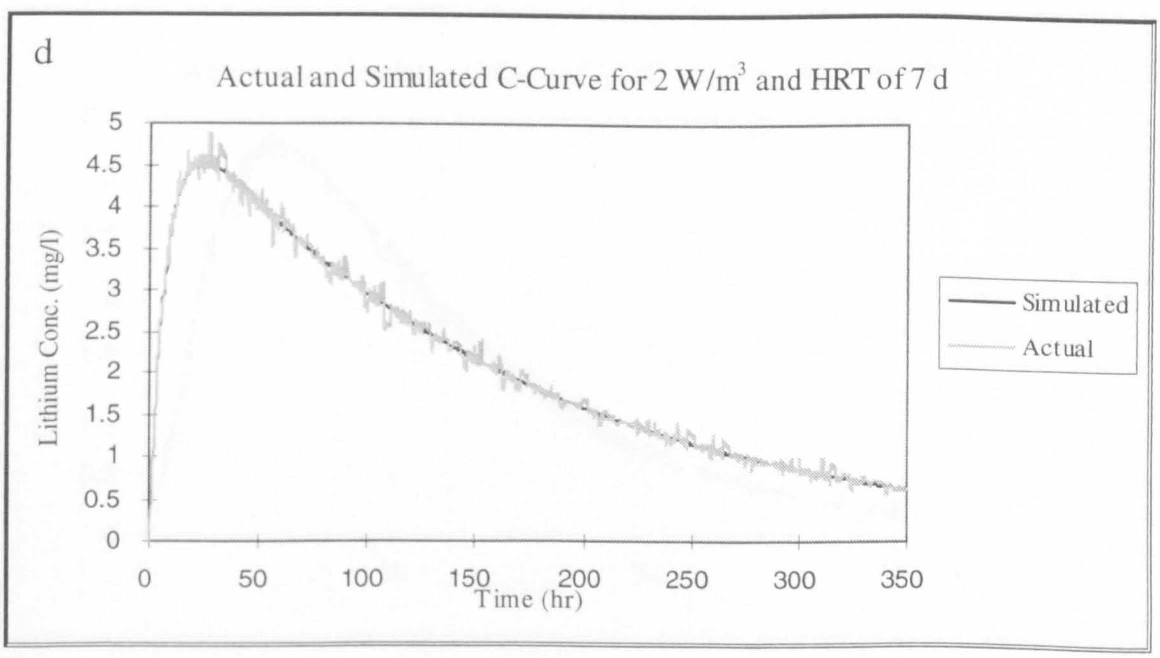
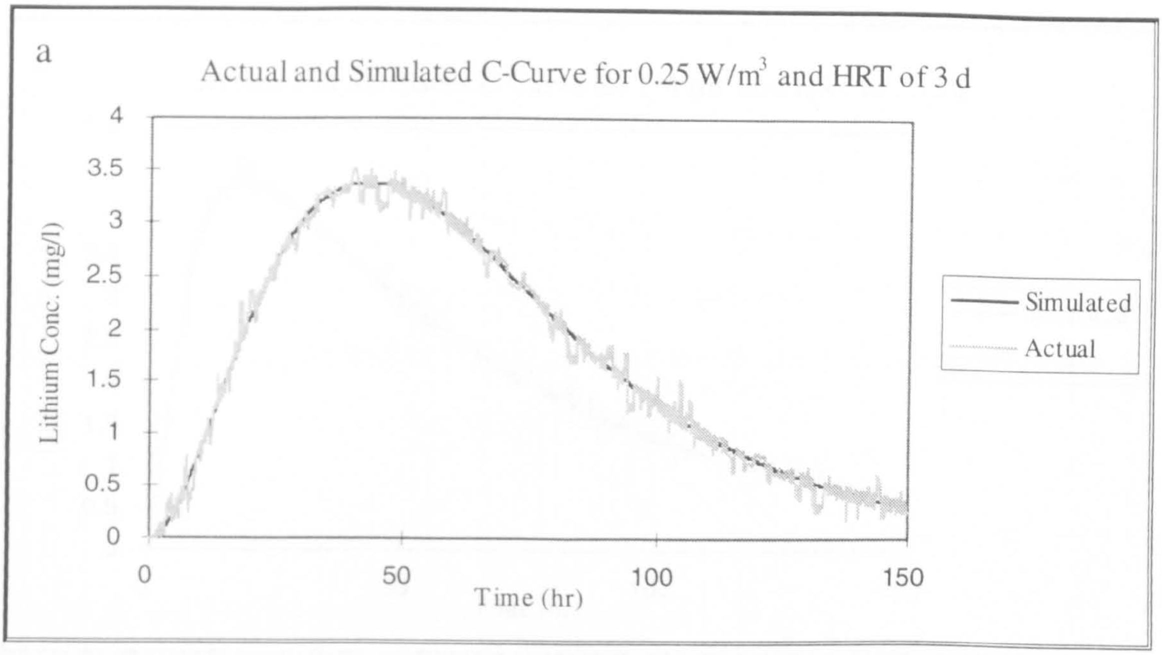


Figure 5.7 (contd) Actual and Simulated C-curves for Runs with 33 g $BOD_5/m^3.d$ Organic Loading



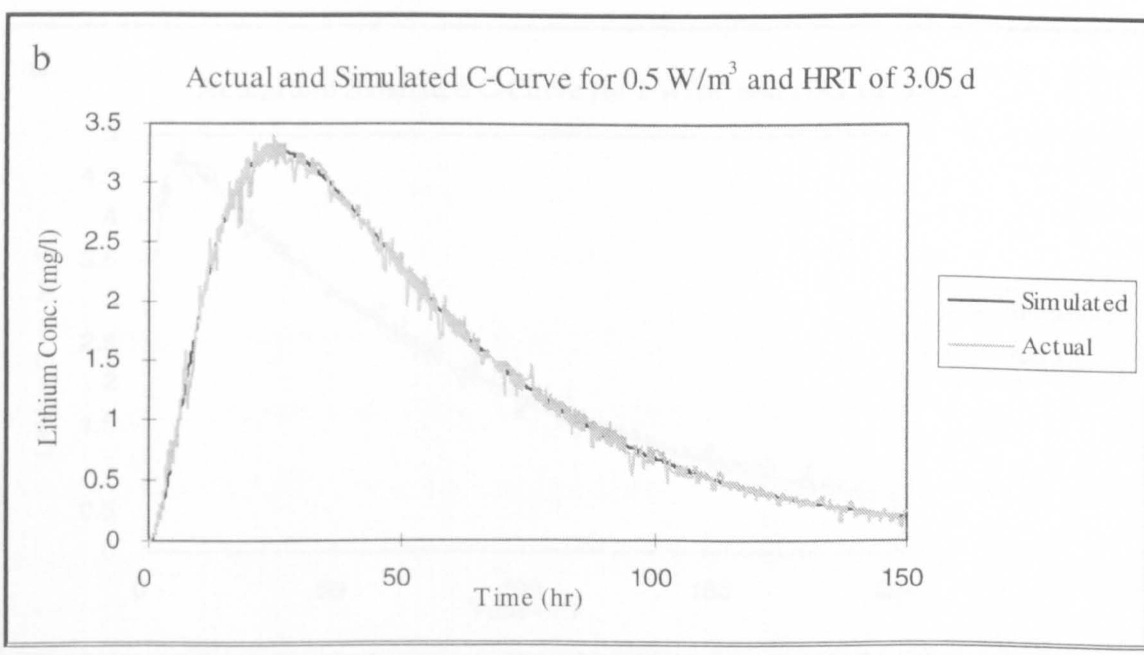
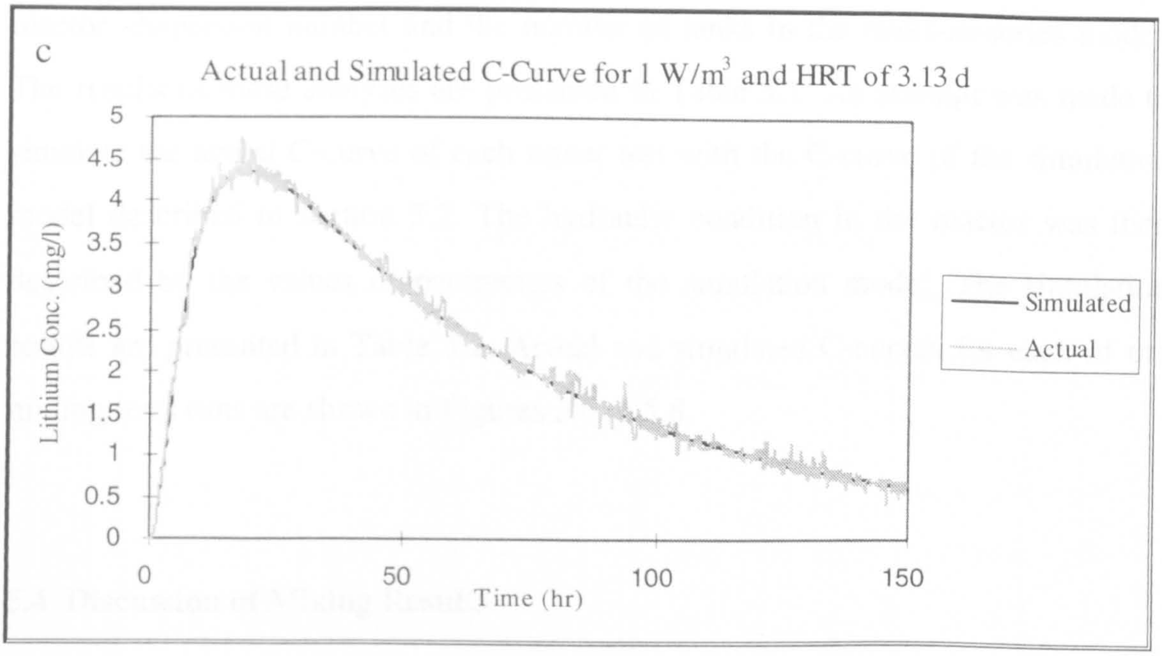


Figure 5.8 Actual and Simulated C-curves for Runs with 62 g BOD_5/m^3.d Organic Loading



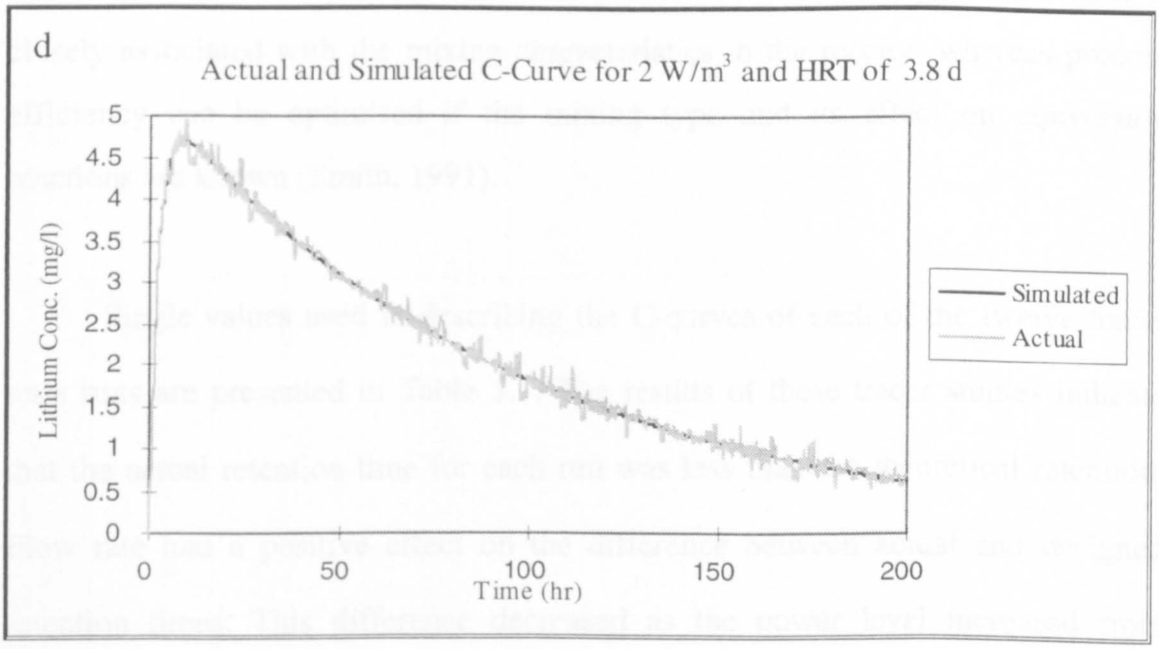


Figure 5.8 (contd) Actual and Simulated C-curves for Runs with 62 g $BOD_5/m^3.d$ Organic Loading

estimate the actual hydraulic retention time, active volume and dead volume of the reactor, dispersion number and the number of tanks in the tanks-in-series model. The results of these analyses are presented in Table 5.1. An attempt was made to simulate the actual C-curve of each tracer test with the C-curve of the simulation model described in section 5.2. The hydraulic condition in the reactor was then described by the values of parameters of the simulation model. The simulation results are presented in Table 5.2. Actual and simulated C-curves for each of the mixing tests runs are shown in Figures 5.6 to 5.8.

5.4 Discussion of Mixing Results

Mixing study results are useful in determining the extent of mixing in reaction tanks and hence the degree of the contact between food and microorganisms. The performance of a process utilizing biological treatment is closely associated with the mixing characteristics in the reactor, whereas process efficiency can be optimised if the mixing type and its effect on conversion reactions are known (Smith, 1991).

Single values used in describing the C-curves of each of the twelve tracer tests runs are presented in Table 5.1. The results of these tracer studies indicate that the actual retention time for each run was less than the theoretical retention. Flow rate had a positive effect on the difference between actual and designed retention times. This difference decreased as the power level increased from 0.25W/m^3 to 2W/m^3. The actual and theoretical retention times were 3.04 d and 3.2 d at 0.25W/m^3 power level, whereas the corresponding values at 2W/m^3 were 9.83 d and 10.31 d, respectively. Dead zones volume in the reactor was seen to decrease with increase in power level. Of the 81 litres volume of the model aerated

Table 5.1: Hydraulic Characteristics of the Experimental Runs at 20°C

Run No.	P/V (W/m^3)	Retention Time d)		Volume (l)			Disper. No.	NTA
		Design	Actual	Design	Active	Dead		
1	0.25	3.2	3.04	81	77	4	0.1713	3.52
2	0.25	6.16	5.92	81	77.8	3.2	0.1872	3.28
3	0.25	10.31	9.83	81	77.2	3.8	0.2743	2.49
4	0.5	3.16	3.05	81	78.1	2.9	0.2880	2.41
5	0.5	6.20	6.02	81	78.6	2.4	0.2438	2.70
6	0.5	10.29	10.01	81	78.8	2.2	0.2232	2.88
7	1.0	3.21	3.13	81	79.1	1.9	0.3243	2.23
8	1.0	6.17	6.03	81	79.2	1.8	0.3291	2.21
9	1.0	10.23	10.04	81	79.5	1.5	0.2834	2.43
10	2.0	3.87	3.83	81	80.1	0.9	0.4909	1.78
11	2.0	7.12	7.04	81	80.1	0.9	0.4470	1.86
12	2.0	11.1	11.01	81	80.34	0.66	0.4221	1.92

P/V: is Power level
NTA: is Number of tanks in tanks-in-series model using Equation 4.6.

Table 5.2: Simulation Model Results

Design Parameters				Simulation Parameters										
Run No.	P/V (W/m^3)	THRT (d)	Temp. (°C)	No. of segmts	Volume of segments (l)				Backmixing parameters			Recycle parameters		
					V1	V2	V3	V4	BM12	BM23	BM34	RE2	RE3	RE4
1	0.25	3	20	3	30	30	21	0	1	1	1	0	0	1
2	0.25	6	20	4	30	15	6	30	0	0	0	1	0	0
3	0.25	10	20	4	30	15	6	30	0	0	0	1	2	0
4	0.5	3	20	3	27	27	27	0	1	1	1	0	0	1
5	0.5	6	20	4	30	15	6	30	0	0	0	1	2	2
6	0.5	10	20	2	40	41	0	0	0	1	1	0	1	1
7	1.0	3	20	4	30	15	6	30	2	2	2	0	0	0
8	1.0	6	20	2	40	41	0	0	1	2	1	0	2	1
9	1.0	10	20	4	30	15	6	30	1	1	1	1	1	0.5
10	2.0	4	20	2	77	4	0	0	0	2	2	1	2	1
11	2.0	7	20	2	41	40	0	0	2	1	2	0	2	1
12	2.0	11	20	2	77	4	0	0	1	2	2	0	1	1

lagoon, the dead zones volume was 4 l at 0.25 W/m^3 power level and 3 days retention time, but it was 0.9 l was at 2 W/m^3 and 4 days retention time. Slade (1991), in a tracer study on a full scale aerated lagoon treatment system consisting of four ponds in series, found that both total dead zones volume and actual residence time were 85 percent of those designed.

The dispersion number, derived from an equation which uses the variance of the C-curve, was affected positively by the power level in the reactor. The dispersion number was observed to vary from 0.1713 to 0.4909 over the range 0.25W/m^3 to 2W/m^3 of the power level applied. These dispersion numbers fall within the range of those for incompletely-mixed aerated lagoons. Thirumurthi (1980) estimated the magnitude of dispersion numbers to be from 2 to 5.6 in three laboratory aerated lagoons aerated by air diffusers at an air flow rate of 0.05 l/min to 0.094 l/min. These high values at low air flow rates can be attributed to the method of feeding the reactor, which was by batch addition rather than continuous feeding, which might have may lead to high mixing in the reactor. Metcalf and Eddy (1979) reported that dispersion numbers for conventional plug-flow activated sludge reactors are probably within the range from zero to 0.2 while for reactors with mechanical aerators designed to operate as completely mixed systems d values are probably in the range from 4.0 to ∞. Aerated lagoon systems may deviate considerably from a completely mixed system, especially at the lower mixing intensities (Bishop, 1975).

As previously mentioned, these single values do not describe the shape of the curve. Accordingly a simulation model was developed to overcome this shortcoming. As can be seen from Table 5.2 the number of segments used in the simulation model decreased with increase in power level, whereas both backmix and recycle flows increased with power level and retention time.

5.5 Aeration System Performance and Discussion

Rational design and operation of an aerated biological treatment process requires a knowledge of the oxygen-transfer capacities of the aeration system under consideration or being installed. Testing the aeration system in terms of its oxygenation rate capacity was a good indicator of the ability of the aeration system used in the facultative aerated lagoon model utilized in this research to transfer oxygen to the liquid in the reactor.

In this research, the aeration capacities of an aeration system for the various power levels, air flow rates, were tested. at two levels of temperature The non-steady state reoxygenation of tap water approach was used. This approach has been described in Chapter Four. Mcwhirter (1965) suggested that this method should be used as a standard method due to its reproducibility and precision. Results in terms of the rate of oxygen supplied, rate of oxygen transferred, overall oxygen transfer coefficient (K_La), and oxygen transfer efficiency of the aeration process at different operational parameters of air flow rates, and hence power levels, and two levels of temperatures (20°C and 30°C) are shown in Table 5.3. The calculations of rates of oxygen transfer in this Table (Table 5.3) are based on the air specific weight of 1.226 kg/m^3 (0.0766 lb/ft^3).

It was noted that oxygen transfer rates decreased with increase in liquid temperature from 20°C to 30°C. This was supported in the literature (Conway *et al.*, 1966; Bennett and Kempe, 1965 and Metcalf and Eddy, 1972, 1979 and 1991) concerned with testing aeration systems in wastewater treatment reactors which stated that temperature has a considerable effect on the oxygen transfer capacity of aeration systems. Figures 5.9 to 5.11 show how the increase in temperature

Table 5.3 Oxygen Supply and Transfer Capacities of the Aeration System Used.

Power Level (W/m^3)	Temp. (°C)	O_2 Supply Rate (g/hr)	O_2 Transferred		Efficiency (%)
			K_La (hr^{-1})	Rate (g/hr)	
0.25	20	4.27	0.46886	0.322	7.54
	30	4.27	0.34227	0.195	4.57
0.5	20	8.54	0.640757	0.440	5.15
	30	8.54	0.50639	0.287	3.37
1.0	20	17.1	1.1560	0.794	4.65
	30	17.1	0.9691	0.550	3.22
2.0	20	34.16	2.0182	1.385	4.1
	30	34.16	1.9157	1.0875	3.18

K_La = Over-all transfer coefficient

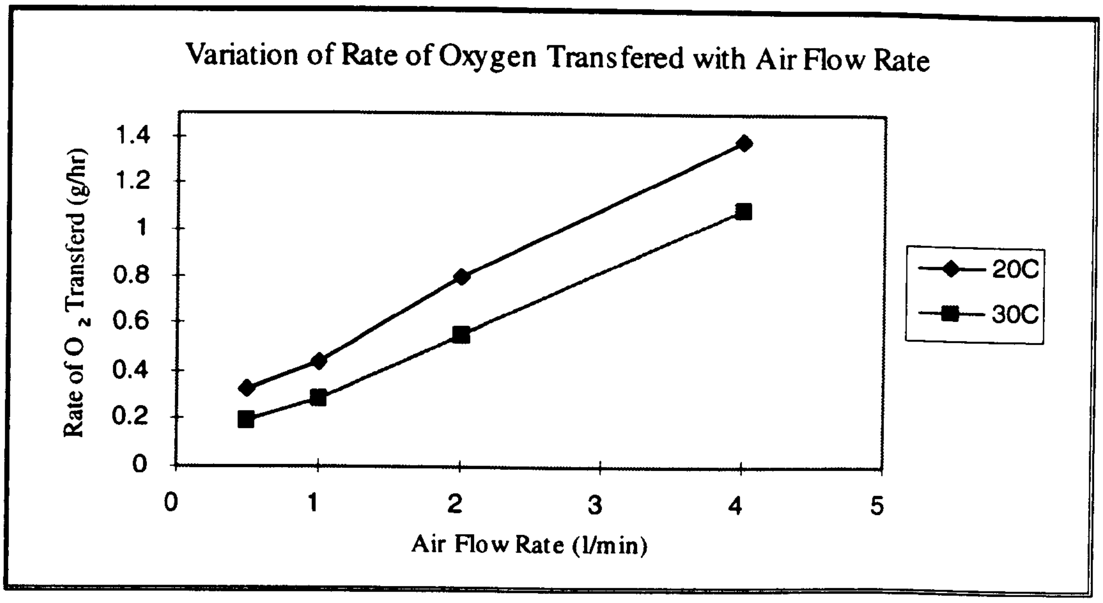


Figure 5.9 Variation of Oxygen Transfer Rate with Air Flow Rate

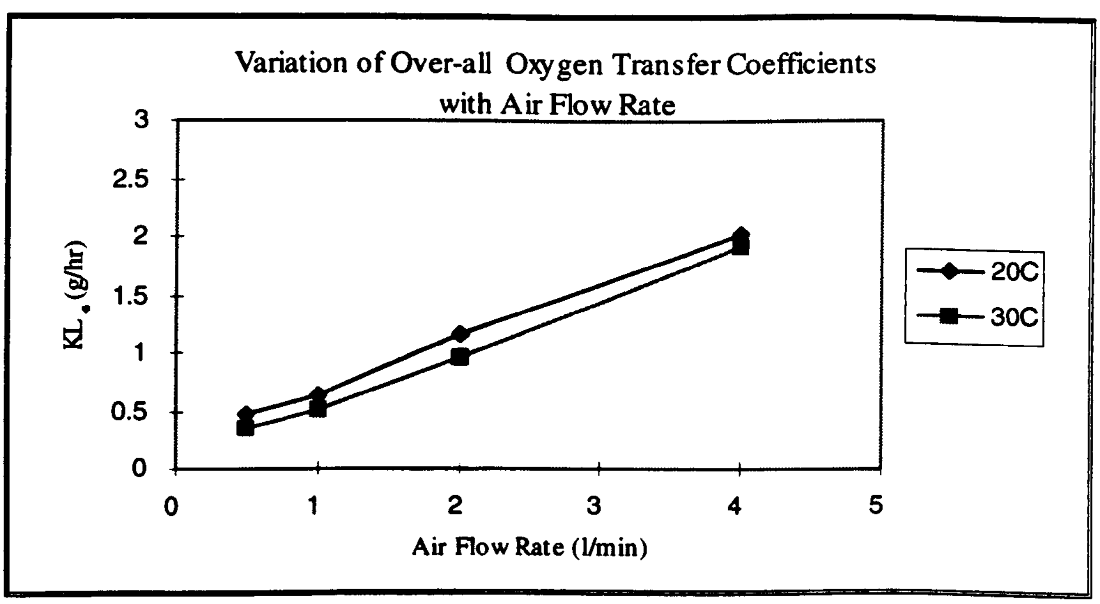


Figure 5.10 Variation of Over-all Oxygen Transfer Rate Coefficienct with Air Flow Rate

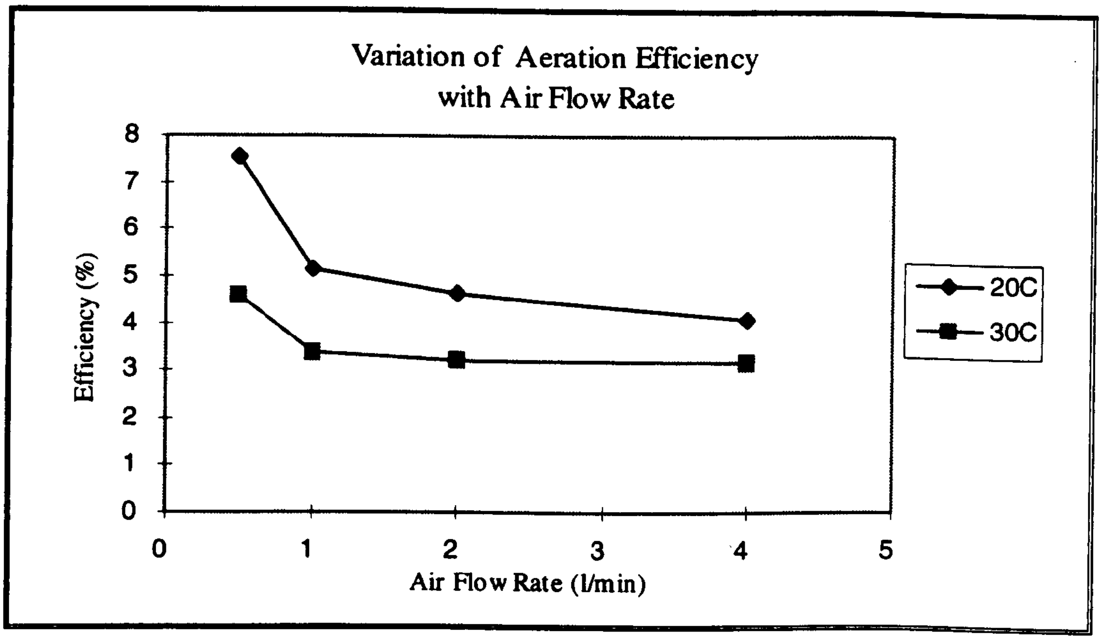


Figure 5.11 Variation of Aeration Efficiency with Air Flow Rate.

from 20°C to 30°C had an inverse effect on all three parameters mentioned above (rate of oxygen transferred, overall oxygen transfer coefficient and efficiency). Figure 5.9 indicates that the effect of temperature on the rate of oxygen transferred was more at higher air flow rates than at lower air flow rates.

It was believed that the rate of oxygen transfer of aeration systems is dependent on the power level in terms of air flow rate supplied. From the results presented in the above table (Table 5.3), as was expected, the rate of oxygen transfer was found to be affected positively by the flow rate of the air introduced into the reactor. Figure 5.9 shows the variations in the oxygen transfer rate over the range of the supplied air flow rate (0.5, 1, 2 and 4 l/min). This Figure reveals that oxygen transfer rate increased with increase in air flow rate and decrease with increase in temperature. This rate of increase was linear over that range of the applied air flow rate. It can be seen from Figures 5.10 and 5.11, that the over-all transfer coefficient (K_La) was affected positively by air flow rate, whereas the oxygen transfer efficiency was inversely affected by the flow rate of the air introduced into the reactor.

Aeration system transfer efficiency in transferring oxygen to the liquid dropped sharply from 7.54 percent and 4.57 percent to 5.15 percent and 3.37 percent when the air flow rate increased from 0.5 l/min to 1 l/min for the two operating temperatures of 20°C and 30°C, respectively. This decrease was less when air flow rate increased from 1 l/min to 2 l/min and from 2 l/min to 4 l/min. Conway *et al.* (1966) had conducted oxygen-transfer studies in full-scale aeration basins utilizing sparged-turbine aerators and found that both K_La and transfer efficiency were 12.7 h^{-1} and 21.4 percent, respectively, at 20°C. These high values compared with the results found in this research may be due to the differences in the operating conditions, such as depth of aerator and the size of basin as well as type of the aerators used. However, lower transfer efficiency of 10 percent will satisfy conditions throughout the year and supply an adequate safety factor (Dawson and Grainge, 1969).

It can be said that the efficiency of the aeration system was maximum at 0.5 l/min air flow rate but at this air flow rate the rate of oxygen transferred was low (0.322 g/hr and 0.195 g/hr for both temperatures 20°C and 30°C, respectively). Thirumurthi (1980) assumed 10 percent efficiency of the aeration system (diffused aeration) used in a laboratory-model aerated lagoon. Oxygen transferred must be adequate to satisfy that required by the biomass during aerobic degradation of the organic matter.

Oxygen-transfer efficiency (OTE) of porous diffusers may decrease with use due to internal clogging or exterior fouling. Internal clogging may be due to impurities in the compressed air that have not been removed by the air filters (Metcalf and Eddy, 1991). External fouling may be due to the formation of biological slimes or inorganic precipitants. However, in this research, for more accurate results, air flow rates were fixed to the required levels by introducing an air flostat which was used to guarantee that only the air flow rate required would be provided to the reactor even if fouling occurred.

TREATMENT PERFORMANCE OF FACULTATIVE AERATED LAGOONS

6.1 Introduction

The removals of biodegradable organics, suspended solids, and pathogens are the main considerations in secondary wastewater treatment (Metcalf and Eddy, 1979). Removal improvements and better removal of other pollutants are required when more stringent standards are imposed and reuse of wastewater is to be considered. Facultative aerated lagoons can be considered as a secondary treatment process so the performance in removing pollutants such as BOD and COD, which are designed to be removed in a secondary treatment process, are herein considered and examined.

In the present study, facultative aerated lagoon performance variations were expressed in terms of:

- removal rate coefficients
- removal efficiencies
- effluent qualities
- temperature coefficients.

Studying the variations of each of the above parameters as a consequence of the variation of the controllable parameters was the objective of this research. The controllable parameters applied, which have previously been mentioned, include:

- volumetric organic loading (20, 33 and 62 g $BOD_5/m^3.d$)
- power level (0.25, 0.5, 1 and 2 W/m^3), and
- temperature (20° and 30°C).

The variation in process performance parameters with the food to microorganism ratio (F/M) was also considered but, although F/M was an additional parameter, it could not be considered as a controllable parameter in facultative aerated lagoons.

Rudd (1972) stated that "BOD_5 and COD are amongst the most important criteria for defining the operational characteristics of a secondary sewage treatment facility since together they provide a seemingly accurate picture of the efficiencies of removal of biodegradable carbon". Accordingly, biochemical oxygen demand (BOD_5) and chemical oxygen demand (COD) as well as suspended solids were the main parameters used in assessing these facultative aerated lagoons. Other important parameters, such as nitrogen, phosphorus, and pathogen removals in the aerated lagoon models were assessed and are presented in Chapter 7. Their variations with operational controllable parameters are also discussed.

All these process performance variations are presented graphically and, when applicable, relationships between dependent variables and independent variables are given in the form of a mathematical model.

6.2 Biochemical Oxygen Demand (BOD) Removals

Biochemical oxygen demand (BOD_5) removals in facultative aerated lagoon models at different conditions of the operational parameters representing the different experimental runs are summarised in Table 6.1. This table shows the distinct effects on BOD_5 of the operational parameters separately and in combination.

Table 6.1: Filtered and Unfiltered BOD_5 Removal Performances

VOLR* ($gBOD_5/m^3.d$)	P/V** (W/m^3)	Temp. (°C)	Removal Rate (%)		Removal Rate Coeff (d^{-1})	
			Filtered	Unfiltered	Filtered	Unfiltered
62.67	0.25	20	22.7	58.5	0.588	0.470
31.86	0.25	20	58.0	69.1	0.680	0.374
19.18	0.25	20	73.9	76.1	0.717	0.318
63.61	0.5	20	42.4	58.8	0.801	0.475
32.33	0.5	20	68.7	71.1	0.876	0.411
19.40	0.5	20	79.8	78.4	0.870	0.362
63.90	1	20	45.3	59.5	0.949	0.490
33.33	1	20	69.5	72.5	0.983	0.439
20.00	1	20	81.1	80.0	1.011	0.400
60.53	2	20	69.7	66.5	1.000	0.497
32.86	2	20	81.6	76.1	1.031	0.455
20.91	2	20	88.8	83.0	1.139	0.445
60.71	0.25	30	75.0	80.8	1.075	0.841
32.28	0.25	30	85.6	84.3	1.261	0.672
20.73	0.25	30	90.6	87.1	1.333	0.561
60.23	0.5	30	74.7	78.1	1.232	0.714
33.13	0.5	30	84.2	83.0	1.315	0.611
21.03	0.5	30	89.7	86.8	1.389	0.548
64.55	1	30	63.7	68.1	1.398	0.711
33.70	1	30	80.5	79.3	1.447	0.640
20.96	1	30	88.5	85.4	1.538	0.587
64.79	2	30	62.5	67.1	1.405	0.681
34.82	2	30	79.5	79.5	1.420	0.644
21.08	2	30	88.4	86.3	1.585	0.630

* VOLR = Volumetric Organic Loading Rate
** P/V = Power level

6.2.1 Effects of Organic Loading Rate

Three levels of organic loading rate (20, 33 and 62 g $BOD_5/m^3.d$) were applied in this research. Sawyer (1968) pointed out that facultative aerated lagoons are applicable where BOD_5 loadings do not exceed 80 g $BOD_5/m^3.d$, because the high power level required for satisfying the oxygen demand will not permit solids sedimentation. Townshend *et al.* (1969) claimed that successful operation of facultative aerated lagoons has been demonstrated with applied organic loadings up to 32 g $BOD_5/m^3.d$. Complete mixing of the pond contents will lead to the loss of the benefits to be derived from anaerobic decomposition (Metcalf and Eddy, 1979).

It was observed that the applied organic loadings in this study significantly influenced the performance of the facultative aerated lagoon model and such effects are discussed in the following sections.

i) Removal Rate Coefficients

The performance of the facultative aerated lagoon model was determined in terms of the several parameters previously mentioned. One of these parameters is the BOD_5 removal rate coefficient. Filtered BOD_5 removal rate coefficient (K_{t_f}) and unfiltered BOD_5 removal rate coefficient (K_{t_u}) were utilized in such performance studies. Removal rate coefficients for BOD_5 were calculated using the formula:

$$S_e/S_0=1/(1+K_t t). \qquad (6.1)$$

where, S_e = effluent BOD_5 concentration, and

S_0 = influent BOD_5 concentration.

The BOD_5 of both filtered and unfiltered samples of effluent were used to calculate removal rate coefficients. Filtered removal rate coefficients were based on unfiltered influent data and filtered effluent data, whereas unfiltered removal rate coefficients were based on unfiltered influent and effluent data.

Both removal rate coefficients were determined for operating temperatures of 20°C and 30°C. At each temperature the effect of the organic loadings applied (20, 33 and 62 g $BOD_5/m^3.d$) on filtered and unfiltered removal rates was studied. Figures 6.1a and 6.1b show these relationships graphically.

Filtered and unfiltered BOD_5 removal rate coefficients are plotted against organic loading at 20°C in Figure 6.1a. This Figure generally indicates that there was a clear negative effect of organic loading on the filtered BOD_5 removal rate coefficient. The plot shows clearly that at a fixed power level, the filtered removal rate coefficient decreased with increase in volumetric organic loading but, as the power level introduced to the system increased, the effect of this organic loading decreased. Feedback with soluble BOD_5 from the deposited sludge could account for the negative effect of organic loading on the filtered BOD_5 removal rate coefficient. With increase in organic loading, solids deposition also increased, due to more solids entering the system, and hence more feedback occurred. A reverse trend to that of the filtered BOD_5 coefficient was noticed on the values of the unfiltered BOD_5 coefficient as a result in the increase in organic loading. At high power level this trend is less noticeable, which means that the effects of organic loading and power level are mutually dependent. The improvement in the values of the unfiltered BOD_5 coefficients which resulted from the increase in the organic

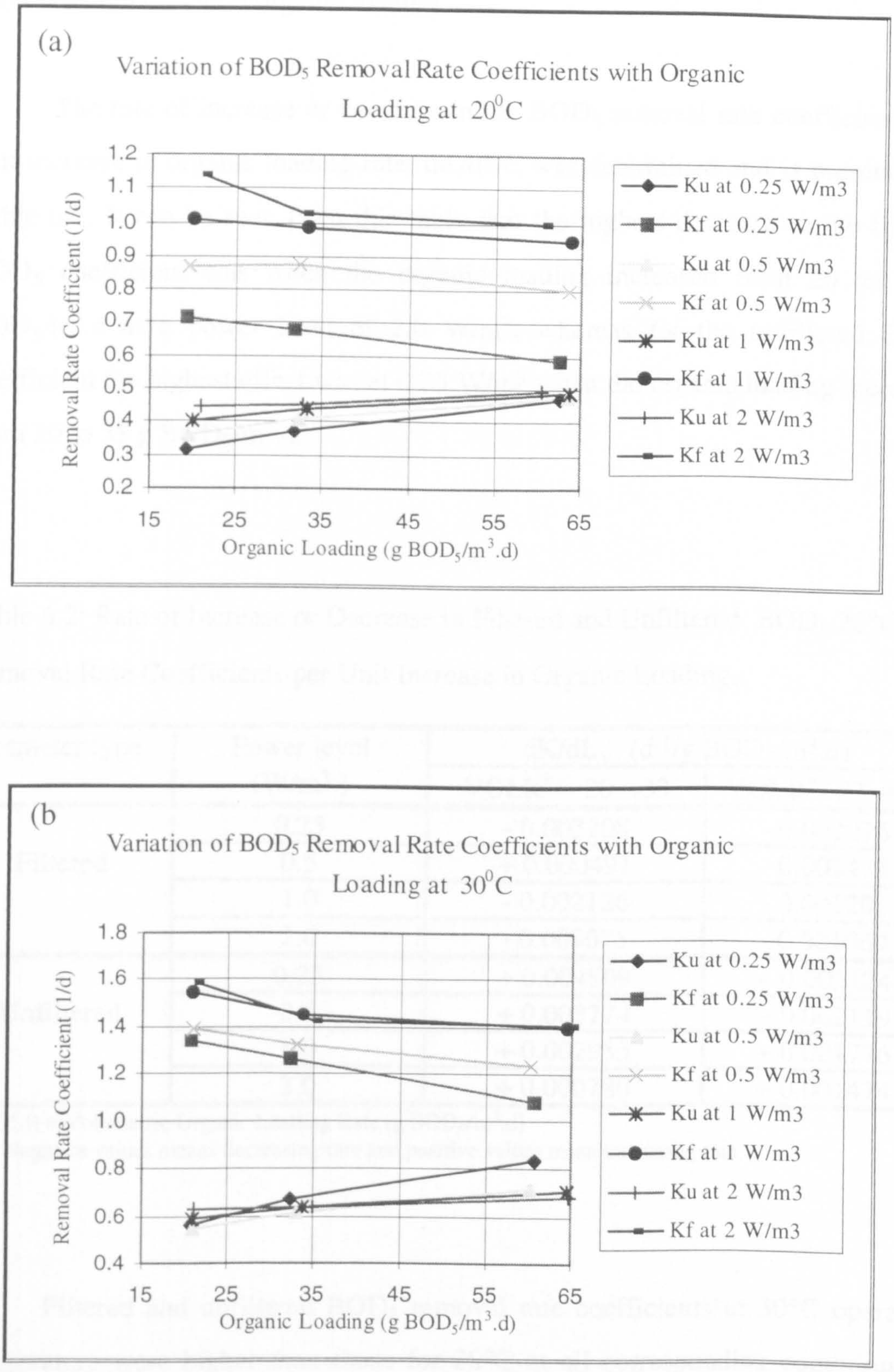


Figure 6.1 Variation of BOD_5 Removal Rate Coefficient with Organic Loading.

loading could be attributed to the higher deposition of solids to the bottom of the lagoon with increase in organic loading.

The rate of increase or decrease in the BOD_5 removal rate coefficients per unit increase in organic loading rate, dK/dLv, was determined and is presented in Table 6.2. It can be seen from this table that the highest decrease in the filtered BOD_5 coefficient was when the organic loading increased from 20 to 33 g $BOD_5/m^3.d$ at a power level of 2.0 W/m^3, whereas for the unfiltered BOD_5 coefficient the highest effect was at 0.25 W/m^3 when the organic loading increased from 20 to 33 g $BOD_5/m^3.d$.

Table 6.2: Rate of Increase or Decrease in Filtered and Unfiltered BOD_5 20°C Removal Rate Coefficients per Unit Increase in Organic Loading.

Parameter type	Power level (W/m^3)	dK/dL_V ($d^{-1}/g\ BOD_5/m^3.d$)	
		VOLR* 20→33	VOLR* 33→62
Filtered	0.25	- 0.003208	- 0.002976
	0.5	+ 0.000491	- 0.002478
	1.0	- 0.002126	- 0.001201
	2.0	- 0.009075	- 0.001069
Unfiltered	0.25	+ 0.004809	+ 0.003124
	0.5	+ 0.003774	+ 0.002119
	1.0	+ 0.002955	+ 0.001776
	2.0	+ 0.000780	+ 0.001474

* VOLR = Volumetric Organic Loading Rate (g $BOD_5/m^3.d$)
NB Negative values means decreasing rate and positive values mean increasing rate

Filtered and unfiltered BOD_5 removal rate coefficients at 30°C operating temperature were higher than those for 20°C at all corresponding conditions of power level and organic loading rate (Table 6.1).

Figure 6.1b shows the variations of filtered and unfiltered BOD_5 removal rate coefficient with volumetric organic loading rate at fixed levels of power input at 30°C. They were affected at different levels by organic loading, with a trend similar to those at 20°C. Similar explanations for the effect of organic loading on the 30°C filtered BOD_5 removal rate coefficient can be given, as for the 20°C coefficients. The 30°C unfiltered BOD_5 removal rate coefficient varied more widely than the 20°C unfiltered BOD_5 coefficient. Reviewing the values of dK/dLv in Table 6.3, the most affected unfiltered BOD_5 coefficient at this operating temperature (30°C) was when the power level was 0.25 W/m^3 and organic loading increased from 20 to 33 g BOD_5/m^3.d.

Table 6.3: Rate of Increase or Decrease in Filtered and Unfiltered BOD_5 30°C Removal Rate Coefficients per Unit Increase in Organic Loading.

Parameter type	Power level (W/m^3)	dK/dL_V ($d^{-1}/g\ BOD_5/m^3.d$)	
		VOLR* 20→33	VOLR* 33→62
Filtered	0.25	- 0.006233	- 0.005859
	0.5	- 0.006340	- 0.003055
	1.0	- 0.008471	- 0.001568
	2.0	- 0.013360	- 0.000528
Unfiltered	0.25	+ 0.009572	+ 0.005325
	0.5	+ 0.005464	+ 0.003789
	1.0	+ 0.004912	+ 0.002278
	2.0	+ 0.001174	+ 0.001228

* VOLR = Volumetric Organic Loading Rate
NB Negative values means decreasing rate and positive values means increasing rate

The effect of organic loading on filtered and unfiltered BOD_5 removal rate coefficients increased with increase in temperature, evidenced by the higher values of dK/dLv at 30°C (Table 6.3) compared with those at 20°C (Table 6.2).

From the figures and tables above it can be said that, for BOD_5 removal, the effect of organic loading rate was generally to affect inversely the filtered BOD_5 removal rate coefficient and affect positively the unfiltered BOD_5 removal rate coefficient. The effect increased with increase in power level for the filtered coefficient and with decrease in power level for the unfiltered coefficient. These effects were higher at 30°C temperature operation than at 20°C. The higher rate of solids entering the system at higher loadings and the increase in solids deposition at lower power levels benefited the unfiltered BOD_5 removal rate coefficient with increase in organic loading. However, increase in the solids deposited increased the soluble BOD_5 feedback rate from the bottom of the lagoon, resulting in reduced filtered BOD_5 removal rate coefficients. Thirumurthi (1980) found that there was a positive effect of organic loading on unfiltered BOD_5 removal rate coefficient, which is in agreement with the findings of this research.

The relationship between organic loading and BOD_5 removal rate coefficients was determined in the form of empirical equations. The following models were derived using the data from the experimental work:

$$BOD_5\ K_{20_f} = 1.234\ L_V^{-0.0935} \qquad R^2 = 0.97 \qquad (6.2)$$

$$BOD_5\ K_{20_u} = 0.1959\ L_V^{0.2193} \qquad R^2 = 0.995 \qquad (6.3)$$

$$BOD_5\ K_{30_f} = 2.0674\ L_V^{-0.117} \qquad R^2 = 0.993 \qquad (6.4)$$

$$BOD_5\ K_{30_u} = 0.3066\ L_V^{0.211} \qquad R^2 = 0.996 \qquad (6.5)$$

ii) Percentage Removals

The effect of organic loading rate on percentage BOD_5 removals was also investigated and the findings are shown in Figures 6.2a and 6.2b. Two forms of percentage removal were utilized in describing the performance of the facultative

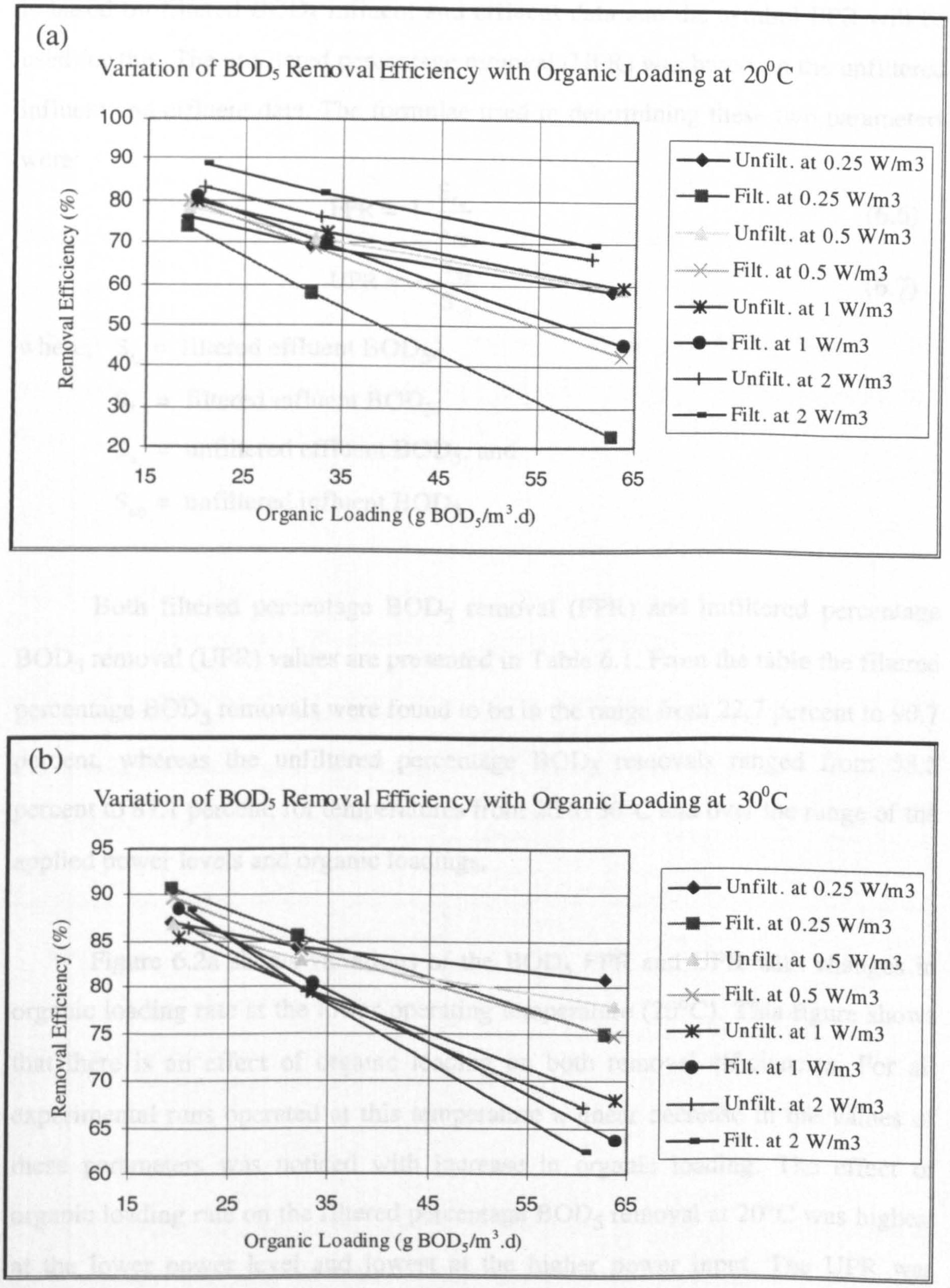


Figure 6.2 Variation of BOD_5 Removal Efficiency with Organic Loading.

aerated lagoon model. The first was the filtered percentage BOD_5 removal, which is based on filtered BOD_5 influent and effluent data and the symbol FPR will be used for this. The unfiltered percentage removal (UPR) was based on the unfiltered influent and effluent data. The formulae used in determining these two parameters were:

$$FPR = 1 - \frac{S_{fe}}{S_{f0}} \tag{6.6}$$

$$UPR = 1 - \frac{S_{ue}}{S_{u0}} \tag{6.7}$$

where, S_{f_e} = filtered effluent BOD_5,

S_{f0} = filtered influent BOD_5,

S_{u_e} = unfiltered effluent BOD_5, and

S_{u0} = unfiltered influent BOD_5

Both filtered percentage BOD_5 removal (FPR) and unfiltered percentage BOD_5 removal (UPR) values are presented in Table 6.1. From the table the filtered percentage BOD_5 removals were found to be in the range from 22.7 percent to 90.7 percent, whereas the unfiltered percentage BOD_5 removals ranged from 58.5 percent to 87.1 percent, for temperatures from 20 to 30°C and over the range of the applied power levels and organic loadings.

Figure 6.2a shows variations of the BOD_5 FPR and UPR with changes in organic loading rate at the lower operating temperature (20°C). This figure shows that there is an effect of organic loading on both removal efficiencies. For all experimental runs operated at this temperature a linear decrease in the values of these parameters was noticed with increase in organic loading. The effect of organic loading rate on the filtered percentage BOD_5 removal at 20°C was highest at the lower power level and lowest at the higher power input. The UPR was inversely affected by organic loading rate, as in the case of FPR, but with UPR this effect varied slightly with change in power level.

Unfiltered percentage removal at 20°C was confined to the range 58.5% to 83%, which was less than that for filtered percentage removal (22.7% to 88.8%) at the same temperature. The rates of decrease or increase in values of the 20°C percentage BOD_5 removals per unit increase in organic loading, dFPR/dLv and dUPR/dLv, are presented in Table 6.4. The most affected BOD_5 FPR and UPR at this temperature (20°C) were observed at 0.25 W/m^3 power level when the organic loading rate was increased from 20 g $BOD_5/m^3.d$ to 33 g $BOD_5/m^3.d$. The conditions under which the effect of increase in organic loading rate on both efficiencies was lowest were 2 W/m^3 power level when organic loading increased from 33 g $BOD_5/m^3.d$ to 62 g $BOD_5/m^3.d$.

Table 6.4: Rate of Decrease or Increase in Filtered and Unfiltered BOD_5 Percentage Removal per Unit Increase in Organic Loading Rate at 20°C·

Parameter type	Power level (W/m^3)	dK/dL_V (% /g $BOD_5/m^3.d$)	
		VOLR* 20→33	VOLR* 33→62
Filtered	0.25	- 1.371473	- 1.140041
	0.5	- 0.859107	- 0.865706
	1.0	- 0.868421	- 0.854389
	2.0	- 0.605692	- 0.413440
Unfiltered	0.25	- 0.596112	- 0.344281
	0.5	- 0.557976	- 0.407795
	1.0	- 0.562500	- 0.458770
	2.0	- 0.582231	- 0.333948

* VOLR = Volumetric Organic Loading Rate

NB Negative values mean decreasing rate and positive values mean increasing rate

The effect of organic loading rate on unfiltered percentage BOD_5 removal at 20°C was less than that for filtered percentage BOD_5 removal and this difference decreased with increase in power level.

For the 30°C experimental runs, the effect of organic loading rate on both filtered and unfiltered percentage BOD_5 removals was also determined. FPR at 30° C temperature ranged from 62.5 percent to 90.6 percent, whereas UPR was found to be in the range from 67.1% to 87.1%. Figure 6.2b reveals that the organic loading rate had an inverse effect, as in the case at 20°C, but to a less extent. This negative effect of organic loading increased with increase in power level. Table 6.5 presents values for dFPR/dLv and dUPR/dLv parameters at this temperature (30° C). This table illustrates that the BOD_5 FPR and UPR at 30°C most affected by increase in organic loading rate was at 1 W/m^3 power level, when organic loading increased from 20 g BOD_5/m^3.d to 33 g BOD_5/m^3.d, unlike that at 20°C which occurred at 0.25 W/m^3 at the same loading range. The lowest affected BOD_5 FPR and UPR were at 0.25 W/m^3 when organic loading rate increased from 33 g BOD_5/m^3.d to 62 g BOD_5/m^3.d.

Table 6.5: Rate of Decrease or Increase in Filtered and Unfiltered BOD_5 Percentage Removal per Unit Increase in Organic Loading Rate at 30°C·

Parameter type	Power level (W/m^3)	$d\%/dL_V$ (% /g BOD_5/m^3.d)	
		VOLR* 20→33	VOLR* 33→62
Filtered	0.25	- 0.430108	- 0.334909
	0.5	- 0.471554	- 0.353839
	1.0	- 0.737463	- 0.542392
	2.0	- 0.725900	- 0.577017
Unfiltered	0.25	- 0.236137	- 0.111250
	0.5	- 0.324749	- 0.181021
	1.0	- 0.565119	- 0.363471
	2.0	- 0.556855	- 0.419346

* VOLR = Volumetric Organic Loading Rate

NB Negative values mean decreasing rate and positive values mean increasing rate

Organic loading affected the filtered BOD_5 removal efficiencies more than the unfiltered ones especially at lower power levels. Comparing Tables 6.4 and

6.5, a higher effect of organic loading can be seen at 20°C than at 30°C and this difference is more pronounced at the lowest organic loading.

iii) Temperature Coefficient

The effect of organic loading rate on the change in BOD_5 removal due to change in temperature in the system was examined in this study. This effect can be described by the temperature coefficient, θ, which is commonly used to adjust the removal rate coefficient to changes in temperature. The basic temperature-correction equation which is derived from the van't Hoff-Arrhenius relationship is:

$$K_2 / K_1 = \theta^{(T_2 - T_1)}. \tag{6.8}$$

where, K_1 = removal rate coefficient at T_1 temperature, and

K_2 = removal rate coefficient at T_2 temperature.

The two levels of temperature applied in this research (20°C and 30°C) were used to estimate the values of temperature coefficients. For both types of BOD_5 removal rate coefficients (K_f and K_u) the temperature coefficient was determined. Filtered BOD_5 temperature coefficients (θ_f) were found from the correction equation used for the filtered removal rate coefficient, whereas unfiltered temperature coefficients (θ_u) were found from unfiltered removal rate coefficients. Both coefficients worked out to be relatively high values and this could be due to the low concentration of the mixed liquor suspended solids, which makes the system sensitive to temperature. Sawyer (1968) justified high values for the temperature coefficients for aerated lagoons, compared with those for activated sludge processes, as being due in the latter to the mass inoculations with preformed organisms unlike the situation in aerated lagoons. Temperature coefficients found in this study were lower than the range (1.06 - 1.09) reported by Eckenfelder (1968) for facultative aerated lagoons but were consistent with those found by Thirumurthi (1979).

The effects of organic loading rate on both BOD_5 temperature coefficients, θ_f and θ_u, are illustrated graphically in Figure 6.3. For θ_f the effect of organic loading was lowest at 0.25 W/m^3 compared with the runs operated at the higher power levels. For the unfiltered BOD_5 temperature coefficient, θ_u, the small changes with organic loading rate in the value of this coefficient at 1 W/m^3 power level, indicate that there was no clear direct effect of organic loading on BOD_5 unfiltered temperature coefficient at this power level. Unfiltered BOD_5 temperature coefficient, θ_u, was most affected by organic loading at 2 W/m^3 power level.

BOD_5 θ_f and θ_u values for all experimental runs were found to have similar ranges, but without any direct relationships with respect to operating conditions. It was found that there were no clear trends in the effect of organic loading rate for either coefficient value.

iv) Effluent Quality

Effluent quality is of major importance in the field of environmental protection. All discharge standards prescribe limits on the concentrations of pollutants which can be discharged to rivers, to land or for reuse applications. Accordingly, effluent quality from the facultative aerated lagoon model was examined and its variations due to the effects of power input, organic loading rate and temperature were studied. Effluent samples from the model aerated lagoon were analysed for BOD_5 during the course of each experimental run.

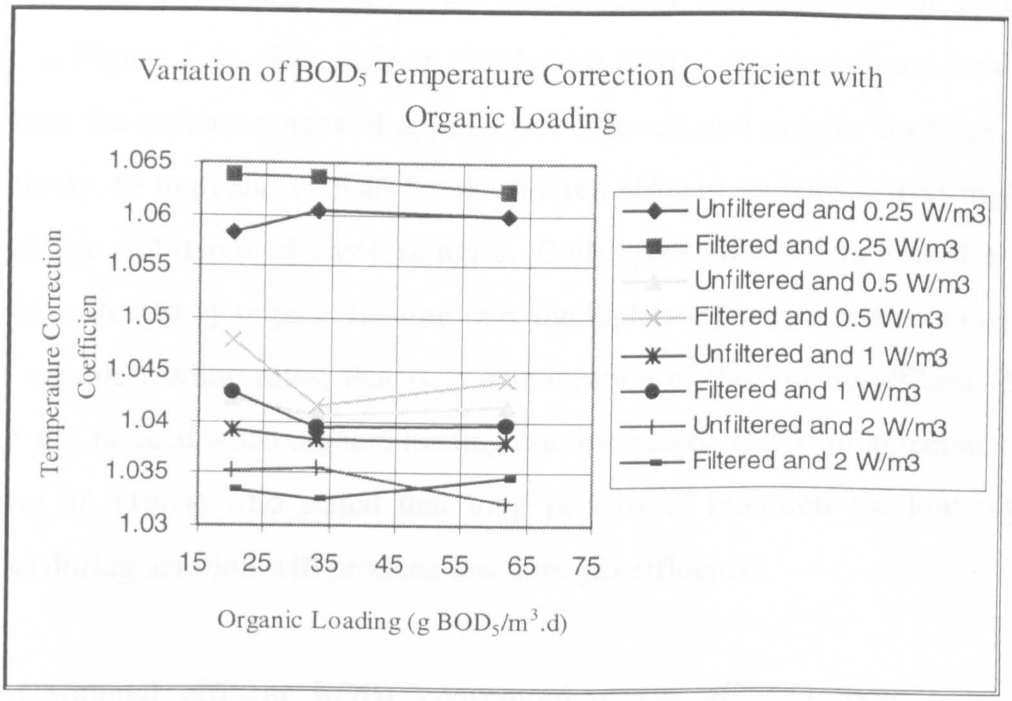


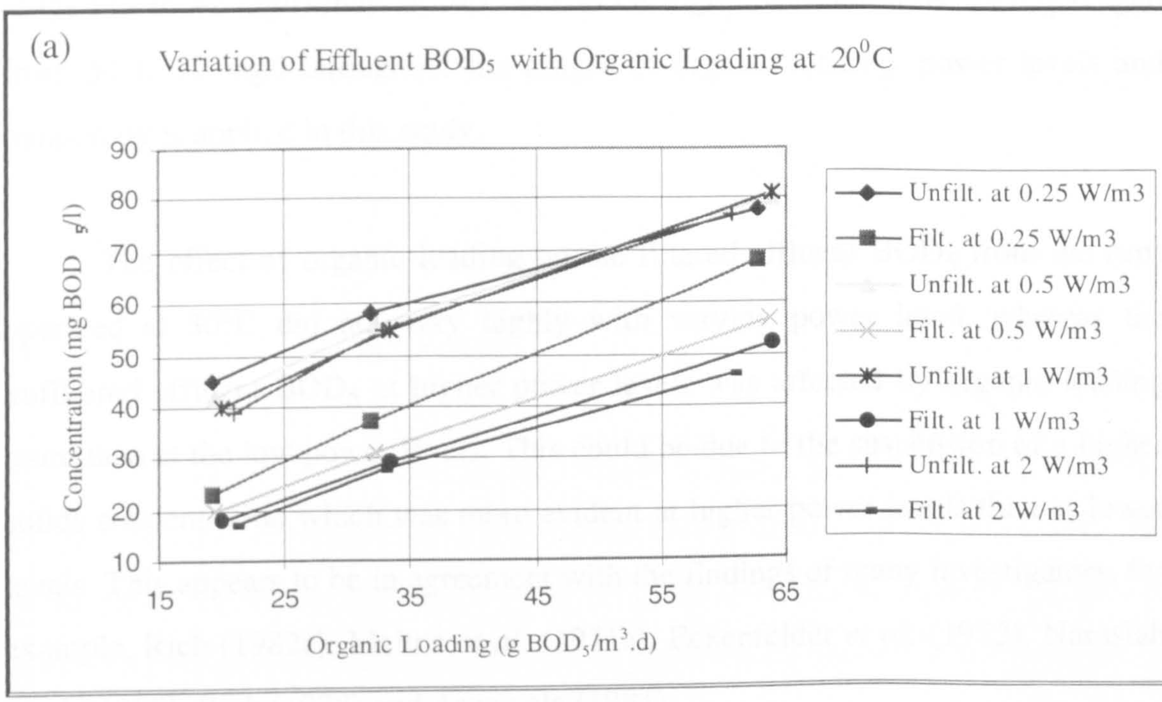
Figure 6.3 Variation of BOD_5 Temperature Correction Coefficient with Organic Loading.

The effect of organic loading rate on both filtered and unfiltered BOD_5 effluent concentrations was studied for both temperature levels (20° and 30°C) and the results are presented graphically in Figure 6.4.

The effect of organic loading rate on filtered and unfiltered BOD_5 concentrations in the effluents of the experimental runs operated at 20°C are shown in Figure 6.4a. The highest and lowest BOD_5 concentrations during the study over the various ranges of applied power levels and organic loadings were, respectively, 68 mg/l and 17 mg/l for the filtered effluent samples and 81 mg/l and 39 mg/l for unfiltered effluent samples. Both types of effluent samples were positively affected by organic loading rate, the higher values being associated with higher organic loading rates; that is, a deterioration of the filtered effluent BOD_5 quality was noticed when organic loading rate increased. This is in agreement with Maris *et al.* (1984) who stated that long periods of retention (or low organic loading) during aeration will produce low strength effluents.

Unfiltered effluent BOD_5 concentration was affected more by a unit increase in organic loading at low organic loadings than at high loadings. This was the same at different levels of aeration power. However, at the lower power level, filtered BOD_5 was affected by organic loading more than at the higher power level. It should be pointed out that this might be an effect of the influent concentrations varying between the sets of runs operated at different power levels, that is, the influent BOD_5 concentrations for the runs operated at 2 W/m^3 were higher than those at 0.25 W/m^3. This effect does not exist at the same power level since the same influent was used.

Effluent quality during the experimental runs operated at 30°C was affected positively by organic loading as at 20°C but to different degrees. The variations of both filtered and unfiltered effluent BOD_5 with organic loading are illustrated in



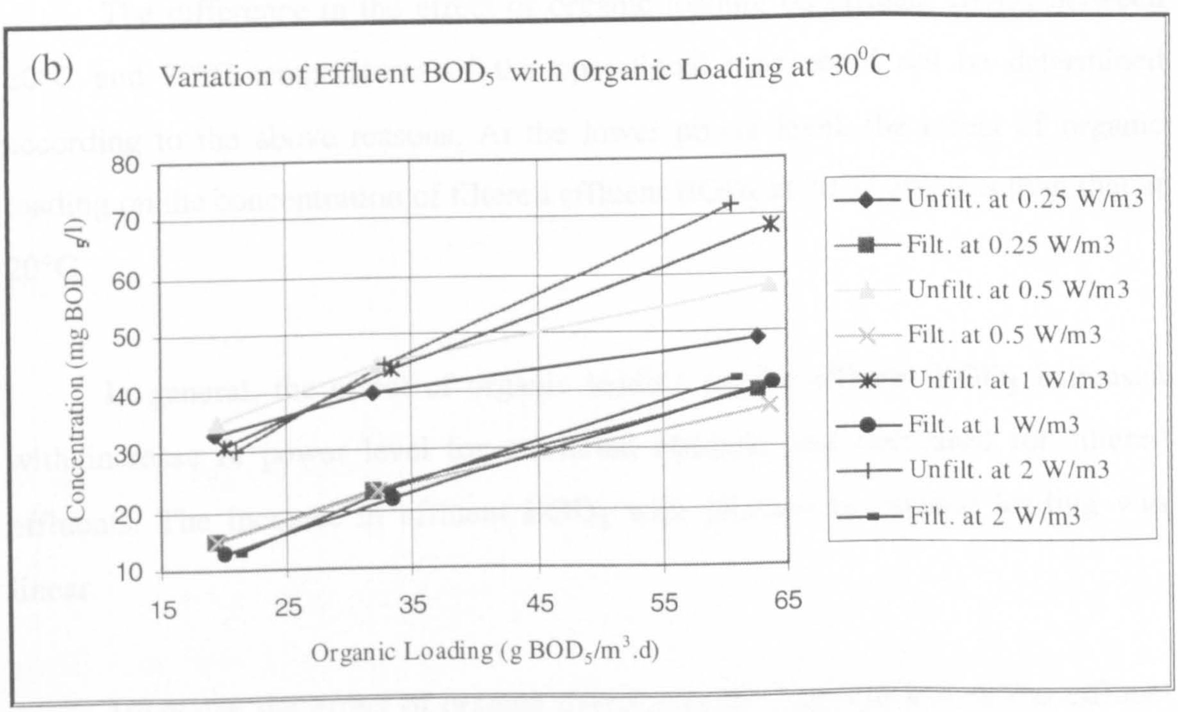


Figure 6.4 Variation of Effluent BOD_5 with Organic Loading.

Figure 6.4b. At this temperature the filtered effluent BOD_5 was confined to a lower range (13 to 42 mg/l) than at 20°C (17 to 68 mg/l). The unfiltered BOD_5 ranged from 30 to 72 mg/l throughout the ranges of organic loading, power levels and temperatures applied in this study.

The effect of organic loading on the filtered effluent BOD_5 from the runs operated at 30°C did not vary highly with varying power level, whereas the unfiltered effluent BOD_5 at higher power levels was affected by organic loading more than at the low power levels. This could be due to the suspension of a higher solids concentration which was more evident at higher power levels than at lower levels. This appears to be in agreement with the findings of many investigators, for example, Rich (1982c), Malina *et al.* (1971a), Eckenfelder *et al.* (1972), Narasiah *et al.* (1987), Rich (1978) and Arceivala (1981).

The difference in the effect of organic loading on effluent BOD_5 between 20°C and 30°C temperatures of the operational runs could not be determined according to the above reasons. At the lower power level, the effect of organic loading on the concentration of filtered effluent BOD_5 at 30°C was less than that at 20°C.

In general, the effect of organic loading on the effluent BOD_5 increased with increase in power level for unfiltered effluents and decreased for filtered effluents. The increase in effluent BOD_5 with increase in organic loading was linear.

However, the effect of organic loading on the improvement of the effluent BOD_5 quality when two hours settling is provided could be expected to insignificant for the range applied in this research. This expected effect of organic loading was actually dependent on the extrapolation of the effluent COD data for samples that were tested when two hours settling was provided (section 6.3). The expected improvement is in the range of 7 to 18% for all the effluent samples operated at the different levels of organic loading, power level, and temperature applied in this research.

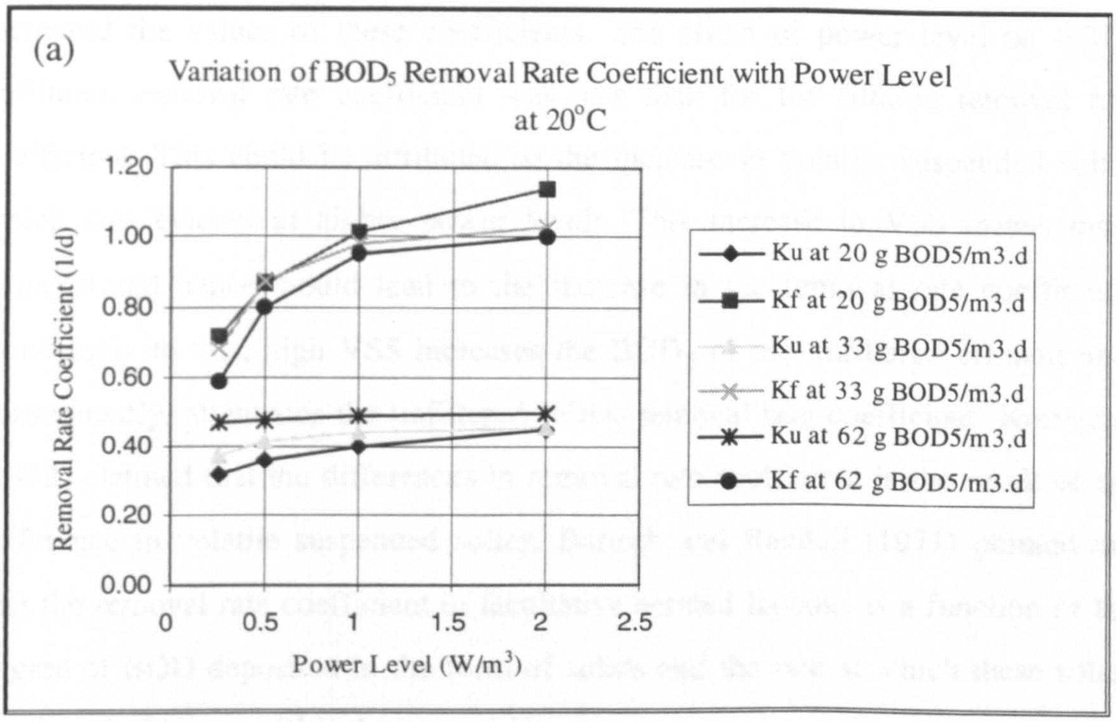
6.2.2 Effects of Power Level Input

This operating variable of the facultative aerated lagoon was one of the variable parameters utilized in this research to determine its effect on the performance of the lagoon. Various levels of aeration power input to facultative aerated lagoons have been recommended by different researchers and the degree of mixing has been claimed to have a high effect on the performance of aerated lagoons (Malina *et al.*, 1971a). Hence it was expected that power level would have a great effect on the performance efficiency of facultative aerated lagoons.

Variations of BOD_5 removal rate coefficients, percentage removals, temperature correction coefficient and effluent quality with power level are presented below.

i) Removal Rate Coefficients

The effect of power level on BOD_5 filtered and unfiltered removal rate coefficients was also investigated, with the results being shown graphically in Figures 6.5a and 6.5b. It can be seen that both BOD_5 removal rate coefficients vary with change in power level.



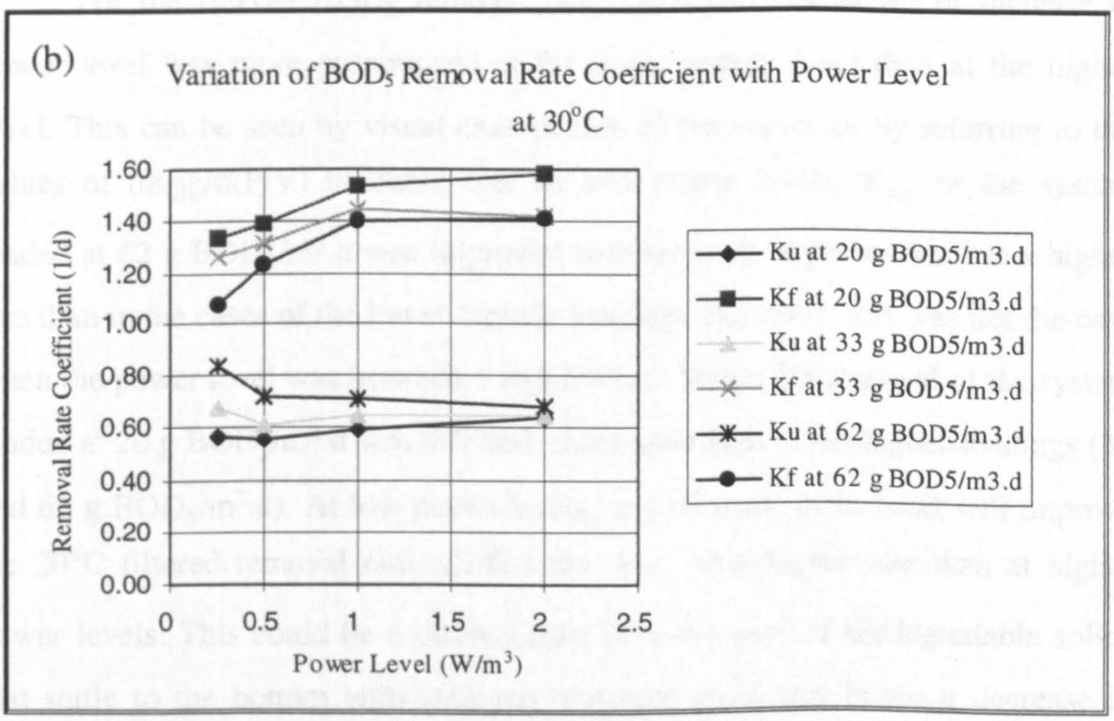


Figure 6.5 Variation of BOD_5 Removal Rate Coefficient with Power Level.

For both coefficients, K_{20_f} and K_{20_u}, increase in power level (Figure 6.5a) increased the values of these coefficients. The effect of power level on BOD_5 unfiltered removal rate coefficient was less than for the filtered removal rate coefficient. This could be attributed to the increase in volatile suspended solids which was evident at higher power levels. This increase in VSS means more biomass and hence would lead to the increase in the removal rate coefficient. Conversely to this, high VSS increases the BOD_5 of the unfiltered effluent and, consequently, attenuates the unfiltered BOD_5 removal rate coefficient. Kormanik (1972) claimed that the differences in removal rate coefficient is the result of the difference in volatile suspended solids. Bartsch and Randall (1971) pointed out that the removal rate coefficient in facultative aerated lagoons is a function of the degree of BOD deposited in the form of solids and the rate at which these solids are liquefied by anaerobic decomposition.

For the filtered BOD_5 removal rate coefficient, the effect of increase in power level was more pronounced at the lower energy level than at the higher level. This can be seen by visual examination of the curves or by referring to the values of $dK_{20}/d(P/V)$ in Table 6.6. At low power levels, K_{20_f} of the system loaded at 62 g BOD_5/m^3.d was improved with increase in power level at a higher rate than in the cases of the lower organic loadings. However, this was not the case when the power level was between 1 and 2 W/m^3, where the removal of the system loaded at 20 g BOD_5/m^3.d was affected much more than at the higher loadings (33 and 62 g BOD_5/m^3.d). At low power levels, any increase in its level will improve the 20°C filtered removal rate coefficients, K_{20_f}, at a higher rate than at higher power levels. This could be a consequence of a decrease of biodegradable solids that settle to the bottom with increase in power level, and hence a decrease in released soluble materials formed by decomposition. This is supported by Rich (1982c), who postulated that these biodegradable solids that settle will release soluble organics and exert a delayed oxygen demand.

Table 6.6: Rate of Decrease or Increase in Filtered and Unfiltered BOD_5 20°C Removal Rate Coefficients per Unit Increase in Power Level.

Parameter type	VOLR* (g BOD_5/m^3.d)	dK/d(P/V) $d^{-1}/(W/m^3)$ P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	0.610	0.282	0.128
	33	0.785	0.213	0.048
	62	0.852	0.295	0.051
Unfiltered	20	0.177	0.076	0.045
	33	0.149	0.057	0.015
	62	0.020	0.029	0.007

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level (W/m^3)

It can be said, therefore, that the effect of power level on filtered BOD_5 removal rate is high in highly loaded systems when the power level is low, whereas the effect of power level is greater in a lightly loaded system when the power level increases and creates conditions closer to a completely mixed system.

Unfiltered BOD_5 removal rate coefficient, K_{20_u} varied over a narrower range, and the effect of increase in power level was less than in the case of K_{20_f} (Figure 6.5a). It can still be observed that a higher effect is evident when power level was increased over the range 0.25 to 1 W/m^3. Unlike the filtered BOD_5 removal rate coefficient, the effect of increased power level on the unfiltered BOD_5 removal coefficient increased with decrease in organic loading over the whole range of power level.

The effects of power level on both filtered and unfiltered BOD_5 removal rate coefficients at 30°C are presented graphically in Figure 6.5b. It suggests that at 30°C, BOD_5 filtered removal rate coefficient values were positively affected by power level, as for 20°C, but to a lesser extent, whereas the unfiltered BOD_5 removal rate coefficient at this temperature (30°C), except at low loadings and higher power levels, was affected inversely by power level. The difference between the variations in values of these coefficient at 20°C and 30°C could be mainly due to the effect of temperature on the behaviour of such treatment processes.

The rate of decrease or increase in filtered and unfiltered BOD_5 removal rate coefficients per unit increase in power level were determined and presented in Table 6.7.

Table 6.7: Rate of Decrease or Increase in Filtered and Unfiltered BOD_5 30°C Removal Rate Coefficients per Unit Increase in Power Level.

Parameter type	VOLR* (g $BOD_5/m^3.d$)	dK/d(P/V) $d^{-1}/(W/m^3)$		
		P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	0.222	0.299	0.046
	33	0.217	0.264	- 0.027
	62	0.630	0.332	0.006
Unfiltered	20	- 0.052	0.079	0.043
	33	- 0.243	0.058	0.004
	62	- 0.508	- 0.006	- 0.030

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level (W/m^3)

For both BOD_5 removal rate coefficients at both levels of temperature (20° C and 30°C) it can be generalized that a greater effect of power level was observed at lower power levels than at higher. The unfiltered removal rate coefficient was affected by increase in power level to a lesser extent than the filtered removal rate coefficient.

The effect of power level on the filtered BOD_5 removal rate coefficient at 20°C was higher than at 30°C. For the unfiltered BOD_5 removal rate coefficient, the effect of power level at 30°C was higher than at 20°C. These can be seen from comparison of the two figures (Figures 6.5a and 6.5b) or Tables 6.6 and 6.7.

The following empirical equations, which show the relationship between power level and BOD_5 removal rate coefficient, were derived using the data collected during the course of the experimental work:

$$BOD_5\ K_{20_f} = 0.943\ P_V^{0.224} \qquad R^2 = 0.996 \qquad (6.9)$$

$$BOD_5\ K_{20_u} = 0.4408\ P_V^{0.094} \qquad R^2 = 0.989 \qquad (6.10)$$

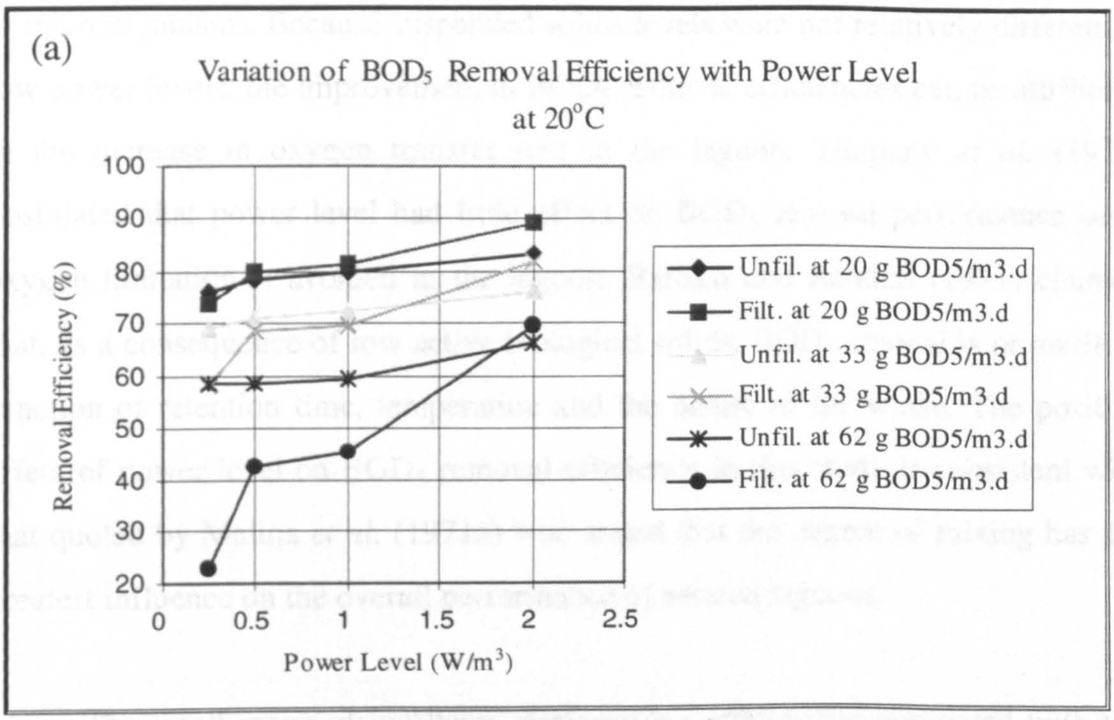
$$BOD_5\ K_{30_f} = 1.409\ P_V^{0.0963} \qquad R^2 = 0.996 \qquad (6.11)$$

$$BOD_5\ K_{30_u} = 0.65\ P_V^{-0.0143} \qquad R^2 = 0.987 \qquad (6.12)$$

ii) Percentage Removals

The effect of power level on BOD_5 removal efficiencies in the model facultative aerated lagoon at two levels of temperature (20°Cand 30°C) was studied and the results are shown in Figures 6.6a and 6.6b. These two figures show the effect of the level of power introduced into the reactor on the BOD_5 removal efficiencies (filtered and unfiltered).

The 20°C BOD_5 removal efficiency is plotted against power level input to the facultative aerated lagoon model in Figure 6.6a. This figure indicates that at 20 °C the BOD_5 removal efficiency increased as aerator power was increased. This can be attributed to the increase in bacterial biomass kept in suspension, which would enhance the bioconversion of biodegradable organic carbon. This increase in the filtered BOD_5 removal efficiency values with power level improves as organic loading increases, whereas the effect of power level on unfiltered removal efficiency, as a result of the increase in power level, do not change with change in organic loading. The low BOD_5 removal efficiencies at low power levels could be attributed to the low level of active biological solids maintained in suspension.



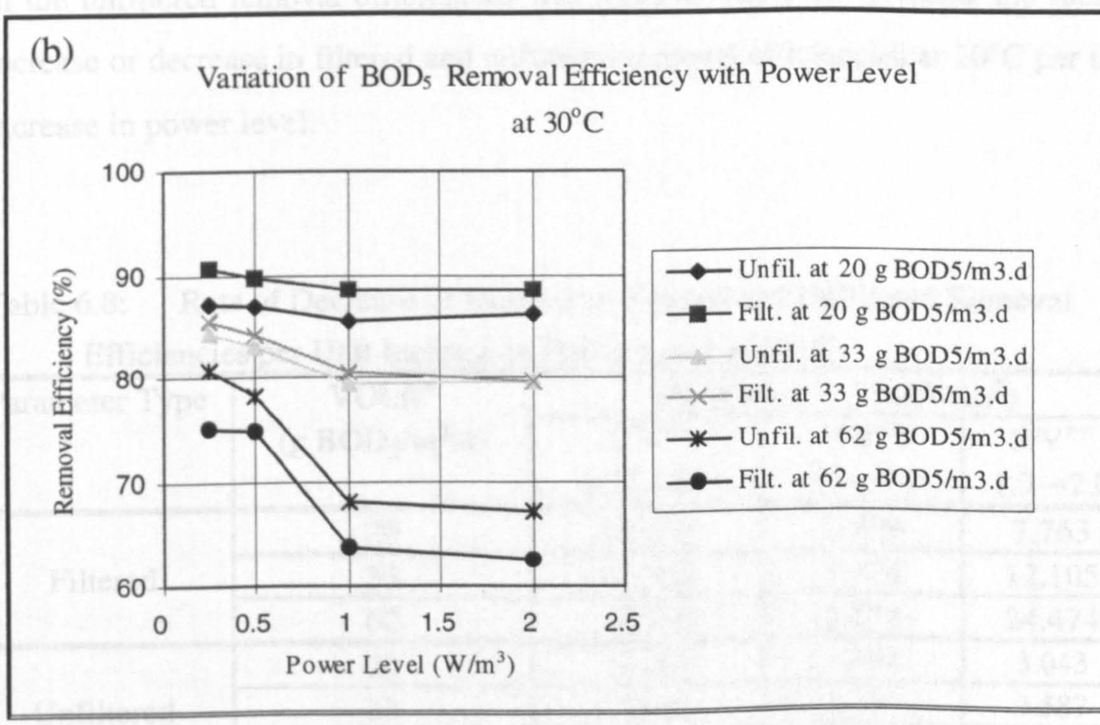


Figure 6.6 Variation of BOD_5 Removal Efficiency with Power Level.

Tikhe (1975) indicated that removal rate coefficient is dependent on the population of microorganisms. Because suspended solids levels were not relatively different at low power levels, the improvement in BOD_5 removal efficiencies can be attributed to the increase in oxygen transfer rate in the lagoon. Timpany *et al.* (1971) postulated that power level had little effect on BOD_5 removal performance once oxygen limitation is avoided in the lagoon. Bartsch and Randall (1971) claimed that, as a consequence of low active biological solids, BOD removal is primarily a function of retention time, temperature and the nature of the waste. The positive effect of power level on BOD_5 removal efficiency in this study is consistent with that quoted by Malina *et al.* (1971a) who stated that the degree of mixing has the greatest influence on the overall performance of aerated lagoons.

The small range of unfiltered performance efficiencies compared with the filtered efficiencies at 20°C means that the effect of the power level on the changes in the unfiltered removal efficiencies was smaller. Table 6.8 presents the rate of increase or decrease in filtered and unfiltered removal efficiencies at 20°C per unit increase in power level.

Table 6.8: Rate of Decrease or Increase in Filtered and Unfiltered Removal Efficiencies per Unit Increase in Power Level at 20° C.

Parameter Type	VOLR* ($g BOD_5/m^3.d$)	d%/d(P/V) %/(W/m^3)		
		P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	23.737	2.509	7.763
	33	42.929	1.574	12.105
	62	78.788	5.678	24.474
Unfiltered	20	9.147	3.299	3.043
	33	7.940	2.732	3.587
	62	1.009	1.474	7.022

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level (W/m^3)

Figure 6.6b shows the variations of both BOD_5 removal efficiencies with power level at 30°C. This figure reveals the negative effect of power level on these two BOD_5 removal efficiencies. The negative effect of power level increases with increase in organic loading. The rate of increase or decrease in both parameters per unit increase in power level for the different loading levels for runs operated at this temperature (30°C) are presented in Table 6.9.

Table 6.9: Rate of Decrease or Increase in Filtered and Unfiltered Removal Efficiencies per Unit Increase in Power Level at 30° C.

Parameter Type	VOLR* (g BOD_5/m^3.d)	d%/d(P/V) %/(W/m^3)		
		P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	- 3.596	- 2.461	- 0.103
	33	- 5.514	- 7.431	- 1.067
	62	- 1.370	- 21.881	- 1.217
Unfiltered	20	- 1.065	- 2.693	0.855
	33	- 5.179	- 7.352	0.109
	62	- 10.684	- 20.076	- 0.952

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level (W/m^3)
NB Negative values means decreasing rate and positive values means increasing rate

The runs most affected by change in power level in terms of filtered BOD_5 removal efficiency were those operated at 62 g BOD_5/m^3.d organic loading and 20 °C temperature and the least effected ones were those operated at 20 g BOD_5/m^3.d, high power levels and 30°C. For unfiltered BOD_5 removal efficiency, the runs least affected by increase in power level were those operated at 20 g BOD_5/m^3.d and 30°C, whereas the most affected runs were those operated at 62 g BOD_5/m^3.d and 30°C.

At 20°C both BOD_5 removal efficiencies were positively affected by increase in power level, whereas, at 30°C they were inversely affected. Both BOD_5

removal efficiencies varied with power level over a smaller range at 30°C than at 20°C which means that the effect of power level was higher at lower temperature than at higher temperature. Comparing the changes in the filtered BOD_5 percentage removal due to changes in power level at the two levels of temperature, it can be said that the increased temperature had reduced the effect of power level.

From the foregoing discussion, the improvement in filtered BOD_5 percentage removal per unit increase in power level at 20°C increases with increase in organic loading. Whereas at 30°C a reduction in the filtered BOD_5 percentage removal value occurred with increase in power level, and this reduction increased with increase in organic loading. The variations in the unfiltered BOD_5 removal efficiency with change in power level were seen to be less than for filtered BOD_5 effluent samples.

iii) Temperature Coefficients

For biochemical oxygen demand the variation of temperature correction coefficients,θ_f and θ_u, due to the change of the aeration power level are presented graphically in Figure 6.7.

Power level affected both temperature coefficients negatively, as is clear from the figure. The highest effect occurred at low power level, whereas the decrease in the values of both temperature correction coeficients due to increase in power level decreased at high power levels.

At the lower power level, operational runs performed at 33 g $BOD_5/m^3.d$ organic loading produced greater effects on both temperature correction coeficients than at other loadings. However, at high levels of power, θ_f for operational runs

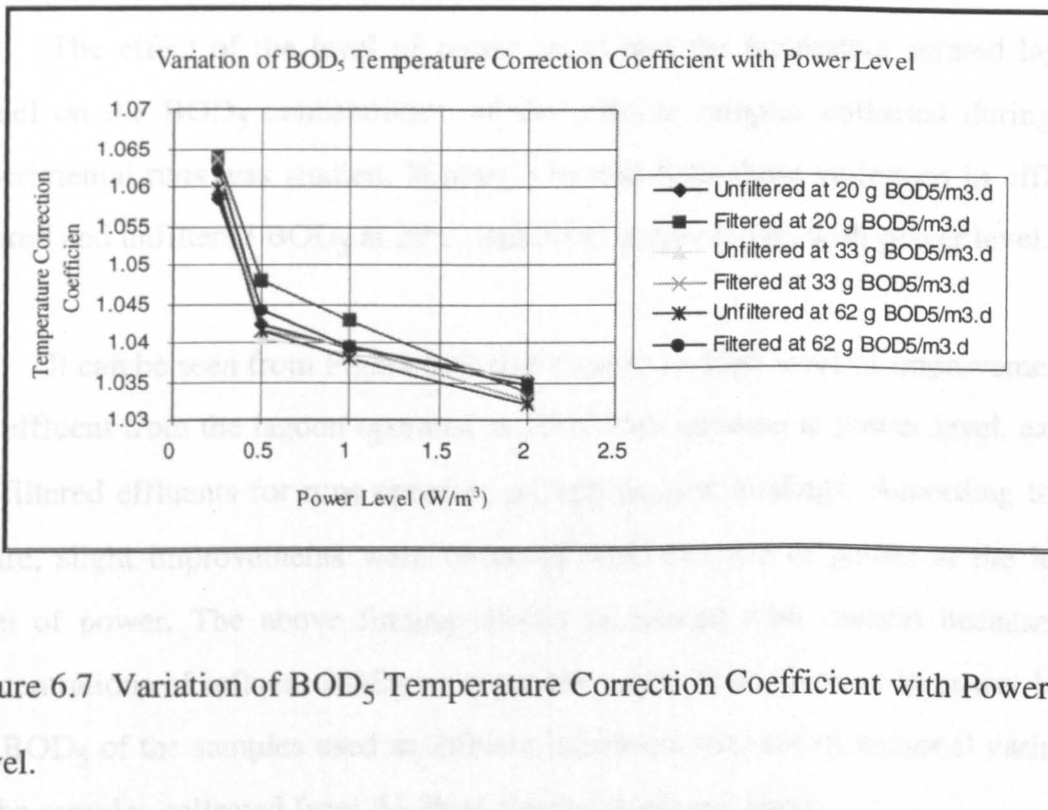


Figure 6.7 Variation of BOD_5 Temperature Correction Coefficient with Power Level.

operated at 20 g BOD_5/m^3.d and θ_u for operational runs operated at 62 g BOD_5/m^3.d, the negative effect of power level was highest.

iv) Effluent Quality

The effect of the level of power input into the facultative aerated lagoon model on the BOD_5 concentration of the effluent samples collected during the experimental runs was studied. Figures 6.8a and 6.8b show variations in effluent filtered and unfiltered BOD_5 at 20°C and 30°C, respectively, with power level.

It can be seen from Figure 6.8a that there is no high level of improvement in the effluent from the lagoon operated at 20°C with increase in power level, except for filtered effluents for runs operated at high organic loadings. According to the figure, slight improvements were observed with increase in power at the lower level of power. The above finding should be treated with caution because the concentrations of influent BOD_5 were not the same. With increase in power level, the BOD_5 of the samples used as influent increased, because of seasonal variation of the samples collected from the local sewage treatment plant.

From Figure 6.8b, it can be inferred that the BOD_5 of the filtered effluent at 30°C operating temperature was not much affected by power level, whereas increase in power adversely affected the BOD_5 of unfiltered effluents only at high loadings and this negative effect could be explained as being due to the resuspension of some solids that had settled to the lagoon bottom. The results presented in Figure 6.8b for runs operated at 30°C are not comparable because, at this temperature, the influents were not of the same quality since at low power levels influent BOD_5 were higher. The effect of power level on effluent BOD_5 for

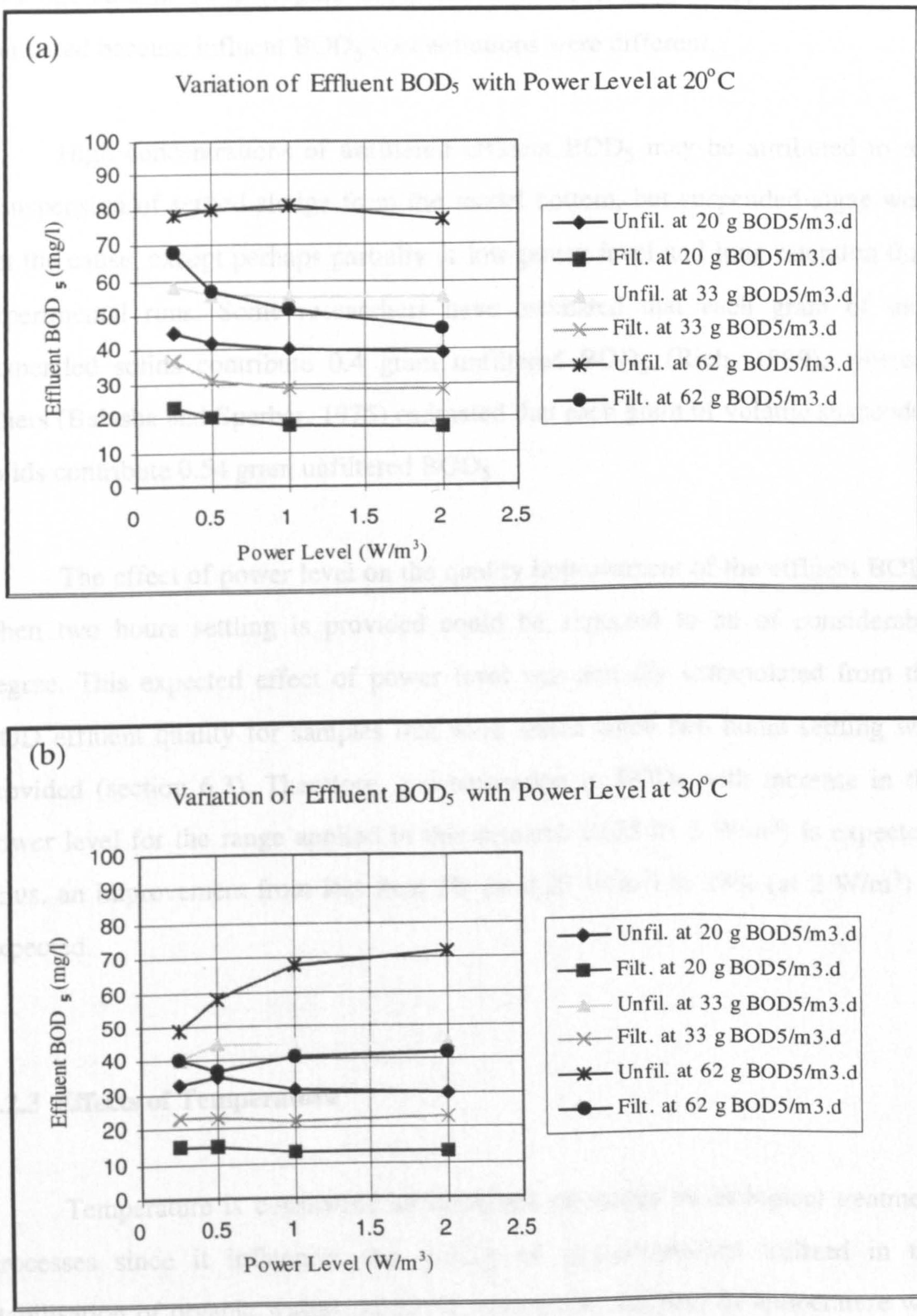


Figure 6.8 Variation of Effluent BOD_5 with Power Level.

runs operated at 20°C (Figure 6.8a) and at 30°C (Figure 6.8b) should not be compared because influent BOD_5 concentrations were different.

High concentrations of unfiltered effluent BOD_5 may be attributed to the resuspension of settled sludge from the model bottom, but suspended algae were not the cause, except perhaps partially at low power level and long retention time experimental runs. Some researchers have estimated that each gram of total suspended solids contribute 0.4 gram unfiltered BOD_5 (Rich, 1989), whereas others (Balasha and Sperber, 1975) estimated that each gram of volatile suspended solids contribute 0.54 gram unfiltered BOD_5.

The effect of power level on the quality improvement of the effluent BOD_5 when two hours settling is provided could be expected to be of considerable degree. This expected effect of power level was actually extrapolated from the COD effluent quality for samples that were tested when two hours settling was provided (section 6.3). Therefore, a deterioration in BOD_5 with increase in the power level for the range applied in this research (0.25 to 2 W/m^3) is expected. Thus, an improvement from less than 7% (at 0.25 W/m^3) to 18% (at 2 W/m^3) is expected.

6.2.3 Effects of Temperature

Temperature is considered an important parameter in biological treatment processes since it influences the activity of microorganisms utilized in the stabilisation of organic wastes; however, conversely, increase in temperature will result in a reduction of the saturation concentration of oxygen in the lagoon contents and with a lesser extent the oxygen transfer rate. Temperature might also influence the effects of organic loading rate and power level on facultative aerated lagoons. Accordingly, the temperature effect on performance of such treatment systems was considered. Two levels of temperature were utilised in order to determine how the removal rate coefficients, removal efficiencies and effluent BOD_5 varied with increase or decrease in temperature level in the facultative aerated lagoon model. Timpany and his colleagues (1971) reported that the BOD_5 removal efficiency of an aerated lagoon is affected by temperature changes in the range 10 to 30°C.

i) Removal Rate Coefficients

Figure 6.9 illustrates the variations of BOD_5 filtered and unfiltered removal rate coefficients with temperature at 20, 33 and 62 g $BOD_5/m^3.d$ organic loadings. These figures reveal that temperature positively affected the removal coefficients. The positive effect of temperature on filtered BOD_5 removal rate coefficients generally increased with decrease in organic loading and power level, whereas this effect of temperature on unfiltered BOD_5 removal rate coefficients increased with increase in organic loading, especially at low power level. At fixed organic loading, the positive effect of temperature on the unfiltered BOD_5 removal rate coefficient decreased with increase in power level. This positive effect of temperature on the removal rate coefficients in facultative aerated lagoons has been noticed for many other biological treatment processes. Narasiah (1983) and other investigators assert the positive effect of temperature of aerated lagoons on the removal rate coefficients. This radical response of facultative aerated lagoons, compared with activated sludge systems, was observed by Sawyer (1968), who justified this by the difference of the inoculated mass of microorganisms.

The effect of temperature on filtered BOD_5 removal rate coefficients was greater than on unfiltered BOD_5 removal rate coefficients, as indicated by the

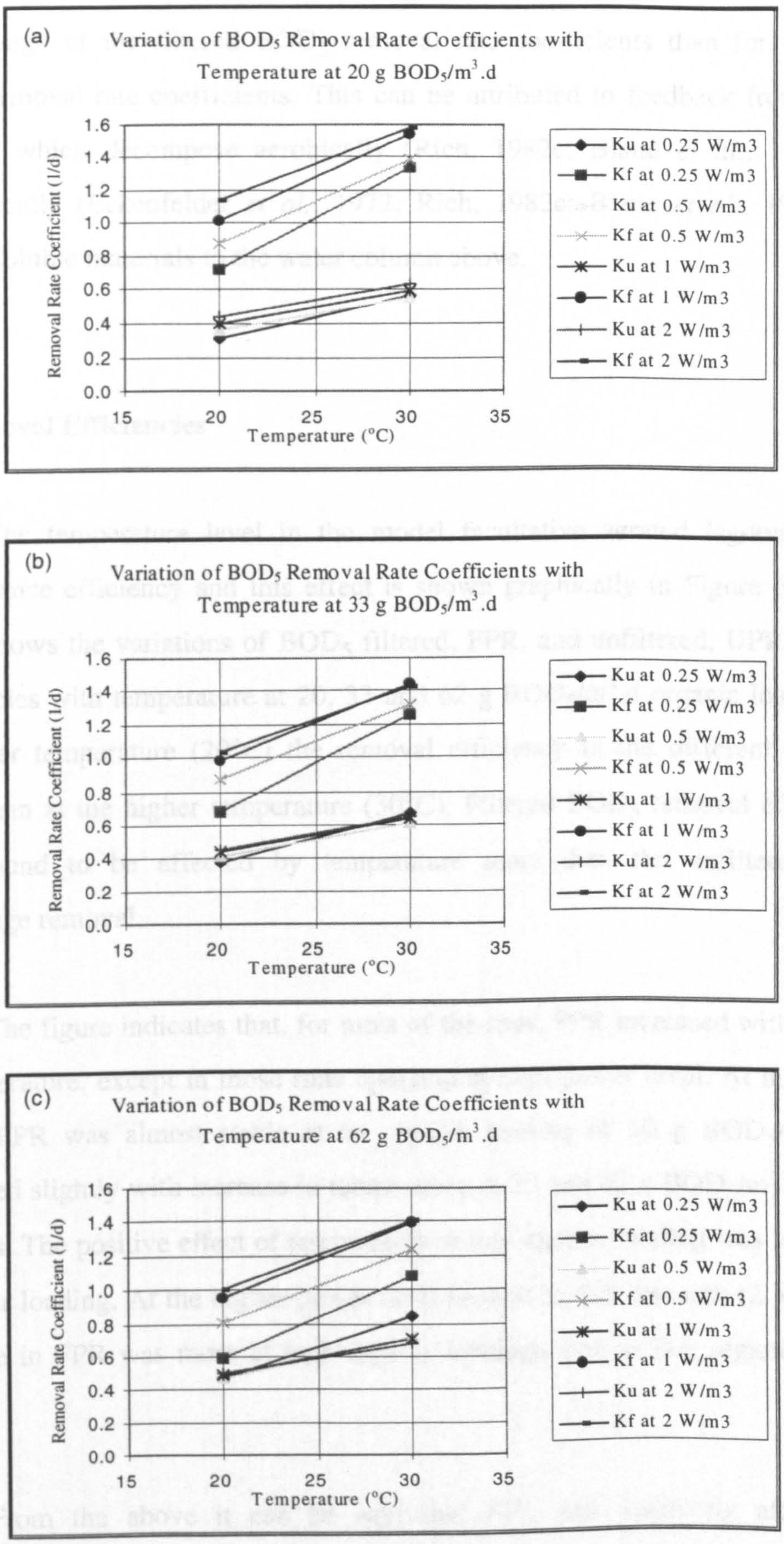


Figure 6.9 Variation of BOD_5 Removal Rate Coefficient with Temperature.

wider range of the filtered BOD_5 removal rate coefficients than for unfiltered BOD_5 removal rate coefficients. This can be attributed to feedback from bottom deposits which decompose aerobically (Rich, 1982c; Blane *et al.*, 1984) and anaerobically (Eckenfelder *et al.*, 1972; Rich, 1982c; Blane *et al.*, 1984), and release soluble materials to the water column above.

ii) Removal Efficiencies

The temperature level in the model facultative aerated lagoon affected performance efficiency and this effect is shown graphically in Figure 6.10. This figure shows the variations of BOD_5 filtered, FPR, and unfiltered, UPR, removal efficiencies with temperature at 20, 33 and 62 g $BOD_5/m^3.d$ organic loadings. At the lower temperature (20°C) the removal efficiency in the different runs was lower than at the higher temperature (30°C). Filtered BOD_5 removal efficiencies were found to be affected by temperature more than the unfiltered BOD_5 percentage removal.

The figure indicates that, for most of the runs, FPR increased with increase in temperature, except in those runs operated at high power level. At high power levels, FPR was almost stable at an organic loading of 20 g $BOD_5/m^3.d$ and decreased slightly with increase in temperature at 33 and 62 g $BOD_5/m^3.d$ organic loadings. The positive effect of temperature at low organic loading was lower than at higher loading. At the higher power level applied in this research (2 W/m^3) the decrease in FPR was more at high organic loadings than at low organic loading rates.

From the above it can be said that FPR was positively affected by temperature level at power levels in the range 0.25 to 1 W/m^3 and this effect

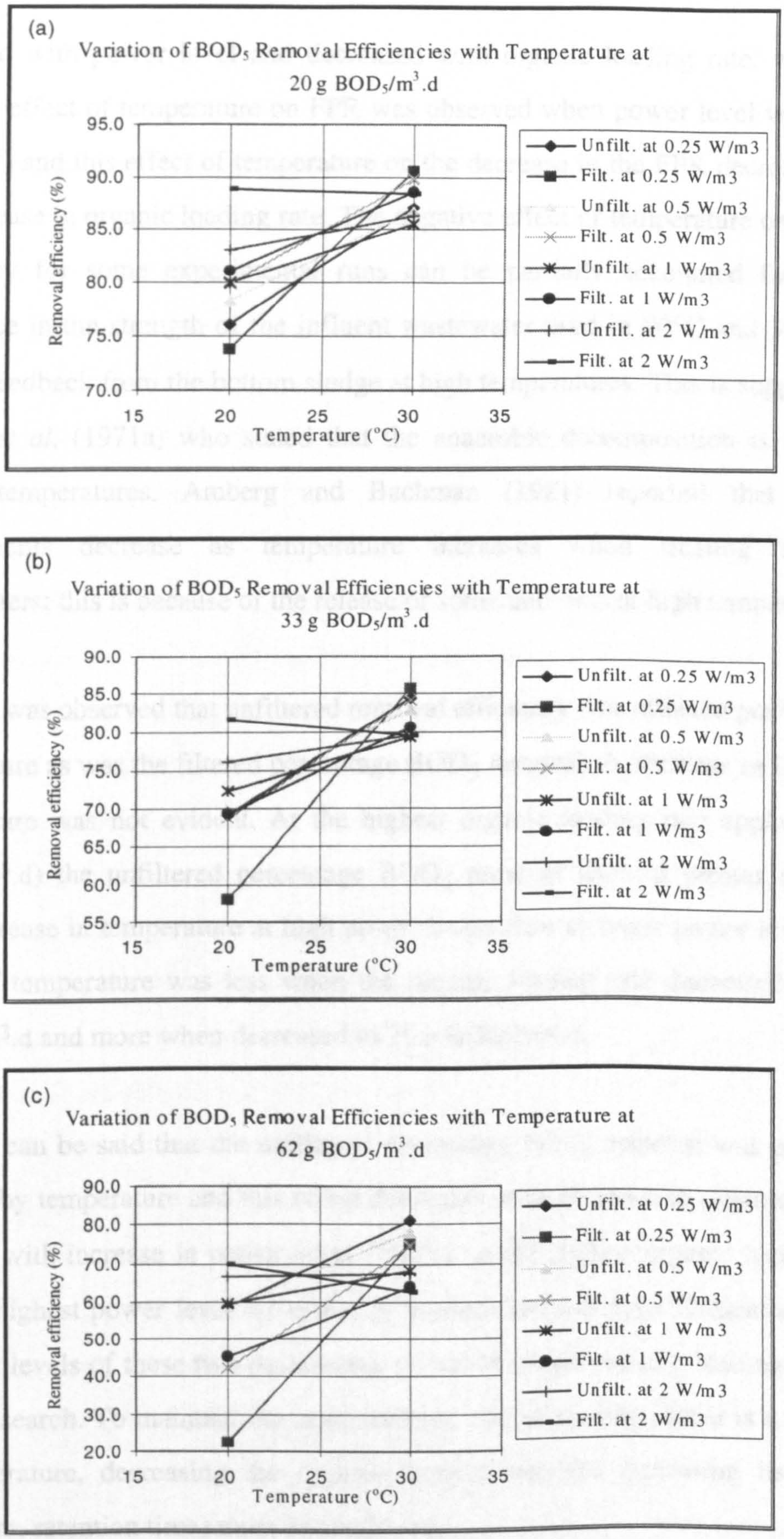


Figure 6.10 Variation of BOD_5 Removal Efficiency with Temperature .

increased with power level and decreased with organic loading rate, whereas a negative effect of temperature on FPR was observed when power level was higher (2 W/m^3) and this effect of temperature on the decrease in the FPR decreased with the decrease in organic loading rate. The negative effect of temperature on removal efficiency for some experimental runs can be partially accounted for by the difference in the strength of the influent wastewater used in 20°C and 30°C tests and by feedback from the bottom sludge at high temperatures. This is supported by Malina *et al.* (1971a) who stated that the anaerobic decomposition is higher at higher temperatures. Amberg and Bachman (1981) reported that nutrient requirements decrease as temperature increases when treating industrial wastewaters; this is because of the release of some nutrients at high temperatures.

It was observed that unfiltered removal efficiency was affected positively by temperature as was the filtered percentage BOD_5 removal. A decrease in UPR with temperature was not evident. At the highest organic loading rate applied (62 g $BOD_5/m^3.d$) the unfiltered percentage BOD_5 removal showed greater influence from increase in temperature at high power levels than at lower power levels. The effect of temperature was less when the organic loading rate decreased to 33 g $BOD_5/m^3.d$ and more when decreased to 20 g $BOD_5/m^3.d$.

It can be said that the unfiltered percentage BOD_5 removal was positively affected by temperature and this effect decreased with decrease in organic loading rate and with increase in power level. That is, at the highest organic loading rate and the highest power level the effect of temperature was least influential among the other levels of these two parameters, power level and organic loading, applied in this research. To maintain the same removal efficiency when there is a decrease in temperature, decreasing the organic loading rate (or increasing its inverse equivalent, retention time) must be employed.

iii) Effluent Quality

Filtered and unfiltered effluent BOD_5 concentration from the facultative aerated lagoon model can be seen (Table 6.1) to be affected by the temperature at which the run was operated. Figures 6.11a, 6.11b and 6.11c show clearly the variations of filtered BOD_5 concentration in the effluent with temperature. The differences between influent concentration at 20°C and 30°C at fixed power level and organic loading make the results less reliable regarding the indication of the effect of temperature on such effluent BOD_5 concentrations.

However, in general, the filtered BOD_5 was seen to decrease with increase in temperature but the magnitude of this effect changed with difference in power level and organic loading. This is in contrast to previous work carried out by Narasiah *et al.* (1987) who found that soluble BOD decreased from summer to winter and attributed this to the effect of temperature on bacterial activity. The effect of increase in temperature on the decrease in filtered effluent BOD_5 concentration was highest at high organic loading and decreased with decrease in organic loading. At the same time, the effect of temperature was highest at low power level and, as power level increased, the effect of temperature decreased. This finding is reliable because for the lowest power level the influent at 20°C had lower BOD_5 concentration than that at 30°C, whereas at the highest power level the influent at 20°C had a higher BOD_5 than that at 30°C.

For the range of power levels and temperatures applied, the highest decrease in the filtered effluent BOD_5 concentration was observed in the run operated at 62 g $BOD_5/m^3.d$ organic loading rate and 0.25 W/m^3 power level. Under these conditions, an improvement of 28 mg/l filtered BOD_5 occurred when the temperature increased from 20 to 30°C. At 20 g $BOD_5/m^3.d$ organic loading rate

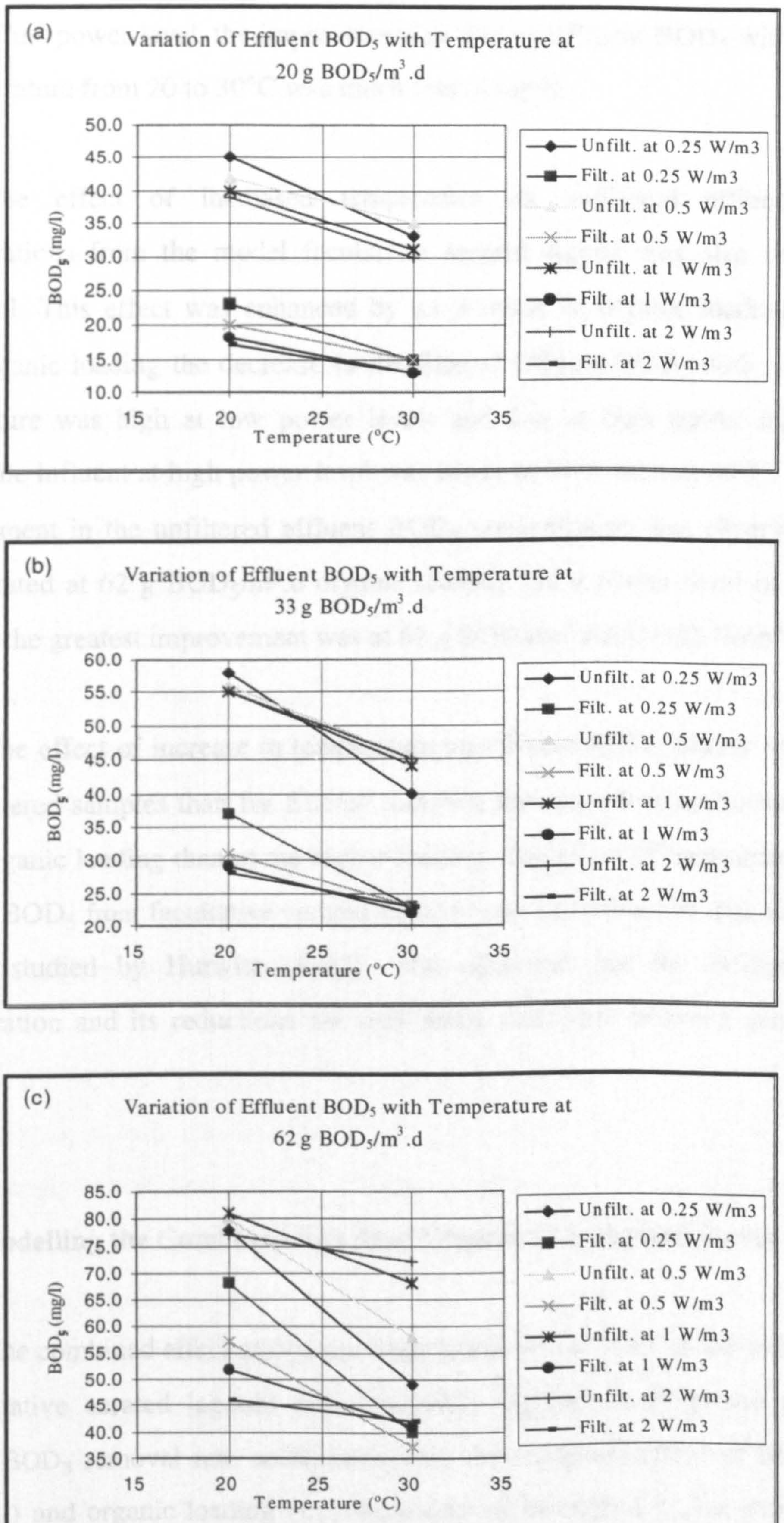


Figure 6.11 Variation of Effluent BOD_5 with Temperature.

and 2 W/m^3 power level, the improvement in filtered effluent BOD_5 with increase in temperature from 20 to 30°C was much less (4 mg/l).

The effect of increased temperature on unfiltered effluent BOD_5 concentrations from the model facultative aerated lagoon was also seen to be beneficial. This effect was enhanced by an increase in organic loading rate. At fixed organic loading the decrease in the filtered effluent BOD_5 with increase in temperature was high at low power levels and low at high power input, even though the influent at high power level was lower at 20°C than at 30°C. The least improvement in the unfiltered effluent BOD_5 concentration was observed in the run operated at 62 g BOD_5/m^3.d organic loading and a power level of 2 W/m^3, whereas the greatest improvement was at 62 g BOD_5/m^3.d and 0.25 W/m^3.

The effect of increase in temperature on effluent BOD_5 quality was higher for unfiltered samples than for filtered samples; this was more pronounced at the lower organic loading than at the higher loading. The effect of temperature on the effluent BOD_5 from facultative aerated lagoons was in contrast to that for aerated lagoons studied by Hurwitz (1963), who observed that the effluent BOD_5 concentration and its reductions are reasonably consistent between summer and winter.

6.2.4 Modelling the Combined Effects of Organic Loading and Power Level

The combined effect of organic loading and power level on the performance of facultative aerated lagoons was noticeably significant. Empirical equations relating BOD_5 removal rate coefficients with the combined effect of both power level (P_v) and organic loading (L_v) were derived by regressing the experimental data. Mutiple regression was used and the following equations were obtained:

$$BOD_5 \ K_{20_f} = 2.67 \, P_V^{0.0722} - 1.64 \, e^{0.0013 L_V} \qquad R^2 = 0.94 \qquad (6.13)$$

$$BOD_5 \ K_{20_u} = 0.224 P_V^{0.173} - 0.049 \, L_V^{0.414} \qquad R^2 = 0.93 \qquad (6.14)$$

$$BOD_5 \ K_{30_f} = -0.172 \, P_V^{-0.591} + 2.316 \, L_V^{-0.1041} \qquad R^2 = 0.91 \qquad (6.15)$$

$$BOD_5 \ K_{30_u} = 0.438 \, e^{-8.44 P_V} + 0.282 \, L_V^{0.2277} \qquad R^2 = 0.76 \qquad (6.16)$$

The above equations were significant at a confidence level of 95 % with $P \leq 0.001$

Equations 6.13 and 6.14 can be used to determine the values of 20°C filtered BOD_5 removal rate coefficient and 20°C unfiltered BOD_5 removal rate coefficients, respectively. Equations 6.15 and 6.16 can be used to determine the corresponding values at 30°C. Alternatively, a value of 1.0455 for filtered BOD_5 temperature coefficient and 1.0434 for unfiltered BOD_5 temperature coefficient can be used to determine the 30°C BOD_5 removal rate coefficient by correcting the values found from Equations 6.13 and 6.14, respectively. Variation of these performance parameters with power level and organic loading simultaneously is depicted Figures 12 and 13.

6.3 Chemical Oxygen Demand (COD) Removals

As for BOD_5, Chemical oxygen demand (COD) removal in the facultative aerated lagoon model was intensively studied. The results of the experimental analysis with respect to COD were used in evaluating the performance of the facultative aerated lagoon model. The analyses were interpreted in terms of the removal rate coefficients, percentage removals, temperature correction coefficients and effluent quality. For the first two parameters, Table 6.10 summarises average values which were derived from the several tested samples during each experimental run.

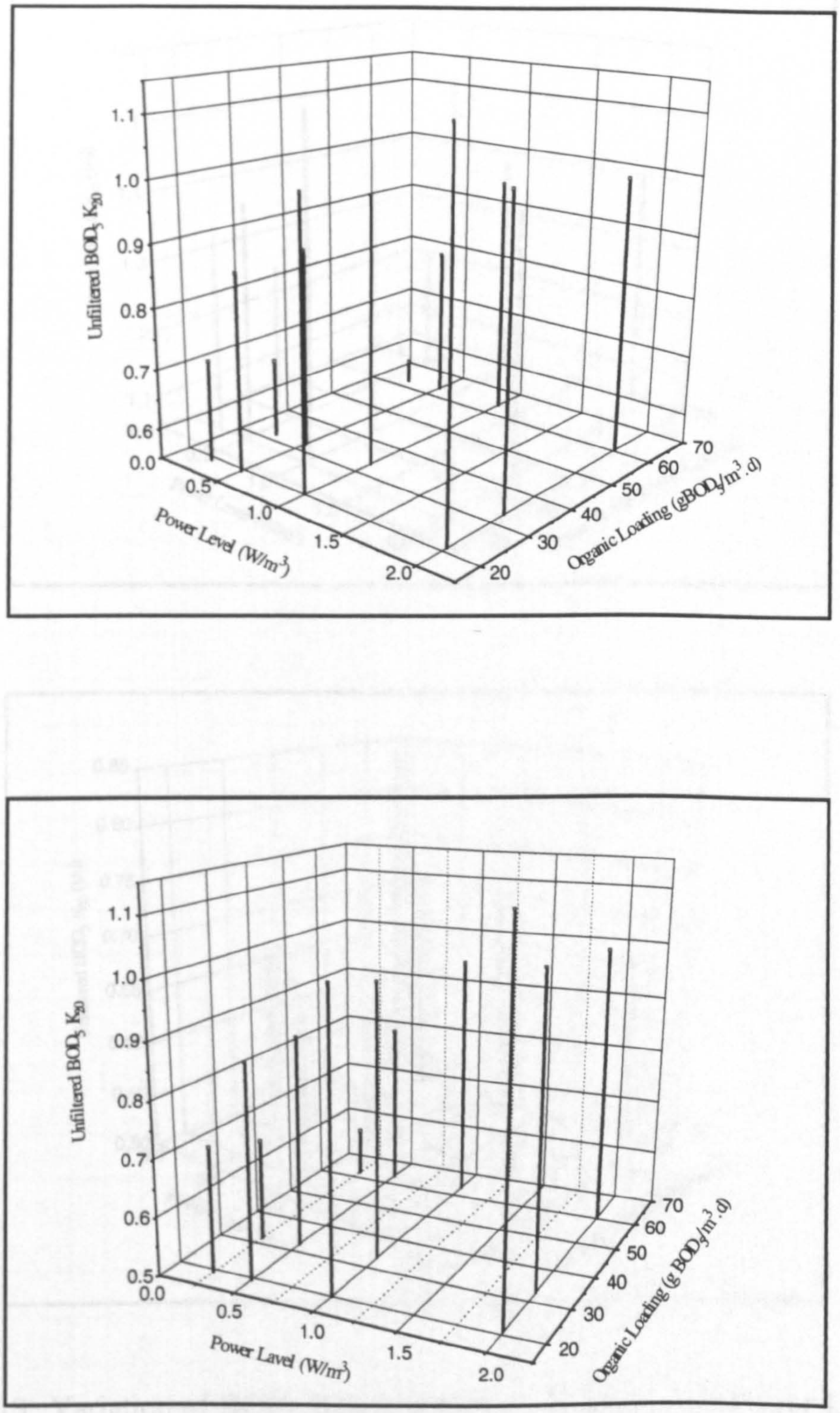


Figure 6.12 Variation of BOD_5 Removal Rate Coefficient with Power Level and Organic Loading at 20°C.

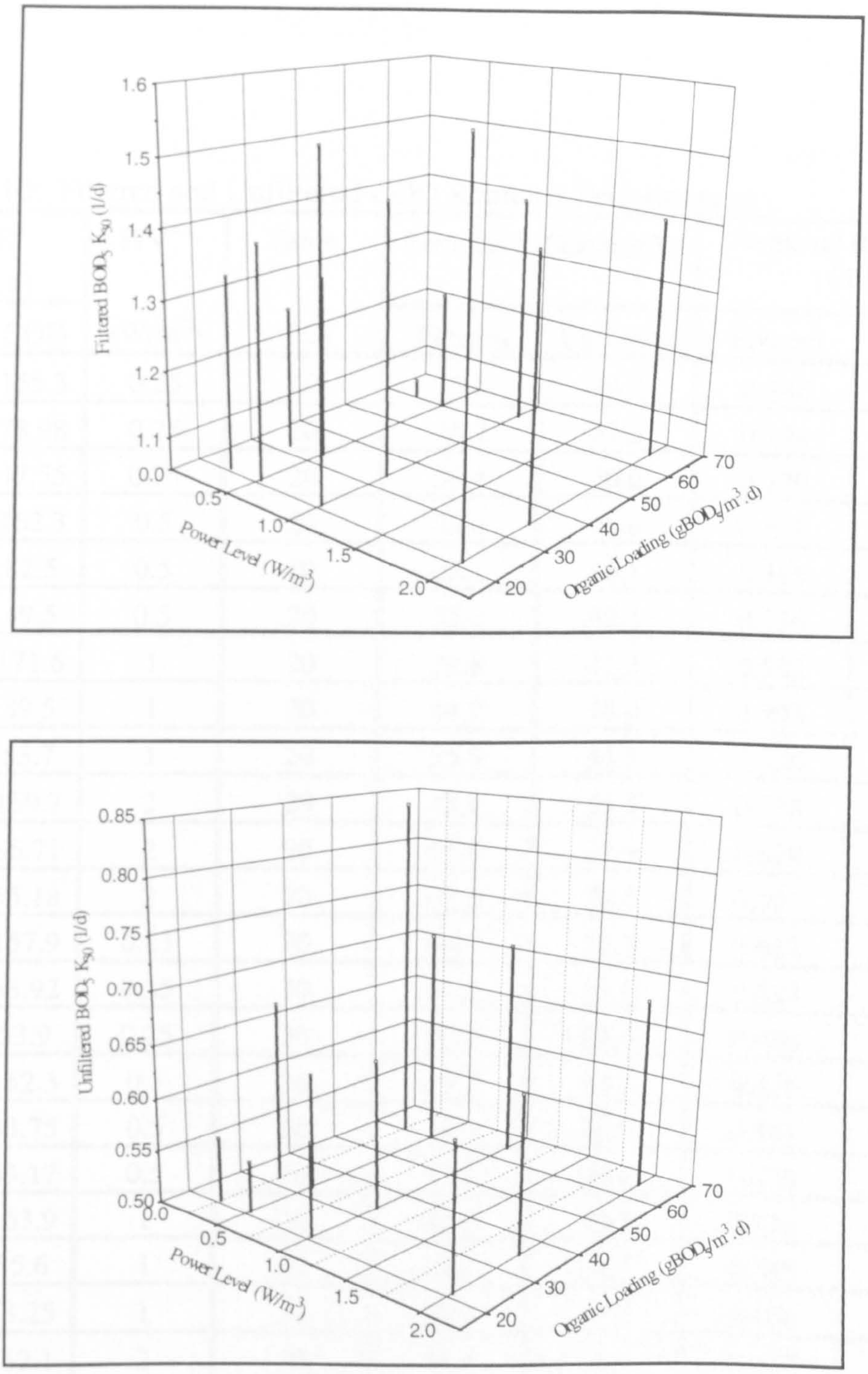


Figure 6.13 Variation of BOD_5 Removal Rate Coefficient with Power Level and Organic Loading at 30°C.

Table 6.10: Filtered and Unfiltered COD Removal Performances

VOLR* (g/m^3.d)		P/V**	Temp.	Removal Efficiency(%)		Removal Rate Coeff (d^{-1})	
BOD_5	COD	(W/m^3)	(°C)	Filtered	Unfiltered	Filtered	Unfiltered
62.67	155.3	0.25	20	15.6	44.2	0.386	0.264
31.86	78.98	0.25	20	38.7	47.2	0.328	0.149
19.18	47.55	0.25	20	50.8	50.0	0.270	0.100
63.61	162.3	0.5	20	29.6	43.6	0.578	0.258
32.33	82.5	0.5	20	45.1	46.1	0.418	0.142
19.40	49.5	0.5	20	55.3	52.3	0.330	0.110
63.90	171.6	1	20	32.8	41.3	0.585	0.235
33.33	89.5	1	20	44.8	48.0	0.393	0.154
20.00	53.7	1	20	55.9	53.3	0.320	0.114
60.53	159.7	2	20	48.1	46.1	0.735	0.214
32.86	86.71	2	20	56.6	52.9	0.529	0.160
20.91	55.18	2	20	62.0	59.1	0.397	0.132
60.71	157.9	0.25	30	49.8	57.3	0.613	0.269
32.28	83.92	0.25	30	61.2	59.6	0.533	0.184
20.73	53.9	0.25	30	66.5	62.3	0.424	0.138
60.23	152.3	0.5	30	49.2	54.6	0.676	0.241
33.13	83.75	0.5	30	59.5	56.0	0.561	0.159
21.03	53.17	0.5	30	63.8	60.4	0.429	0.127
64.55	163.9	1	30	43.2	47.7	0.838	0.304
33.70	85.6	1	30	55.7	54.7	0.585	0.201
20.96	53.25	1	30	60.1	56.2	0.401	0.128
64.79	162.1	2	30	41.6	46.9	0.787	0.294
34.82	87.12	2	30	52.3	53.6	0.520	0.193
21.08	52.74	2	30	62.4	59.1	0.422	0.145

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level

6.3.1 Effects of Organic Loading Rate

Organic loading on the system and the retention time will control the rate of accumulation of new bacterial cells (Barnhart, 1972) and the oxidation of organic materials. The three levels of organic loading (20, 33 and 62 g BOD_5/m^3.d or 52, 85 and 161 g COD/m^3.d) applied in this study were studied for their affects on COD removal performance in the facultative aerated lagoon. To determine the effect of organic loading alone the effects of other factors must be avoided. At a fixed level of aeration power, various levels of organic loading (20, 33 and 62 g BOD_5/m^3.d or 52, 85 and 161 g COD/m^3.d) were applied to determine how the facultative aerated lagoon performance for removing COD responded to such levels of organic loading as a single operating variable. This was repeated for four levels of power input and two temperatures (20° and 30°C).

i) Removal Rate Coefficients

By examining the summarised results in Table 6.10 the effect of organic loading on COD removal rate coefficient can be observed. This effect is shown more clearly in Figures 6.14a and 6.14b., which indicate a positive effect of organic loading on the COD removal rate coefficient.

Table 6.10 and Figure 6.14a show the variations of both 20°C filtered (K_{20_f}) and 20°C unfiltered (K_{20_u}) COD removal rate coefficients with organic loading. It can be inferred that COD K_{20_f} had a wider range than COD K_{20_u}. The figure reveals that COD K_{20_f} had a smaller range at lower organic loadings than at higher loadings. Organic loading affected COD K_{20_f} positively and this effect was higher at higher power levels than at lower levels of aeration power. With no apparent explanation, organic loading had an opposite effect on COD K_{20_f} to that on BOD_5 K_{20_f}; the former was affected positively by organic loading whereas the

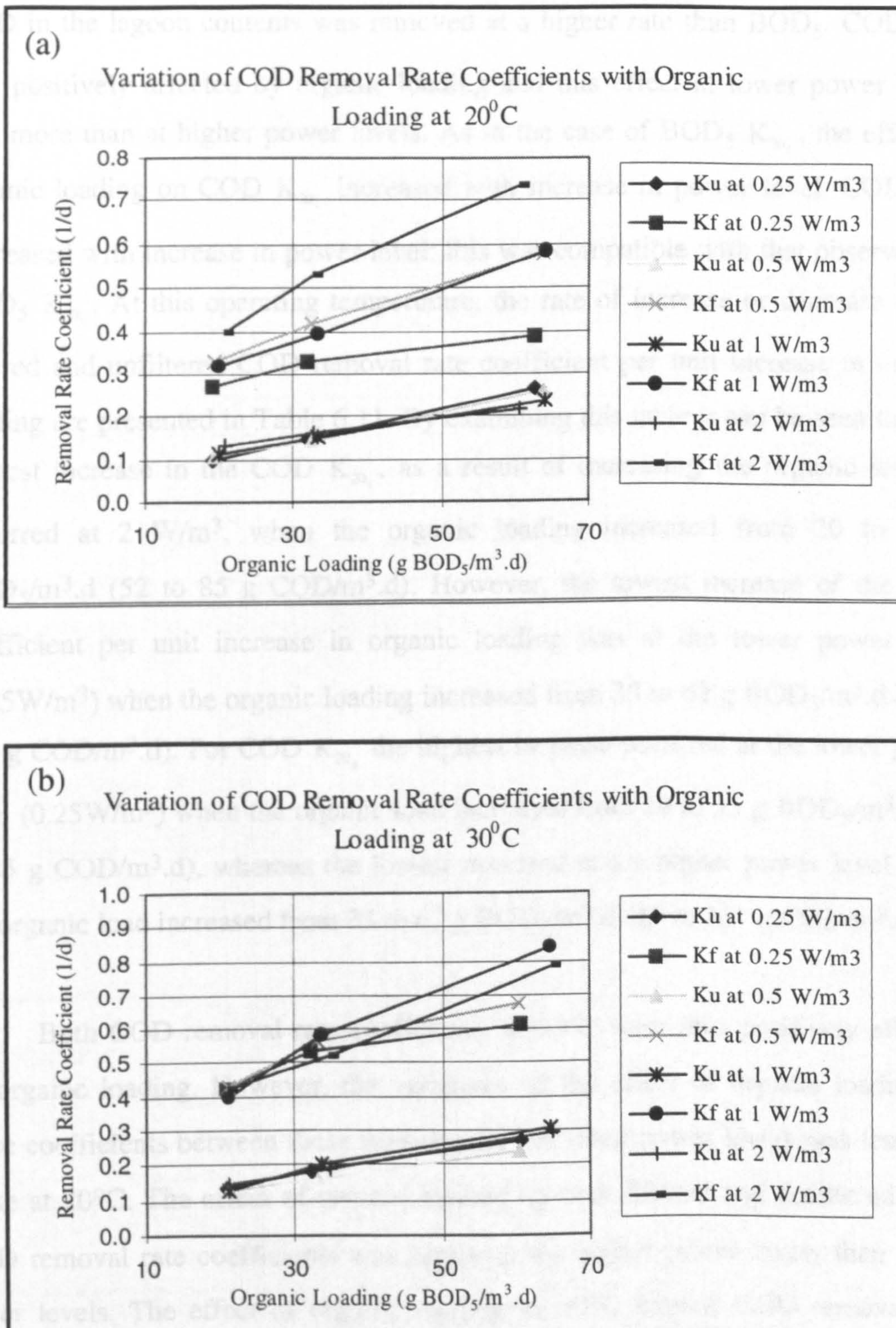


Figure 6.14 Variation of COD Removal Rate Coefficient with Organic Loading.

latter was inversely affected by organic loading. The reason could be that soluble COD in the lagoon contents was removed at a higher rate than BOD_5. COD K_{20_u} was positively affected by organic loading and this effect at lower power levels was more than at higher power levels. As in the case of BOD_5 K_{20_f}, the effect of organic loading on COD K_{20_f} increased with increase in power level. COD K_{20_u} decreased with increase in power level; this was compatible with that observed for BOD_5 K_{20_u}. At this operating temperature, the rate of increase or decrease in the filtered and unfiltered COD removal rate coefficient per unit increase in organic loading are presented in Table 6.11. By examining this table it can be seen that the highest increase in the COD K_{20_f}, as a result of increasing the organic loading, occurred at 2 W/m^3, when the organic loading increased from 20 to 33 g BOD_5/m^3.d (52 to 85 g COD/m^3.d). However, the lowest increase of the same coefficient per unit increase in organic loading was at the lower power level (0.25W/m^3) when the organic loading increased from 33 to 62 g BOD_5/m^3.d (85 to 161 g COD/m^3.d). For COD K_{20_u} the highest increase occurred at the lower power level (0.25W/m^3) when the organic load increased from 20 to 33 g BOD_5/m^3.d (52 to 85 g COD/m^3.d), whereas the lowest occurred at the higher power level when the organic load increased from 33 to 62 g BOD_5/m^3.d (85 to 161 g COD/m^3.d).

Both COD removal rate coefficients at 30°C were also positively affected by organic loading. However, the variations of the effect of organic loading on these coefficients between those runs operated at fixed power levels was less than those at 20°C. The effect of organic loading on both filtered and unfiltered 30°C COD removal rate coefficients was higher at the higher power levels than at the lower levels. The effect of organic loading on 30°C filtered COD removal rate coefficient (K_{30_f}) was the opposite to that on BOD_5 K_{30_f}. The rate of increase or decrease in both filtered and unfiltered 30°C COD removal rate coefficients per unit increase in BOD_5 organic loading are presented in Table 6.12. It is clear from the table that COD K_{30_u} was affected by organic loading less than COD K_{30_f}.

Table 6.11: Rate of Increase or Decrease in the COD 20°C Filtered and Unfiltered Removal Rate Coefficients per Unit Increase in Organic Loading.

Parameter type	Power level (W/m^3)	dK/dLV ($d^{-1}/g\ BOD_5/m^3.d$) VOLR* 20→33 g BOD_5/m^3.d 52→85 g COD/m^3.d	VOLR* 33→62 g BOD_5/m^3.d 85→161 g COD/m^3.d
Filtered	0.25	0.004588	0.001875
	0.5	0.006804	0.005110
	1.0	0.005488	0.006278
	2.0	0.011041	0.007446
Unfiltered	0.25	0.003868	0.003735
	0.5	0.002518	0.003701
	1.0	0.003013	0.002643
	2.0	0.002405	0.001941

* VOLR = Volumetric Organic Loading
NB Negative values means decreasing rate and positive values means increasing rate

Table 6.12: Rate of Increase or Decrease in the COD 30°C Filtered and Unfiltered Removal Rate Coefficients per Unit Increase in Organic Loading.

Parameter type	Power level (W/m^3)	dK/dLV ($d^{-1}/g\ BOD_5/m^3.d$) VOLR* 20→33 g BOD_5/m^3.d 52→85 g COD/m^3.d	VOLR* 33→62 g BOD_5/m^3.d 85→161 g COD/m^3.d
Filtered	0.25	0.009456	0.002840
	0.5	0.010962	0.004218
	1.0	0.014429	0.0082
	2.0	0.007143	0.008916
Unfiltered	0.25	0.004033	0.002965
	0.5	0.002608	0.003021
	1.0	0.005738	0.003324
	2.0	0.003513	0.003385

* VOLR = Volumetric Organic Loading
NB Negative values means decreasing rate and positive values means increasing rate

Comparison of Table 6.11 and Table 6.12 shows that there is no fixed rule for the difference between the effect of organic loading at 20°C and 30°C on COD removal rate coefficients. From the above it can be said that there was a positive effect of organic loading on COD removal rate coefficients. The effect of organic loading was higher at low loadings than at high loadings, whereas with the increase in the power level the effect of organic loading did not vary with such a clear trend.

According to the experimental data it was possible to find models to describe the relationship between organic loading (g $BOD_5/m^3.d$) and COD removal rate coefficients (d^{-1}) for both operational temperatures (20° and 30°C). The following empirical equations, which are significant at a confidence level of 95 % with $P < 0.001$, can be used to predict COD removal rate coefficients from organic loading on the system:

$$\text{COD } K_{20_f} = 0.0817\, L_V^{0.469} \qquad R^2 = 0.964 \qquad (6.17)$$

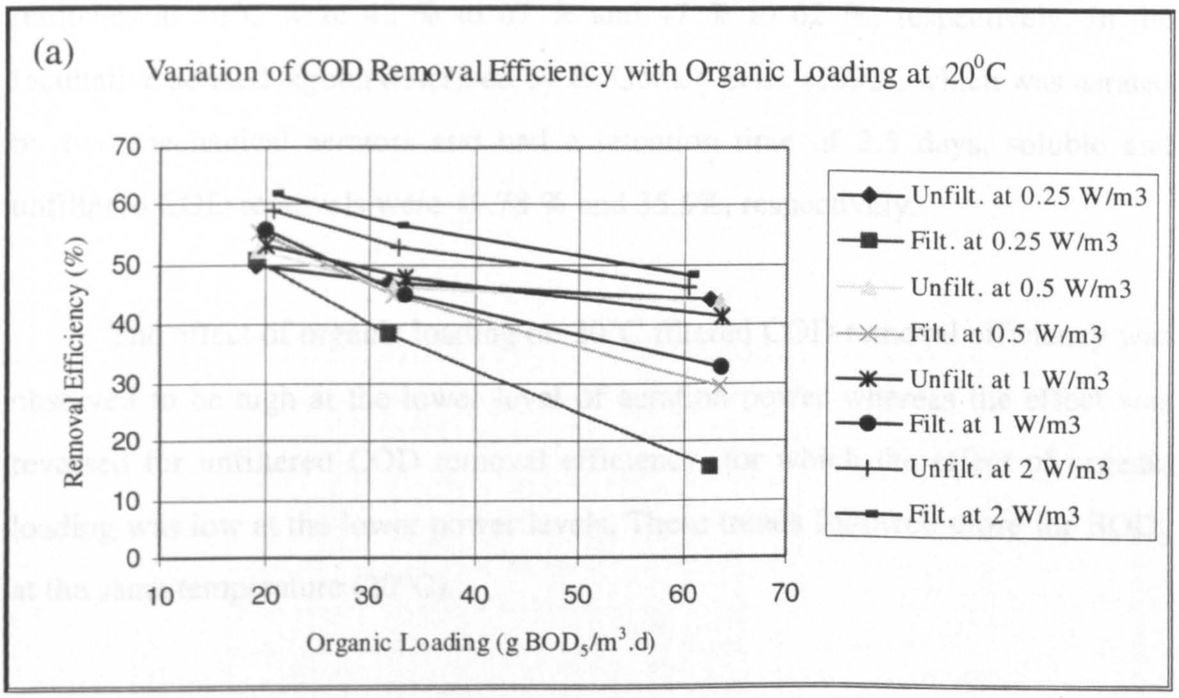
$$\text{COD } K_{20_e} = 0.0151\, L_V^{0.669} \qquad R^2 = 0.995 \qquad (6.18)$$

$$\text{COD } K_{30_f} = 0.0926\, \sqrt{L_V} \qquad R^2 = 0.993 \qquad (6.19)$$

$$\text{COD } K_{30_e} = 0.018\, L_V^{0.6618} \qquad R^2 = 0.995 \qquad (6.20)$$

ii) Percentage Removals

The two forms of COD removal percentage (filtered and unfiltered) were determined in a similar way as that for BOD_5. Table 6.10 and Figures. 6.15a and 6.15b indicate that there is an inverse effect of organic loading on such pollution constituent removal at temperatures of 20°C and 30°C. At the lower temperature (20°C) the model facultative aerated lagoon achieved 16 % to 62 % removal of the soluble COD and 41 % to 59 % removal of the unfiltered COD over the various



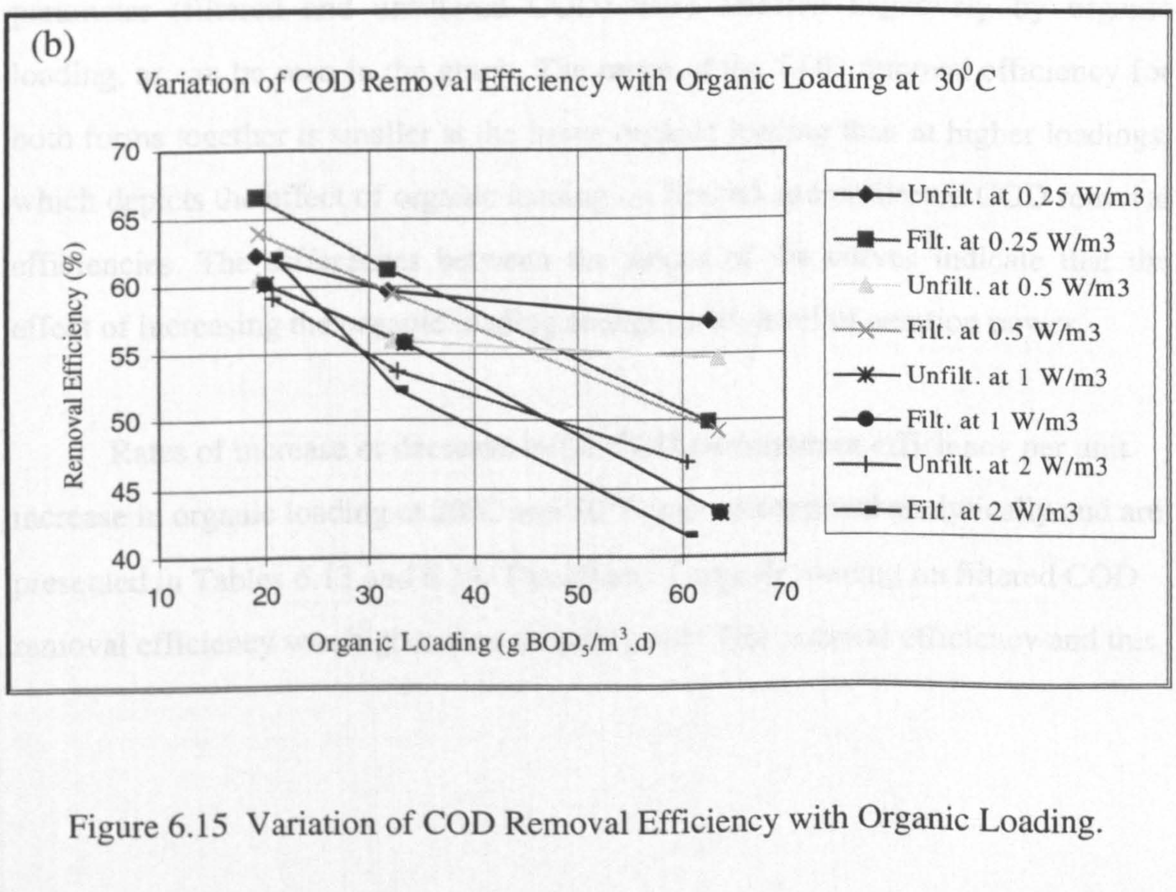


Figure 6.15 Variation of COD Removal Efficiency with Organic Loading.

ranges of applied power levels and organic loadings. Corresponding COD removals at 30°C were 42 % to 67 % and 47 % to 62 %, respectively. In the facultative aerated lagoon described by El-Gohary *et al.* (1993), which was aerated by two mechanical aerators and had a retention time of 2.5 days, soluble and unfiltered COD removals were 49.78 % and 35.5%, respectively.

The effect of organic loading on 20°C filtered COD removal efficiency was observed to be high at the lower level of aeration power whereas the effect was reversed for unfiltered COD removal efficiency, for which the effect of organic loading was low at the lower power levels. These trends followed those for BOD_5 at the same temperature (20°C).

The effect of organic loading on COD removal efficiencies when mixed liquor temperature was 30°C is shown graphically in Fig. 6.15b. Both forms of this parameter (filtered and unfiltered COD) were affected negatively by organic loading, as can be seen in the graph. The range of the COD removal efficiency for both forms together is smaller at the lower organic loading than at higher loadings, which depicts the effect of organic loading on filtered and unfiltered COD removal efficiencies. The differences between the slopes of the curves indicate that the effect of increasing the organic loading changes with level of aeration power.

Rates of increase or decrease in the COD performance efficiency per unit increase in organic loading at 20°C and 30°C were determined analytically and are presented in Tables 6.13 and 6.14. The effect of organic loading on filtered COD removal efficiency was higher than on unfiltered COD removal efficiency and this

Table 6.13: Rate of Decrease or Increase in Filtered and Unfiltered COD Percentage Removal per Unit Increase in Organic Loading Rate at 20°C·

Parameter type	Power level (W/m^3)	d%/dLV (% /g BOD_5 /m^3.d) VOLR* 20→33 g BOD_5/m^3.d 52→85 g COD/m^3.d	d%/dLV (% /g BOD_5 /m^3.d) VOLR* 33→62 g BOD_5/m^3.d 85→161 g COD/m^3.d
Filtered	0.25	- 0.95494	- 0.74822
	0.5	- 0.78222	- 0.49768
	1.0	- 0.82758	- 0.39486
	2.0	- 0.45088	- 0.30421
Unfiltered	0.25	- 0.21999	- 0.09753
	0.5	- 0.48422	- 0.07751
	1.0	- 0.39106	- 0.21933
	2.0	- 0.52395	- 0.24411

* VOLR = Volumetric Organic Loading

NB Negative values means decreasing rate and positive values means increasing rate

Table 6.14: Rate of Decrease or Increase in Filtered and Unfiltered COD Percentage Removal per Unit Increase in Organic Loading Rate at 30°C·

Parameter type	Power level (W/m^3)	d%/dLV (%/g BOD_5 /m^3.d) VOLR* 20→33 g BOD_5/m^3.d 52→85 g COD/m^3.d	d%/dLV (%/g BOD_5 /m^3.d) VOLR* 33→62 g BOD_5/m^3.d 85→161 g COD/m^3.d
Filtered	0.25	- 0.45300	- 0.40036
	0.5	- 0.35713	- 0.38000
	1.0	- 0.34762	- 0.40677
	2.0	- 0.73045	- 0.35871
Unfiltered	0.25	- 0.23512	- 0.07956
	0.5	- 0.37025	- 0.04956
	1.0	- 0.11608	- 0.22773
	2.0	- 0.39845	- 0.22524

* VOLR = Volumetric Organic Loading

NB Negative values means decreasing rate and positive values means increasing rate

is more clear at the lower power levels for both temperatures (20° and 30°C). The increase in temperature reduced the effect of organic loading on both coefficients. This could be due to the higher removals at higher temperatures.

The relationships between COD removal efficiencies, both filtered (FPR) and unfiltered (UPR), and BOD_5 volumetric organic loading were determined in the form of empirical equations. From the experimental data the following models were obtained:

$$\text{COD } FPR_{20} = 318.88\, L_v^{-0.5687} \qquad R^2 = 0.967 \qquad (6.21)$$

$$\text{COD } UPR_{20} = 89.013\, L_v^{-0.1715} \qquad R^2 = 0.997 \qquad (6.22)$$

$$\text{COD } FPR_{30_f} = 161.772\, L_v^{-0.3033} \qquad R^2 = 0.997 \qquad (6.23)$$

$$\text{COD } UPR_{30_u} = 91.161\, L_v^{-0.1388} \qquad R^2 = 0.997 \qquad (6.24)$$

At a confidence level of 95% the above equations were significant with $P < 0.001$.

iii) Temperature Coefficient

Temperature coefficients used for correcting COD removal rate coefficients for a biological treatment process operated not at standard temperature were determined using Equation 6.8. COD removal rate coefficients for runs operated at both temperatures (20°C and 30°C) were used in calculating this temperature coefficient. Filtered (θ_f) and unfiltered (θ_u) temperature coefficients were computed. The effect of organic loading on both forms of temperature coefficient are shown graphically in Figure 6.16. At fixed power levels, the variation of temperature coefficient with organic loading are clearly illustrated in the figure. No common trend in the values of such coefficient were observed when organic loading increased from 20 to 33 g $BOD_5/m^3.d$ (from 52 to 85 g $COD/m^3.d$) and

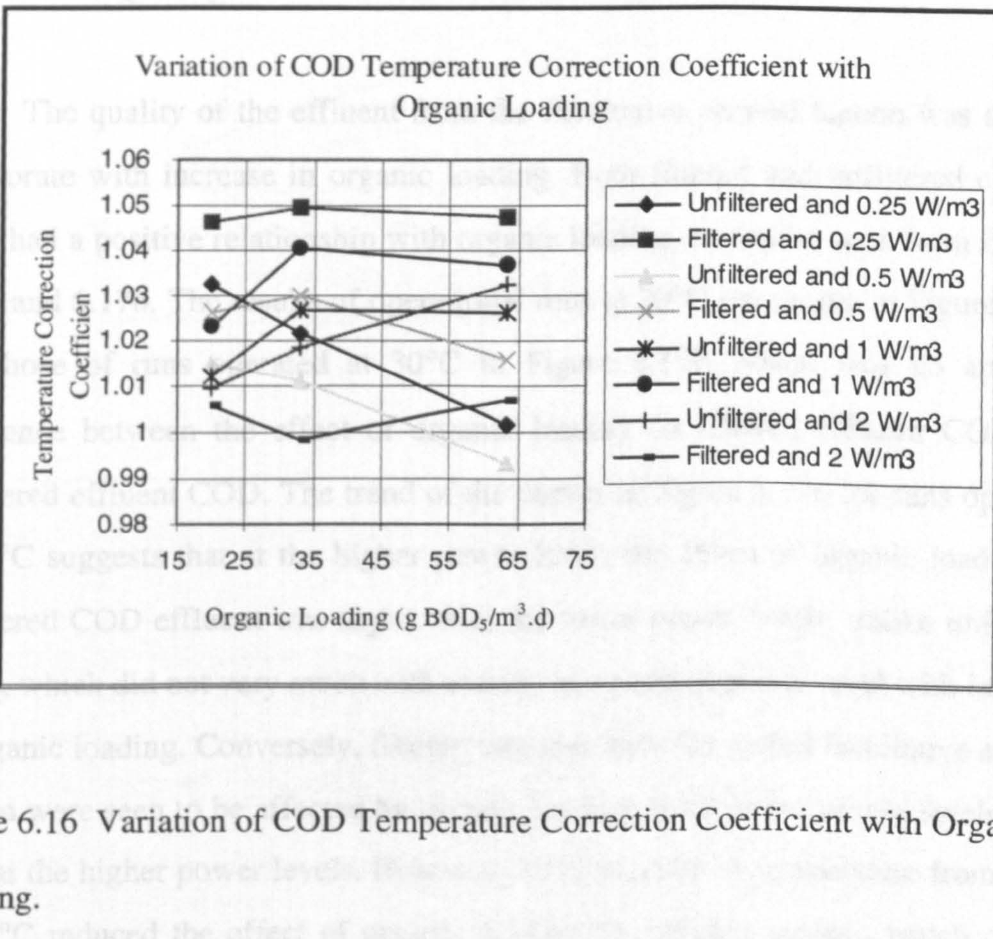


Figure 6.16 Variation of COD Temperature Correction Coefficient with Organic Loading.

from 33 to 62 g $BOD_5/m^3.d$ (from 85 to 161 g $COD/m^3.d$). It can be inferred from the figure that COD θ_u was affected by organic loading more than θ_f, particularly at the lower power levels. At the lower power levels, both temperature coefficients were affected by organic loading more than at the higher power levels.

iv) Effluent Quality

The quality of the effluent from the facultative aerated lagoon was seen to deteriorate with increase in organic loading. Both filtered and unfiltered effluent COD had a positive relationship with organic loading, as can be seen from Figures 6.17a and 6.17b. The results of operational runs at 20°C are shown in Figure 6.17a and those of runs operated at 30°C in Figure 6.17b. There was no apparent difference between the effect of organic loading on filtered effluent COD and unfiltered effluent COD. The trend of the curves in Figure 6.17a for runs operated at 20°C suggests that at the higher power levels the effect of organic loading on unfiltered COD effluent was higher than the lower power levels, unlike unfiltered BOD_5 which did not vary much with change in operating power level with increase in organic loading. Conversely, filtered samples from the model facultative aerated lagoon were seen to be affected by organic loading at the lower power levels more than at the higher power levels. However, 10°C increase in temperature from 20°C to 30°C reduced the effect of organic loading on effluent quality; which can be observed by comparing the curves in these two figures. At this temperature (30°C), a unit increase in organic loading did not increase the effluent COD as much as in the case of experimental runs operated at 20°C.

It can be summarised from the above that the effluent COD from facultative aerated lagoons was affected positively by organic loading. This effect varied with the power level at which the run was operated and with temperature of the mixed

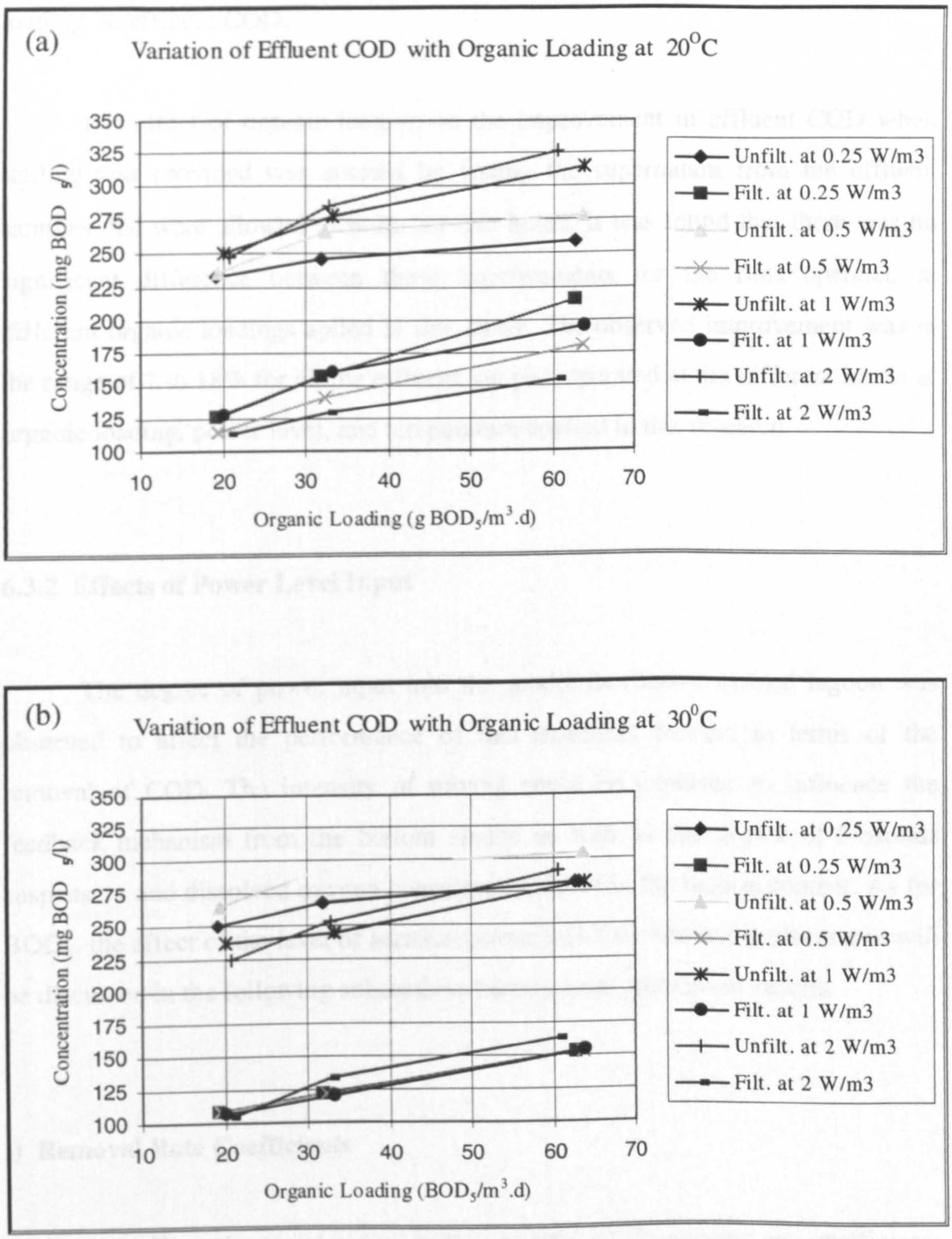


Figure 6.17 Variation of Effluent COD with Organic Loading.

liquor. Increasing the temperature resulted in a reduction of the effect of organic loading on effluent COD.

The effect of organic loading on the improvement in effluent COD when settling was provided was studied by testing the supernatant from the effluent samples that were allowed to settle for two hours. It was found that there was no significant difference between these improvements for the runs operated at different organic loadings aplied in this study. The observed improvement was in the range of 7 to 18% for all the effluent samples operated at the different levels of organic loading, power level, and temperature applied in this research.

6.3.2 Effects of Power Level Input

The degree of power input into the model facultative aerated lagoon was observed to affect the performance of this treatment process in terms of the removal of COD. The intensity of mixing could be expected to influence the feedback mehanism from the bottom sludge as well as the degree of biosolids suspension and dissolved oxygen concentration level in the lagoon content. As for BOD_5, the effect of the level of aeration power on COD removal performance will be discussed in the following subsections based on the operational results.

i) Removal Rate Coefficients

The effect of power level on 20°C and 30°C COD removal rate coefficients (filtered and unfiltered) is shown in Table 6.10. Relationships between power level and removal rate coefficients are presented graphically in Figures 6.18a and 6.18b. At 20°C operating temperature, as in the case of BOD_5 K_{20_f}, Figure 6.18a reveals

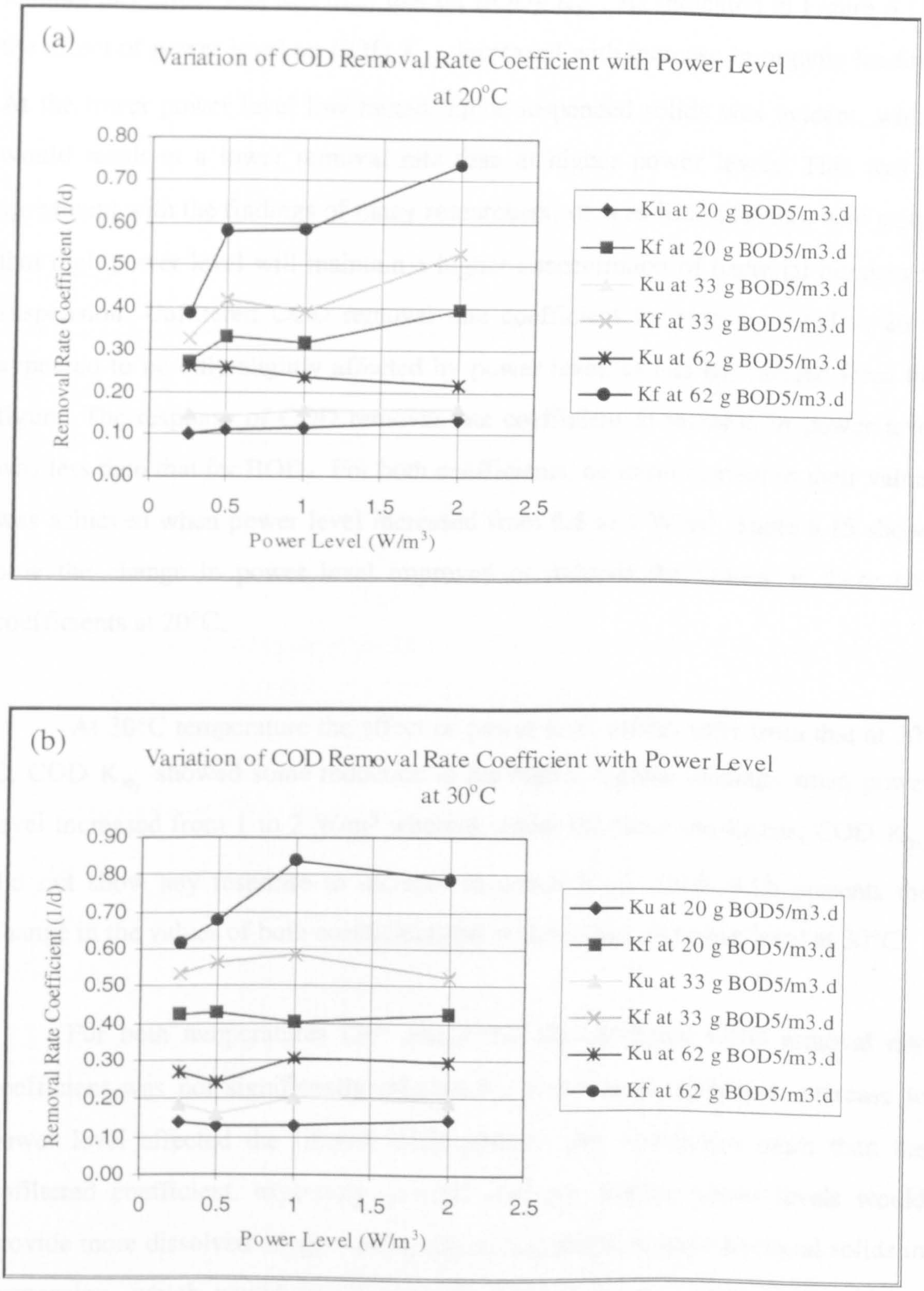


Figure 6.18 Variation of COD Removal Rate Coefficient with Power Level.

that filtered COD was affected positively by power level. At the lower organic loading this effect was less than that on BOD_5 K_{20_f}. As indicated in Figure 6.18a the effect of power level on COD K_{20_f} increased with increase in organic loading. At the lower power level low mixed liquor suspended solids was evident, which would result in a lower removal rate than at higher power levels. This was in agreement with the findings of many researchers, such as Rich (1983a), who stated that high power level will maintain a higher concentration of bacterial biomass in suspension. Unfiltered COD removal rate coefficient for runs operated at 20°C appeared to be only slightly affected by power level, as can be inferred from the figure. The response of COD removal rate coefficient to increase in power level was less than that for BOD_5. For both coefficients, no improvement in their values was achieved when power level increased from 0.5 to 1 W/m^3. Table 6.15 shows how the change in power level improved or reduced the values of these two coefficients at 20°C.

At 30°C temperature the effect of power level differs little from that at 20° C. COD K_{30_f} showed some reduction at the higher organic loadings when power level increased from 1 to 2 W/m^3 whereas, under the same conditions, COD K_{30_u} did not show any response to increase in power level. Table 6.16 presents the change in the values of both coefficients per unit increase in power level at 30°C.

For both temperatures (20° and 30°C) the unfiltered COD removal rate coefficient was not significantly affected by power level. However, increase in power level affected the filtered COD removal rate coefficient more than the unfiltered coefficient, especially at high loadings. Higher power levels would provide more dissolved oxygen to the lagoon and maintain more bacterial solids in suspension, which would facilitate better filtered organic removal. The higher suspended solids concentrations resulting from higher power levels (1 to 2 W/m^3) were evident; this may be the reason for the small improvement in unfiltered

Table 6.15: Rate of Decrease or Increase in Filtered and Unfiltered COD 20°C Removal Rate Coefficients per Unit Increase in Power Level.

Parameter type	VOLR*		dK/d(P/V) $d^{-1}/(W/m^3)$		
	g BOD_5/m^3.d	g COD/m^3.d	P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	52	0.242374	- 0.02180	0.077894
	33	85	0.361656	- 0.05146	0.136638
	62	161	0.769865	0.012693	0.150774
Unfiltered	20	52	0.038983	0.008396	0.017654
	33	85	- 0.02691	0.023599	0.006217
	62	161	- 0.02415	- 0.04628	- 0.02085

* VOLR = Volumetric Organic Loading Rate

** P/V = Power Level (W/m^3)

NB Negative values means decreasing rate and positive values means increasing rate

Table 6.16: Rate of Decrease or Increase in Filtered and Unfiltered COD 30°C Removal Rate Coefficients per Unit Increase in Power Level.

Parameter type	VOLR*		dK/d(P/V) $d^{-1}/(W/m^3)$		
	g BOD_5/m^3.d	g COD/m^3.d	P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	52	0.021406	- 0.05594	0.020978
	33	85	0.114949	0.046493	- 0.06467
	62	161	0.249280	0.323690	- 0.05034
Unfiltered	20	52	- 0.04123	0.001823	0.016372
	33	85	- 0.10134	0.084924	- 0.00844
	62	161	- 0.11104	0.126194	- 0.00949

* VOLR = Volumetric Organic Loading Rate

** P/V = Power Level (W/m^3)

NB Negative values means decreasing rate and positive values means increasing rate

removal rate coefficient. Very low traces of algae, determined in terms of chlorophyll "a", were observed at the lower power level, whereas no algae were observed at higher levels of energy input. Hence, resuspension of solids at higher power levels would seem to be the main cause of low rates of improvement in unfiltered removal coefficients.

Empirical equations derived from the experimental data which show the relationships between power level (P_V) and COD removal rate coefficients (K) were determined. These empirical equations which are significant at a confidence level of 95 % with $P < 0.001$ are:

$$\text{COD } K_{20_f} = 0.4669\, P_V^{0.2138} \qquad R^2 = 0.943 \qquad (6.25)$$

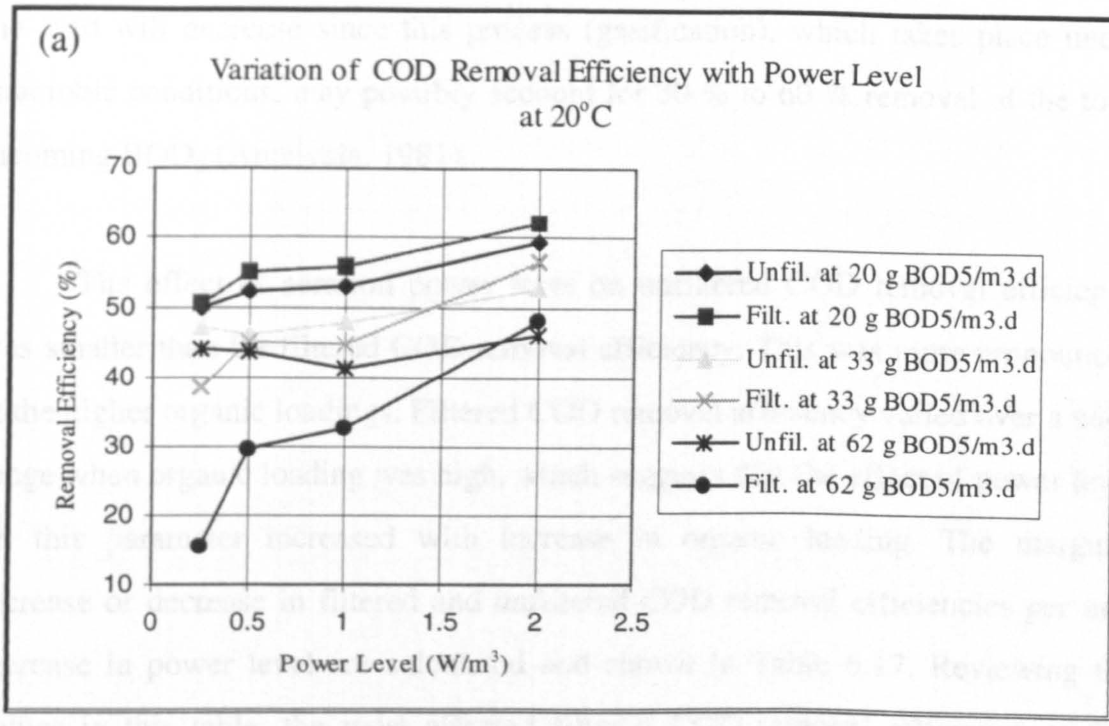
$$\text{COD } K_{20_u} = 0.171\, P_V^{0.021} \qquad R^2 = 0.90 \qquad (6.26)$$

$$\text{COD } K_{30_f} = 0.5737\, P_V^{0.041} \qquad R^2 = 0.946 \qquad (6.27)$$

$$\text{COD } K_{30_u} = 0.202\, P_V^{0.05} \qquad R^2 = 0.914 \qquad (6.28)$$

ii) Percentage Removals

COD removal efficiencies at 20°C and 30°C operating temperatures were determined and relationships with power level are summarised in Table 6.10 and presented graphically in Figures 6.19a and 6.19b. At 20°C filtered and unfiltered COD removal efficiencies were affected positively by power level. The increase in the benthal feedback during the lower power levels would cause the removal of COD to be smaller than if the power level was higher. Bryant (1993) estimated from his laboratory results that 0.43 mg COD is the soluble feedback per mg TSS settled. Sackellares *et al.* (1987) assert that the dissolved substrate and nutrient feedback rates from the benthal zone are sensitive to the dissolved oxygen concentration in the overlying water, temperature, velocity gradients in the



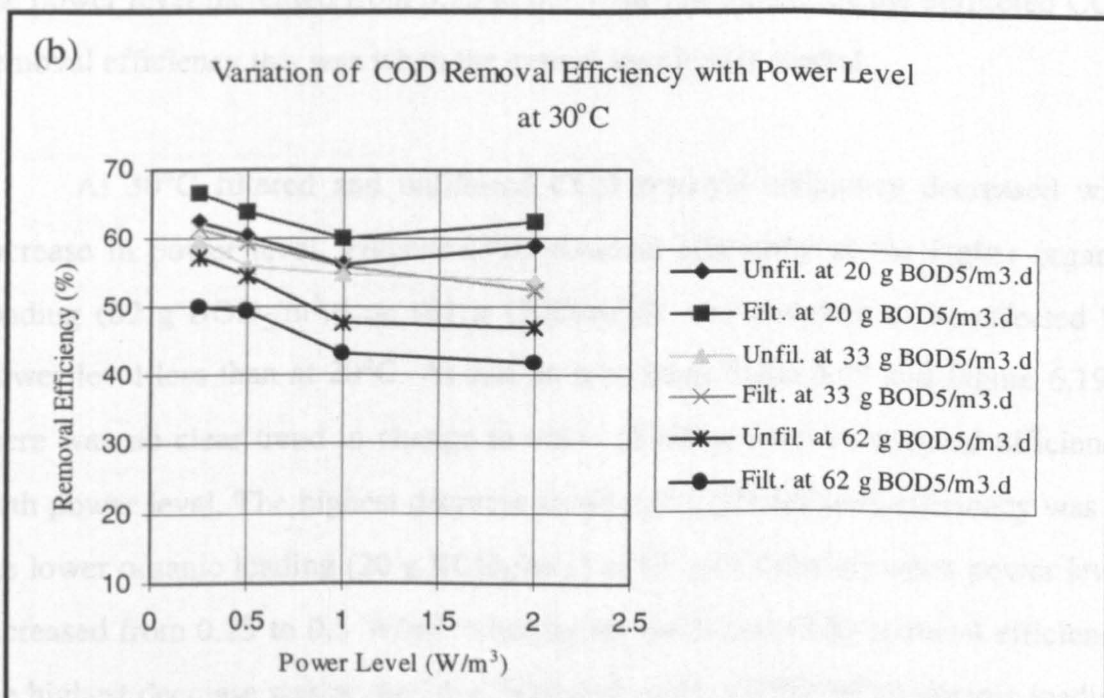


Figure 6.19 Variation of COD Removal Efficiency with Power Level.

overlying water and solids loading rate. If gasification occurred after liquefaction the load will decrease since this process (gasification), which takes place under anaerobic conditions, may possibly account for 30 % to 60 % removal of the total incoming BOD_5 (Arceivala, 1981).

The effect of aeration power level on unfiltered COD removal efficiency was smaller than for filtered COD removal efficiency. This was more pronounced at the higher organic loadings. Filtered COD removal efficiency varied over a wide range when organic loading was high, which suggests that the effect of power level on this parameter increased with increase in organic loading. The marginal increase or decrease in filtered and unfiltered COD removal efficiencies per unit increase in power level are calculated and shown in Table 6.17. Reviewing the values in this table, the most affected filtered COD removal efficiency at this operating temperature (20°C) was when the system was highly loaded and when the power level increased from 0.25 to 0.5 W/m^3; however, for the unfiltered COD removal efficiency this was when the system was lightly loaded.

At 30°C filtered and unfiltered COD removal efficiency decreased with increase in power level. Filtered COD removal efficiency at the higher organic loading (62 g BOD_5/m^3.d or 161 g COD/m^3.d) was observed to be affected by power level less than at 20°C. As can be seen from Table 6.18 and Figure 6.19b, there was no clear trend in change in value of either form of removal efficiency with power level. The highest decrease in filtered COD removal efficiency was at the lower organic loading (20 g BOD_5/m^3.d or 52 g COD/m^3.d) when power level increased from 0.25 to 0.5 W/m^3, whereas for unfiltered COD removal efficiency the highest decrease was at the 33 g BOD_5/m^3.d (85 g COD/m^3.d) organic loading when power level increased from 0.25 to 0.5 W/m^3.

Table 6.17: Rate of Decrease or Increase in COD Filtered and Unfiltered Removal Efficiencies per Unit Increase in the Power Level at 20° C.

Parameter type	VOLR*		d%/d(P/V) %/(W/m^3)		
	g BOD_5/m^3.d	g COD/m^3.d	P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	52	17.88667	1.218301	6.090792
	33	85	25.85724	- 0.61720	11.73807
	62	161	55.78793	6.373272	15.38952
Unfiltered	20	52	9.292929	1.871226	5.884482
	33	85	- 4.59877	3.968173	4.838338
	62	161	- 2.27857	- 4.59116	4.787718

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level (W/m^3)
NB Negative values means decreasing rate and positive values means increasing rate

Table 6.18: Rate of Decrease or Increase in COD Filtered and Unfiltered Removal Efficiencies per Unit Increase in the Power Level at 30° C.

Parameter type	VOLR*		d%/d(P/V) %/(W/m^3)		
	g BOD_5/m^3.d	g COD/m^3.d	P/V** 0.25→0.5	P/V** 0.5→1.0	P/V** 1.0→2.0
Filtered	20	52	- 10.6966	- 7.27954	2.217989
	33	85	- 7.04932	- 7.49776	- 3.38980
	62	161	- 2.70687	- 11.9920	- 1.59637
Unfiltered	20	52	-7.37939	- 8.51104	2.931850
	33	85	- 14.4301	- 2.51331	- 1.06385
	62	161	- 10.7534	- 13.8748	- 0.79165

* VOLR = Volumetric Organic Loading Rate
** P/V = Power Level (W/m^3)
NB Negative values means decreasing rate and positive values means increasing rate

Filtered COD removal efficiency and unfiltered COD removal efficiency in the model facultative aerated lagoon were affected positively by power level when operating temperature was 20°C, whereas at 30°C temperature both were noticed to be affected negatively by power level. The trend in the effect of power level on both COD removal efficiencies was almost consistent with that for BOD_5. At high loadings the increase in temperature had reduced the extent of the effect of power level on filtered COD removal efficiency

Modelling the relationship between COD removal efficiencies (FPR and UPR) and power level was achieved. Empirical equations derived from the experimental data were determined as follows:

$$\text{COD FPR}_{20} = 47.428\ P_v^{0.251} \qquad R^2 = 0.946 \qquad (6.29)$$

$$\text{COD UPR}_{20} = 49.459\ P_v^{0.047} \qquad R^2 = 0.992 \qquad (6.30)$$

$$\text{COD FPR}_{30} = 54.02\ P_v^{-0.0702} \qquad R^2 = 0.983 \qquad (6.31)$$

$$\text{COD UPR}_{30} = 54.44\ P_v^{-0.0627} \qquad R^2 = 0.996 \qquad (6.32)$$

These equations are significant at a confidence level of 95 % with $P < 0.001$.

iii) Temperature Coefficients

Filtered and unfiltered COD temperature correction coefficients' variations with power level are shown in Figure 6.20, from which it can be seen that there was no clear trend in the effect of power level on those coefficients unlike that for BOD_5. The highest fluctuations in these coefficients at a fixed organic loading occurred in the highly loaded systems. This means that at high organic loading (62 g BOD_5/m^3.d or 161 g COD/m^3.d) increase in power level had made the system performance in removing COD more susceptible to temperature changes.

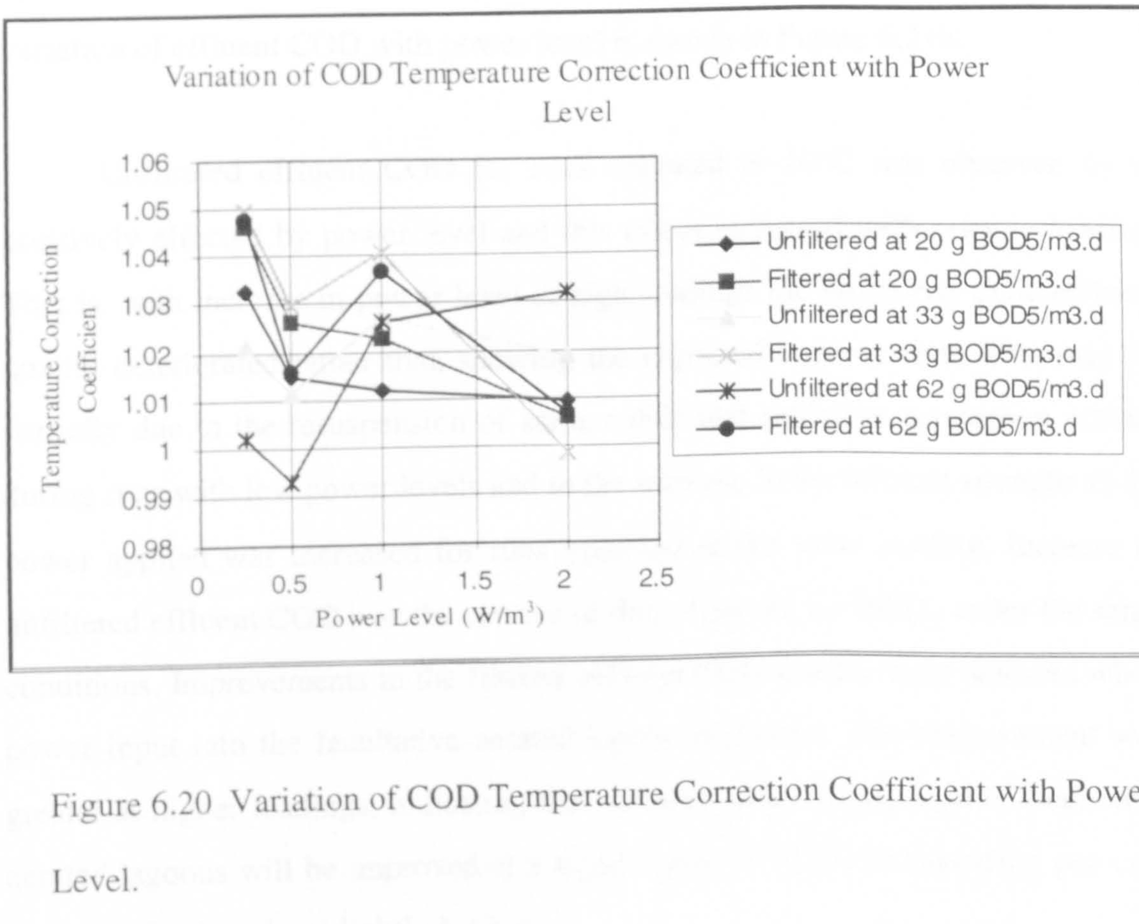


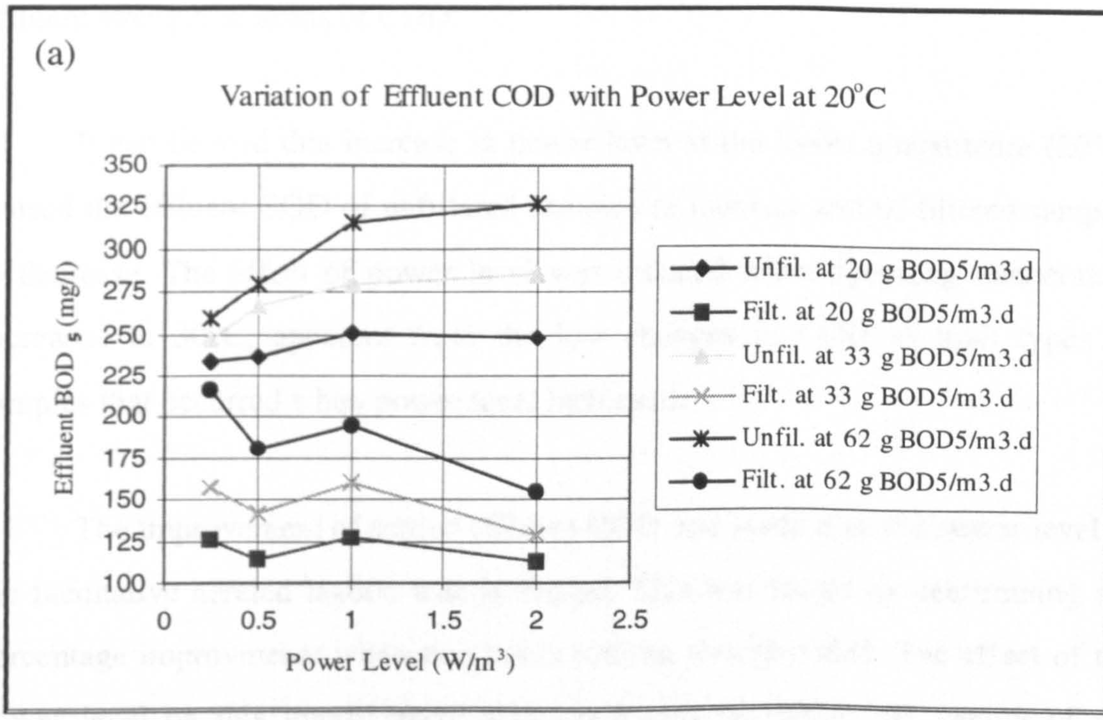
Figure 6.20 Variation of COD Temperature Correction Coefficient with Power Level.

iv) Effluent Quality

Both filtered and unfiltered COD of the effluent samples collected from the facultative aerated lagoon model varied with level of aeration power introduced into the system. COD effluent quality for the model facultative aerated lagoon operated at 20°C is shown in Figure 6.21a; for the system operated at 30°C the variation of effluent COD with power level is shown in Figure 6.21b.

Unfiltered effluent COD for runs operated at 20°C was observed to be positively affected by power level and this effect increased with organic loading. That is, with increase in power level at high loadings the unfiltered COD effluent quality deteriorated rather than showing the expected improvement. This may be partially due to the resuspension of some solids that settled to the lagoon bottom during runs with low power levels and to the increase in the influent strength as the power applied was increased for runs operated at the same loading. Increase in unfiltered effluent COD was the reverse of that observed for BOD_5 under the same conditions. Improvements in the filtered effluent COD quality were detected when power input into the facultative aerated lagoon increased. The improvement was greater at higher loadings, indicating that effluents from highly loaded facultative aerated lagoons will be improved at a higher rate, of COD concentration per unit power level, than those lightly loaded.

For runs operated at 30°C the effect of power level on effluent COD is illustrated in Figure 6.21b. It can be inferred from this figure that in the case of the COD of unfiltered effluent the effect of power level increase was detectable at the lower power input, whereas, at the higher power level this effect was not observed, even though influent COD was lower, for the same organic loading at high power levels. The COD of filtered effluent samples collected during the course of the experimental runs was not affected to any great extent by power level. Even when



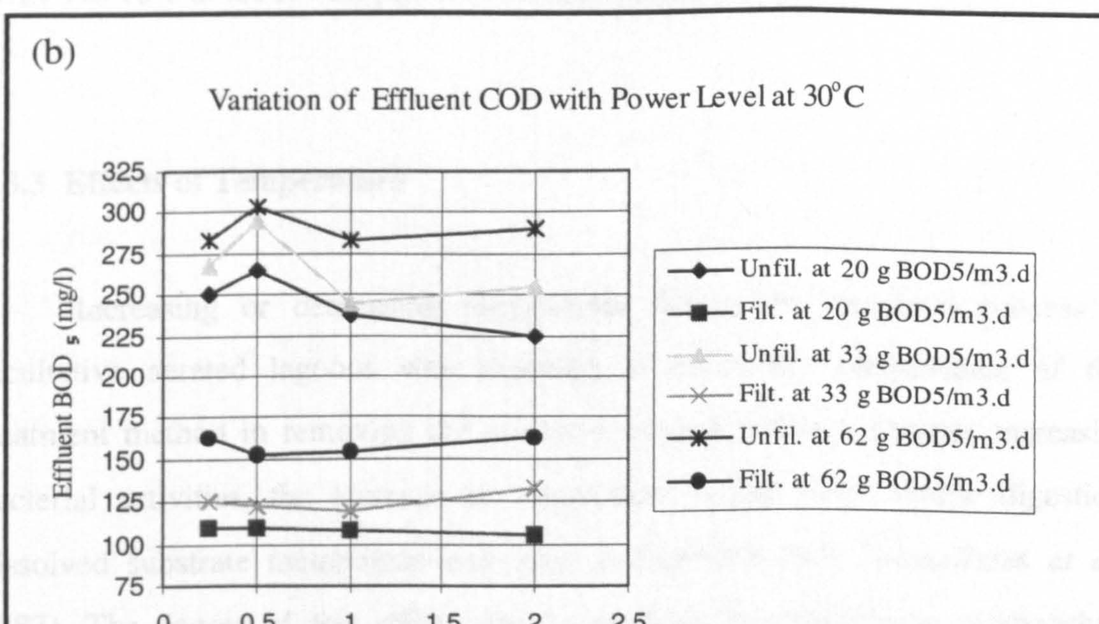


Figure 6.21 Variation of Effluent COD with Power Level.

low strength influent was used for high power level runs there was an increase in effluent strength in terms of COD.

It can be said that increase in power level at the lower temperature (20°C) caused the effluent COD of unfiltered samples to increase and of filtered samples to decrease. The effect of power level was reduced when operating temperature increased to 30°C, apparent from the low changes in COD of both types of samples that occurred when power level increased.

The improvement of settled effluent COD was studied as the power level in the facultative aerated lagoon was increased. This was found by determining the percentage improvmenet when two hours settling was provided. The effect of the power level on this improvement was significant for which the quality of the effluent, in terms of COD, improved from 7% at the highest power level applied (2 W/m^3) to 18% at the lowest power level applied (0.25 W/m^3).

6.3.3 Effects of Temperature

Increasing or decreasing temperature during the treatment process in facultative aerated lagoons was expected to affect the performance of this treatment method in removing the chemical oxygen demand. Besides increasing bacterial activities, the increase in temperature would affect sludge digestion, dissolved substrate metabolism and nutrient feedback rates (Sackellares *et al.*, 1987). The degree of this effect was investigated by determining relationships between temperature and the various parameters employed to describe the performance of the model facultative aerated lagoon with respect to COD.

i) Removal Rate Coefficients

At fixed organic loading and power level the effect of temperature on filtered (K_f)and unfiltered (K_u) COD removal rate coefficients is depicted in Figures 6.22a, 6.22b and 6.22c. At 20 g BOD_5/m^3.d (52 g COD/m^3.d) organic loading the increase in temperature had improved the value of COD K_f but this improvement was smaller than in the case of BOD_5 K_f under similar conditions. The effect of temperature on COD K_f was highest at the lower power level. No remarkable increase was noted in COD K_u with increase in temperature at this organic loading (20 g BOD_5/m^3.d or 52 g COD/m^3.d). The variations in both COD removal rate coefficients with temperature at 33 g BOD_5/m^3.d (85 g BOD/m^3.d) organic loading (Figure 6.22b) were not very different from that at 20 g BOD_5/m^3.d (52 g COD/m^3.d) organic loading. Increasing the organic loading to 62 g BOD_5/m^3.d, or 161 g COD/m^3.d, (Figure 6.22c) increased the effect of temperature on BOD_5 K_f, whereas COD K_u was affected by temperature much less than K_f.

The effect of temperature on COD removal rate coefficient was highest at the higher organic loading. This increase in temperature effect with increase in organic loading was not compatible with that for the BOD_5 removal rate coefficient. Filtered COD removal rate coefficient exhibited a greater response to temperature than the unfiltered COD removal rate coefficient. No clear trend was apparent in the effect of temperature on either COD removal rate coefficients when power level increased or decreased within the range tested.

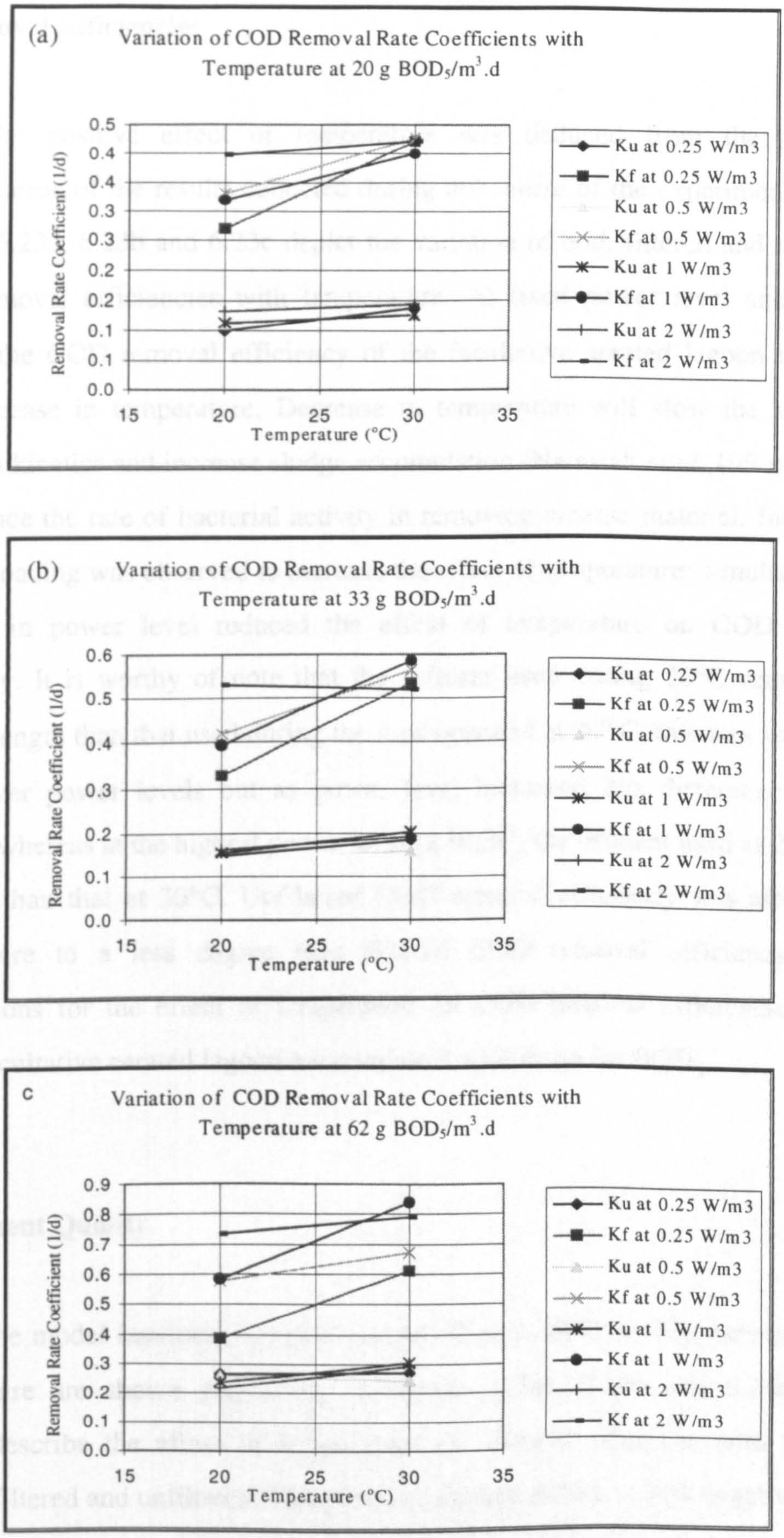


Figure 6.22 Variation of COD Removal Rate Coefficient with Temperature.

ii) Removal Efficiencies

The positive effect of temperature was deduced from the graphical representation of the results collected during the course of the experimental work. Figures 6.23a, 6.23b and 6.23c depict the variation of both filtered and unfiltered COD removal efficiencies with temperature. At fixed power level and organic loading the COD removal efficiency of the facultative aerated lagoon increased with increase in temperature. Decrease in temperature will slow the anaerobic digestion kinetics and increase sludge accumulation (Narasiah *et al.*,1990) and will also reduce the rate of bacterial activity in removing organic material. Increase in organic loading was observed to increase the effect of temperature; simultaneously, increase in power level reduced the effect of temperature on COD removal efficiency. It is worthy of note that the influent used during 20°C runs was of lower strength than that used during the runs operated at 30° C; this was so for runs with lower power levels but as power level increased this difference became smaller, whereas at the highest power level (2 W/m^3) the influent used at 20°C was stronger than that at 30°C. Unfiltered COD removal efficiency was affected by temperature to a less degree than filtered COD removal efficiency. These observations for the effect of temperature on COD removal efficiencies in the model facultative aerated lagoon are consistent with those for BOD_5.

iii) Effluent Quality

The model facultative aerated lagoon effluent COD quality variations with temperature are shown graphically in Figures 6.24a, 6.24b and 6.24c. These figures describe the effect of temperature on effluent COD for both types of sample, filtered and unfiltered. Temperature affected effluent COD negatively. The effluent was of lower COD at high temperature than at low temperature. The

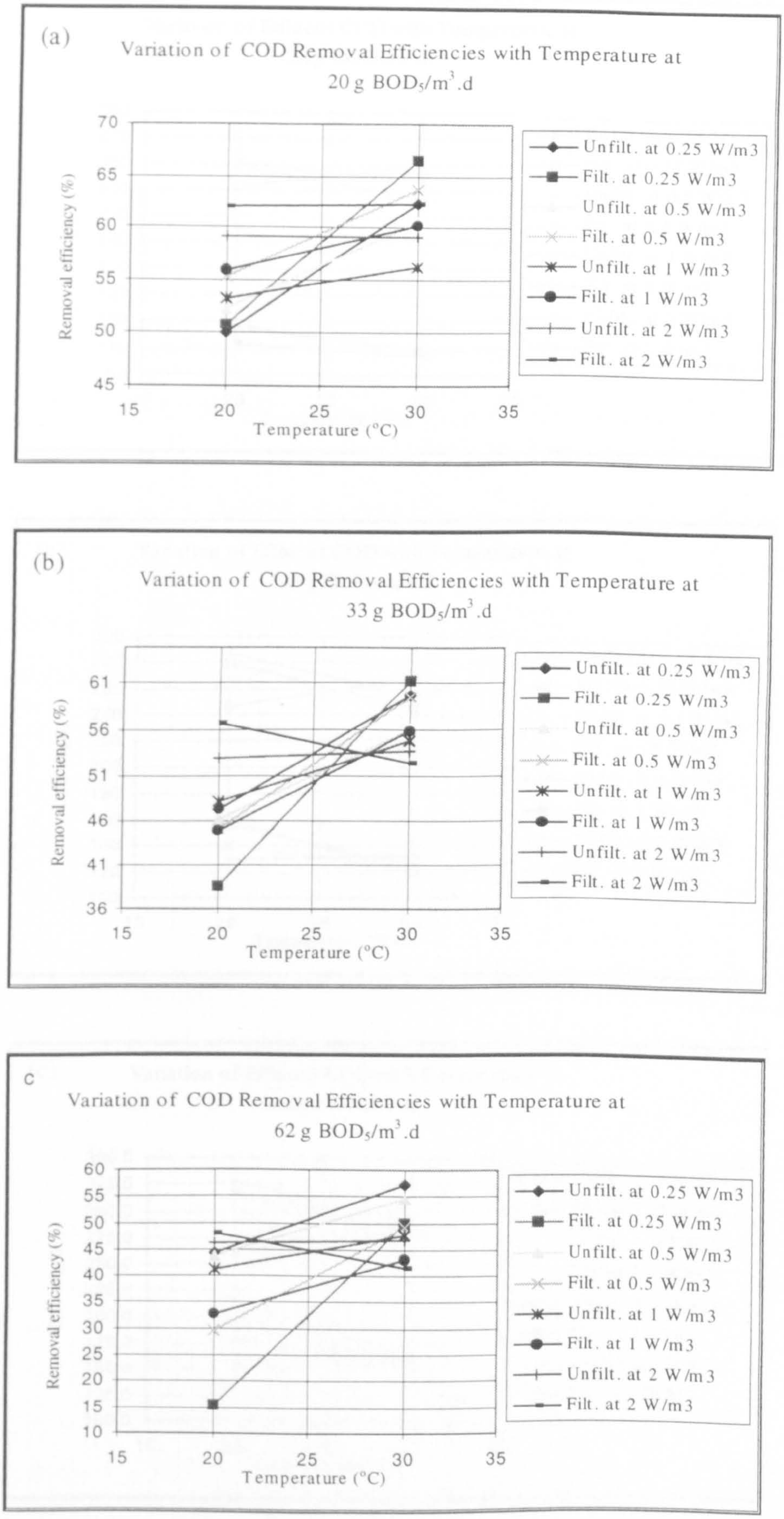


Figure 6.23 Variation of COD Removal Efficiency with Temperature.

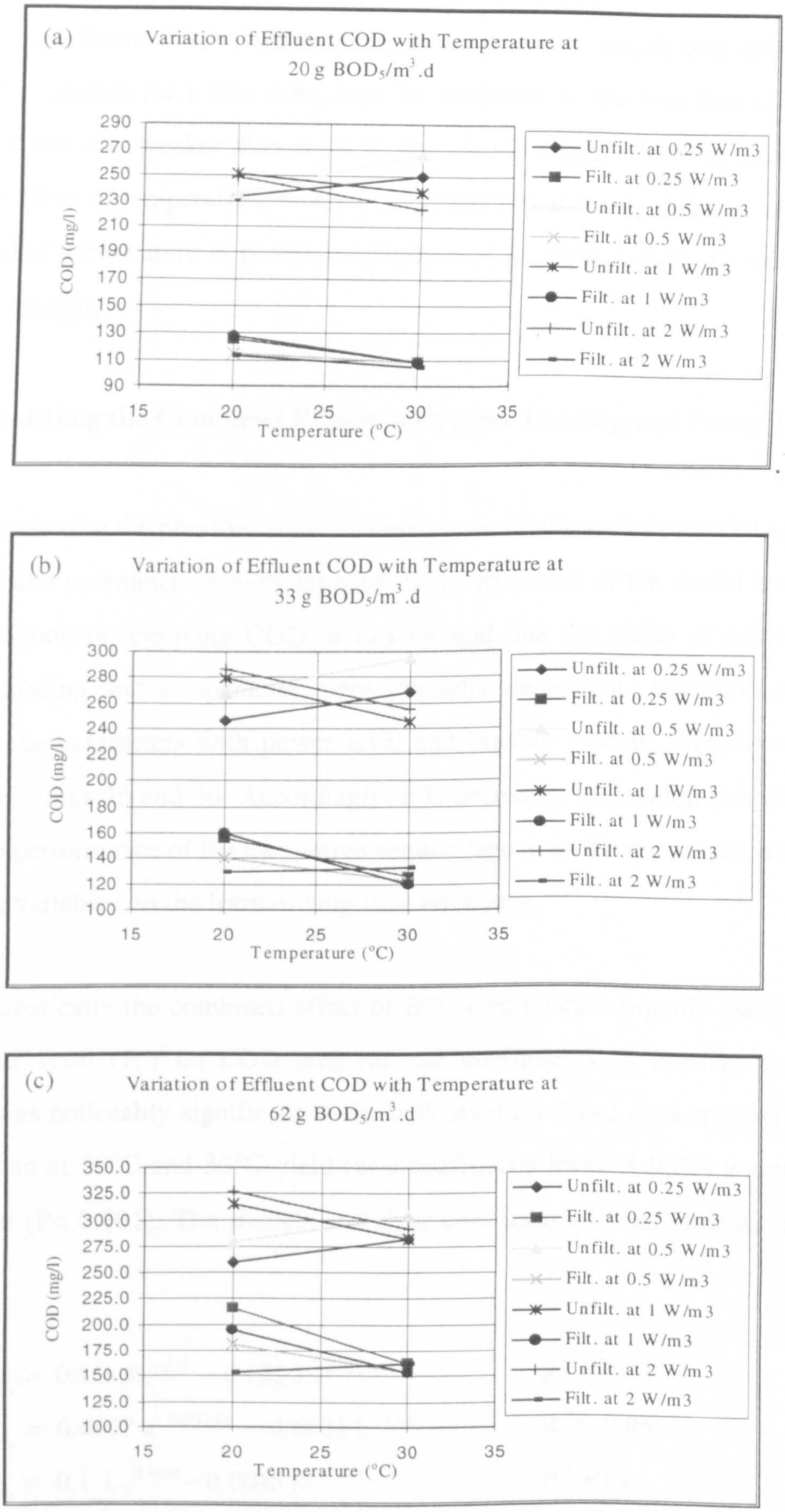


Figure 6.24 Variation of Effluent COD with Temperature.

increase in effluent COD with increase in temperature, which was observed for unfiltered samples for a few runs, may be attributed to the fact that at 20°C the influent COD was weaker than at 30°C. Increase in organic loading increased the negative effect of temperature; however, it would be extremely difficult to compare the effect of temperature as power level increased or decreased due to variations in influent strength.

6 3.4 Modelling the Combined Effects of Organic Loading and Power Level

Reviewing the previous figures showing the relationships between operating variables and parameters, which describe the performance of the model facultative aerated lagoon in removing COD, it can be said that the effect of power level, organic loading and temperature were mutually dependent. Variation of these performance parameters with power level and organic loading simultaneously is depicted Figures 25 and 26. Accordingly, it is necessary to develop models which relate the performance of the facultative aerated lagoon to the combination of these operating variables, in the form of empirical equations.

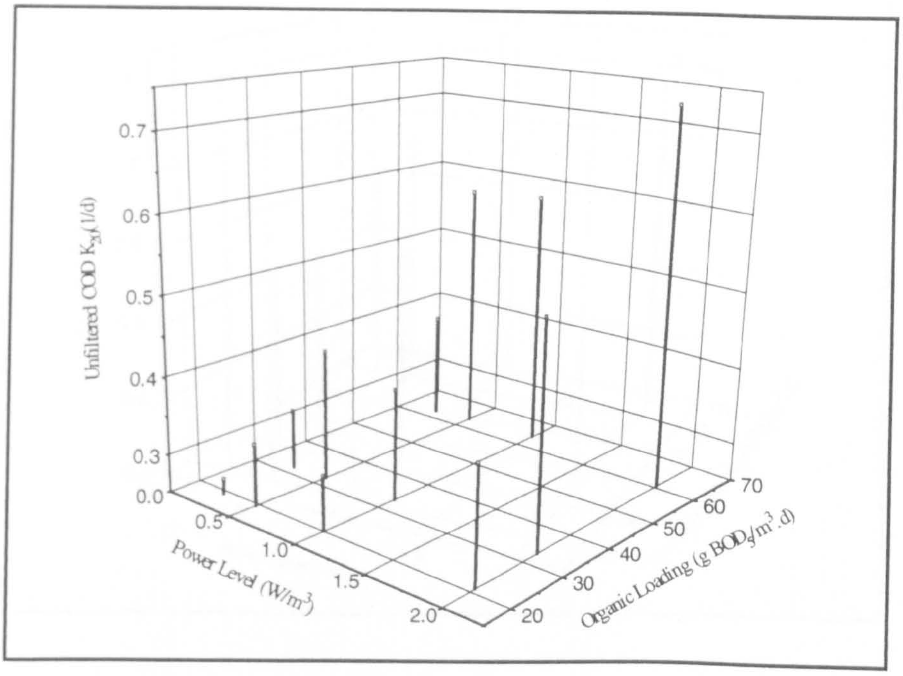
Statistically the combined effect of BOD_5 volumetric organic loading (L_V) and power level (P_V) on COD removal rate coefficients in facultative aerated lagoons was noticeably significant at the 0.05 level for those runs operated at 20° C. The data at 20°C and 30°C yield (at a confidence level of 95%) a significant regression ($P< 0.001$). The models and their coefficients of determination are as follows:

$$\text{COD} \quad K_{20_f} = 0.168\, P_V^{0.639} - 0.022\, L_V^{0.717} \qquad R^2 = 0.87 \qquad (6.33)$$

$$\text{COD} \quad K_{20_u} = 0.0607\, e^{-0.0217 P_V} - 0.0022\, L_V^{1.065} \qquad R^2 = 0.95 \qquad (6.34)$$

$$\text{COD} \quad K_{30_f} = 0.1\, L_V^{0.4804} - 0.0045\, P_V^{-1.81} \qquad R^2 = 0.9 \qquad (6.35)$$

$$\text{COD} \quad K_{30_u} = 0.0155\, L_V^{0.6904} + 0.00789\, P_V^{1.113} \qquad R^2 = 0.96 \qquad (6.36)$$



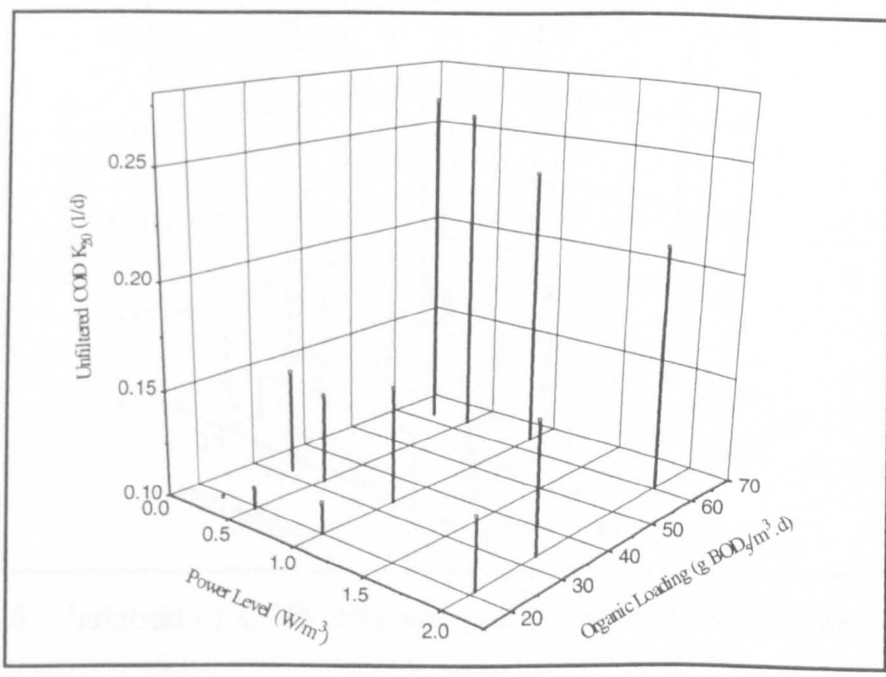


Figure 6.25 Variation of COD Removal Rate Coefficient with Power Level and Organic Loading at 20°C.

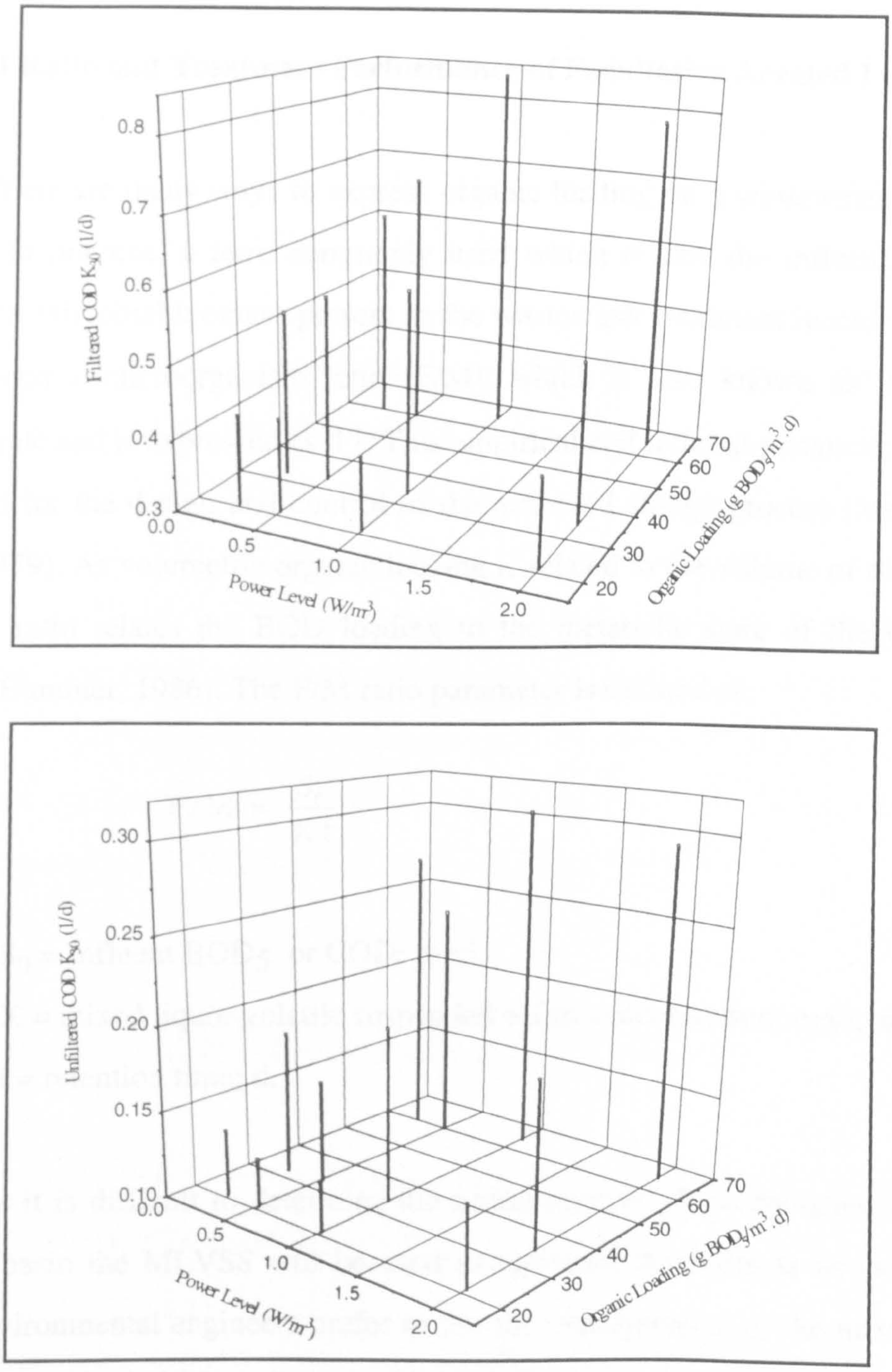


Figure 6.26 Variation of COD Removal Rate Coefficient with Power Level and Organic Loading at 30°C .

6.4 F/M Ratio and Treatment Performance of Facultative Aerated Lagoons

There are many ways to express organic loading on a wastewater treatment reactor. In practice, a term commonly used which relates the influent substrate loading to microbial biomass present in the wastewater treatment reactor is known as the food-to-microorganism ratio (F/M), which is also known as the sludge loading rate and is expressed as d^{-1}. This empirical and rational parameter has been proposed for the design and control of the activated sludge process (Metcalf and Eddy, 1979). As volumetric organic loading is related to the volume of the reactor, the F/M ratio relates the BOD loading to the metabolic state of the biological reactor (Hammer, 1986). The F/M ratio parameter is defined as:

$$F/M = \frac{S_0}{X\,t} \tag{6.37}$$

where, S_0 = Influent BOD_5, or COD; mg/l,

X = mixed liquor volatile suspended solids concentration; mg/l, and

t = retention time; d.

As it is difficult to determine the actual number of microorganisms in the aeration basin the MLVSS will be used to represent the biomass in the lagoon. Some environmental engineers prefer to use the concentration of the mixed liquor suspended solids (MLSS) rather than MLVSS. In the remainder of this section only BOD_5 and MLVSS will be used in determining the values of F/M ratio.

Since the concentration of MLSS or MLVSS in facultative aerated lagoons cannot be controlled because there is no sludge recycling and solids concentration will vary with varying applied BOD_5 to the lagoon and power level (section 6.5), F/M will not be considered as a controllable parameter. In this investigation, the variation of performance parameters of such lagoons with F/M ratio will be presented for illustration only. Changes of the sludge loading rate will cause the biological behaviour of the sludge to change and hence result in consequent changes in the rate of biological oxidation as well as the volume of microbial biomass produced (Gray, 1989; Horan, 1990).

In this study, with the levels of the controllable factors of power input, organic loading and temperature the F/M ratio was between 0.41 d^{-1} and 1.74 d^{-1} High values were observed to be associated with high organic loading and low power levels. This range of F/M ratio values is greater than those given by Metcalf and Eddy (1979) as a design parameter for the conventional activated sludge process and most of its modifications. The reason for these high sludge loadings in facultative aerated lagoons is mainly attributable to the low solids concentration maintained in suspension. The following subsections will present both BOD_5 and COD removal rate coefficient variations with this parameter.

6.4.1 BOD_5 Removal Rate Coefficients Versus F/M Ratio

Variations in the values of BOD_5 removal rate coefficient with F/M ratio for various levels of power and temperatures are plotted in Figures 6.27a and 6.27b.

Figure 6.27a shows variations in the values of filtered BOD_5 removal rate coefficient with F/M ratio. The filtered BOD_5 removal rate coefficient in such a biological treatment process was observed to be inversely related to F/M ratio. The correlation was significant; however, the increase in F/M ratio value was not accompanied by any adverse reduction in the value of the filtered BOD_5 removal rate coefficient. As temperature increased to 30°C, variation with this parameter (F/M ratio) was relatively greater than at the lower temperature (20°C). At both

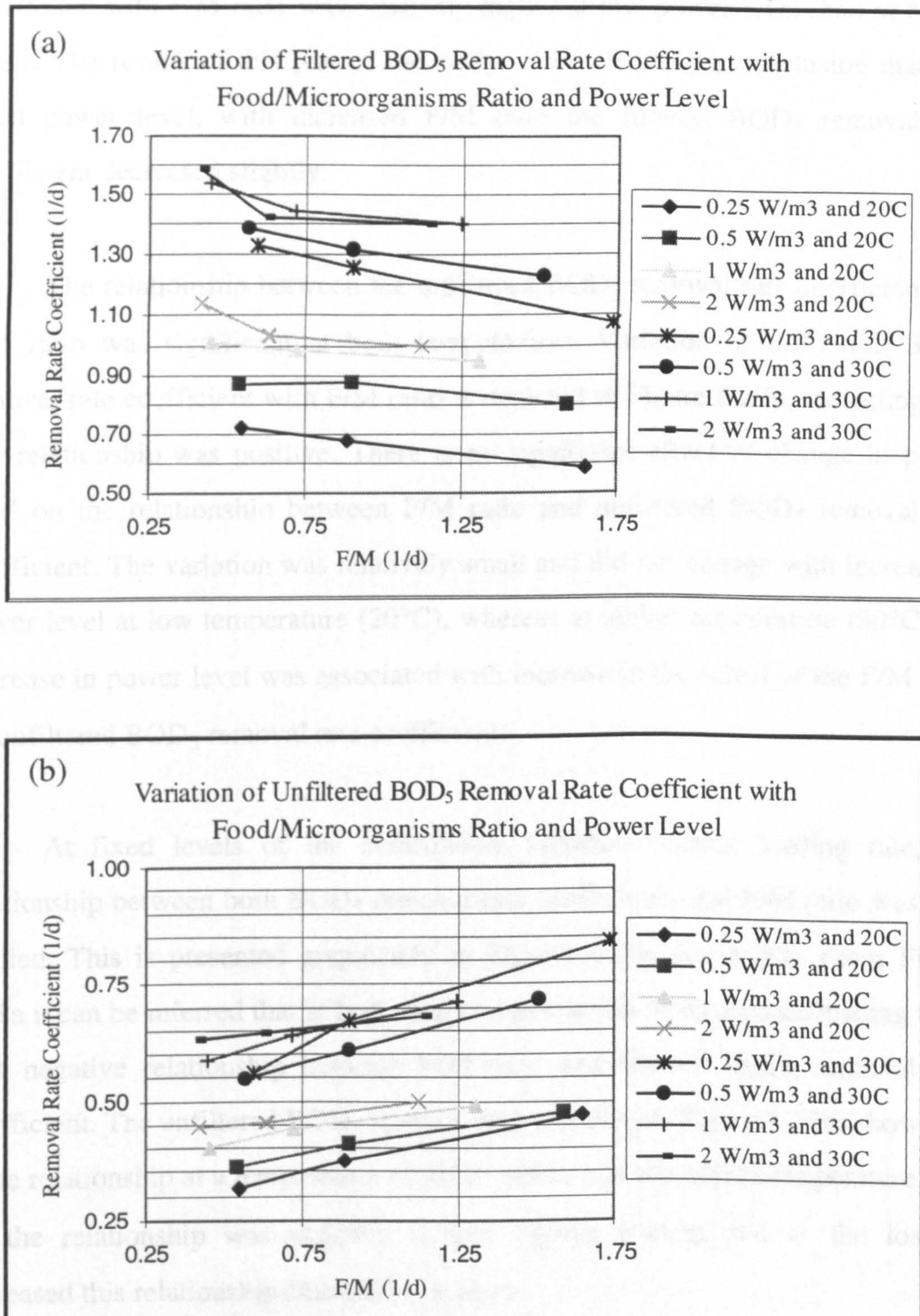
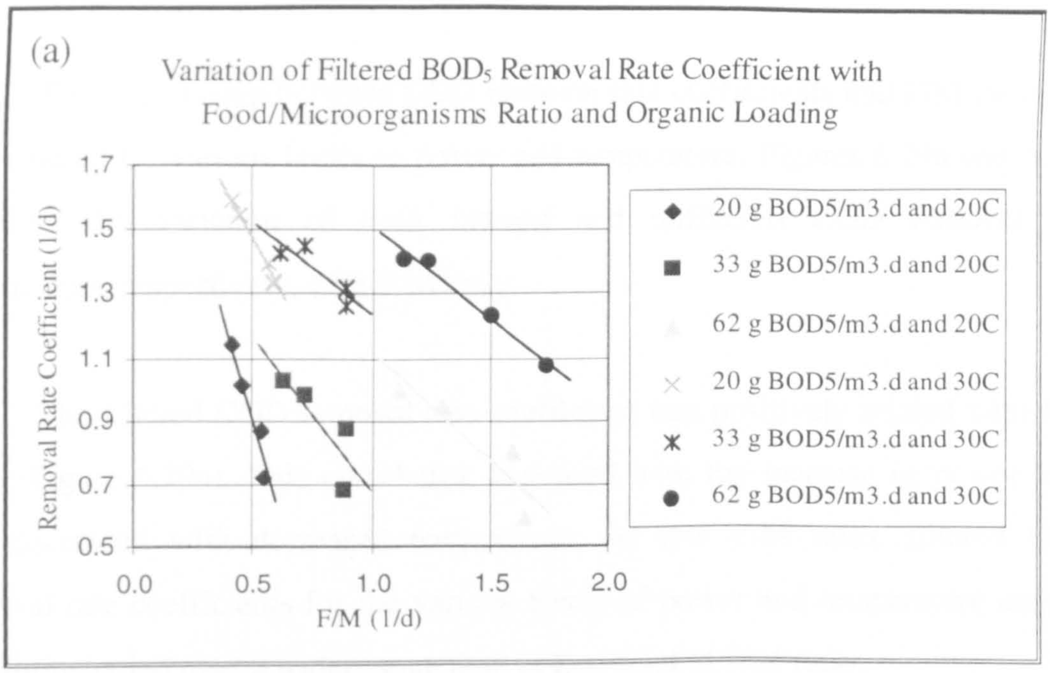


Figure 6.27 Variation of BOD_5 Removal Rate Coefficient with Food/Microorganisms Ratio and Power Level.

temperatures the negative relationship of the filtered BOD_5 removal rate coefficient with F/M ratio was relatively higher at low power level than at higher levels. The results of this part of the analysis thus led to the conclusion that at a fixed power level, with increased F/M ratio the filtered BOD_5 removal rate coefficient decreased slightly.

The relationship between the unfiltered BOD_5 removal rate coefficient and F/M ratio was significant at both temperatures. Variation of unfiltered BOD_5 removal rate coefficient with F/M ratio is depicted in Figure 6.27b, indicating that this relationship was positive. There is no significant effect of change in power level on the relationship between F/M ratio and unfiltered BOD_5 removal rate coefficient. The variation was relatively small and did not change with increase in power level at low temperature (20°C), whereas at higher temperature (30°C) the decrease in power level was associated with increase in the effect of the F/M ratio on unfiltered BOD_5 removal rate coefficient.

At fixed levels of the controllable variable, organic loading rate, the relationship between both BOD_5 removal rate coefficients and F/M ratio was also studied. This is presented graphically in Figures 6.28a and 6.28b. From Figure 6.28a it can be inferred that at both temperatures and at fixed organic loading there is a negative relationship between F/M ratio and filtered BOD_5 removal rate coefficient. The unfiltered BOD_5 removal rate coefficient (Figure 6.28b) shows the same relationship at a temperature of 20°C, whereas at the higher temperature (30° C) the relationship was negative at low organic loading and as the loading increased this relationship changed to positive.



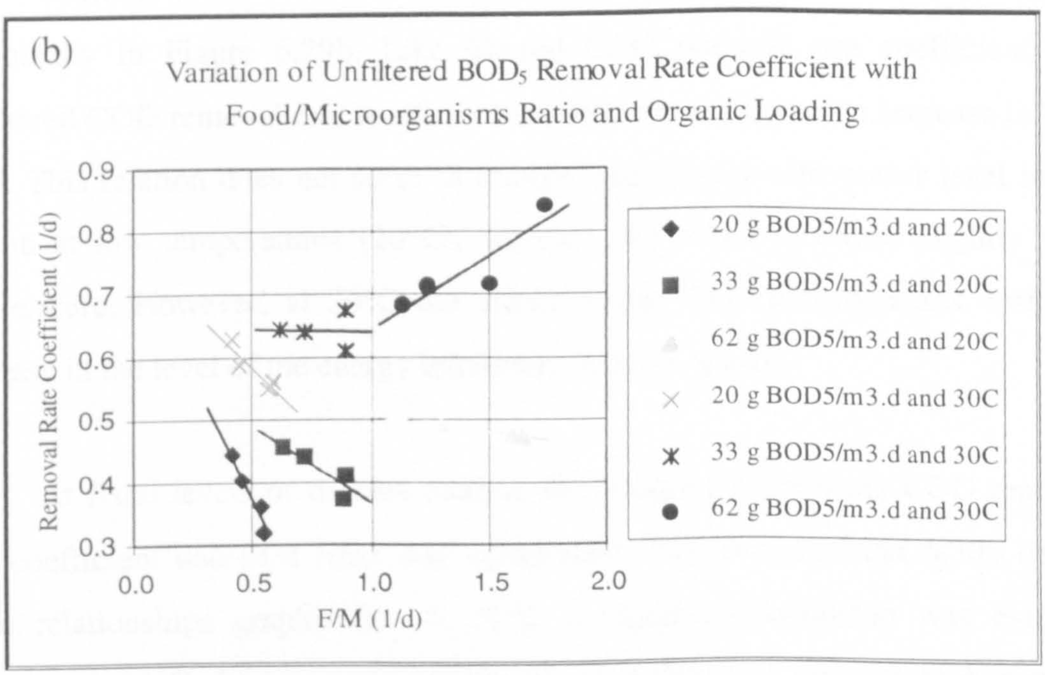


Figure 6.28 Variation of BOD_5 Removal Rate Coefficient with Food/Microorganisms Ratio and Organic Loading.

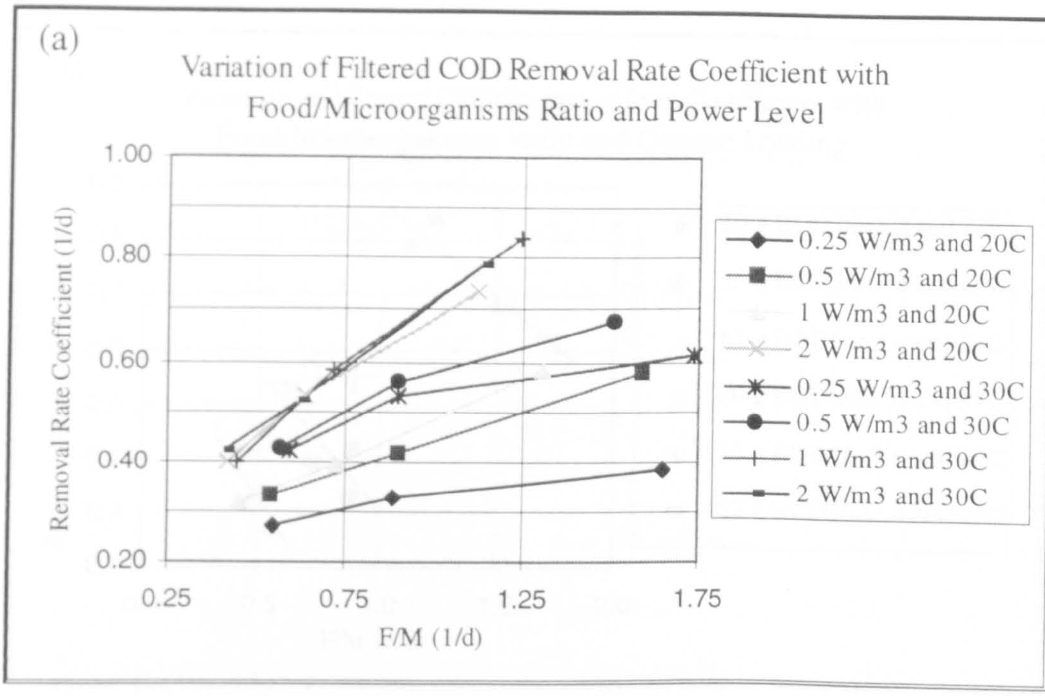
6.4.2 COD Removal Rate Coefficients Versus F/M Ratio

The relationship between COD removal rate coefficients and F/M ratio was also studied for various levels of power and temperature. Figures 6.29a and 6.29b illustrate the variation of both filtered and unfiltered COD removal rate coefficients, respectively, with F/M ratio.

The filtered COD removal rate coefficient was positively related with F/M ratio (Figure 6.29a). This correlation increased with the increase in power level and decreased with decreased temperature. At low F/M ratio, filtered COD removal rate coefficients for the various levels of power and temperature used in this study varied over a wider range than at low level of F/M ratio.

Unfiltered COD removal rate coefficient variation with F/M ratio is shown graphically in Figure 6.29b. Like filtered COD removal rate coefficient, the unfiltered COD removal rate coefficient is positively related to the increase in F/M ratio. This relation does not seem to change significantly with power level in the lagoon at low temperatures (20°C), whereas this effect increases slightly with temperature. However, at 30°C the effect of the F/M ratio increases with the increase in the level of the energy introduced into the lagoon.

At fixed levels of organic loading the relationship between COD removal rate coefficient and F/M ratio was determined. Figures 6.30a and 6.30b depict these relationships graphically. At 20°C a negative relationship was evident, whereas at 30°C the negative relationship was observed only at high organic loading. Unfiltered COD removal rate coefficient (Figure 6.30b) was positively related to F/M ratio only at high organic loading.



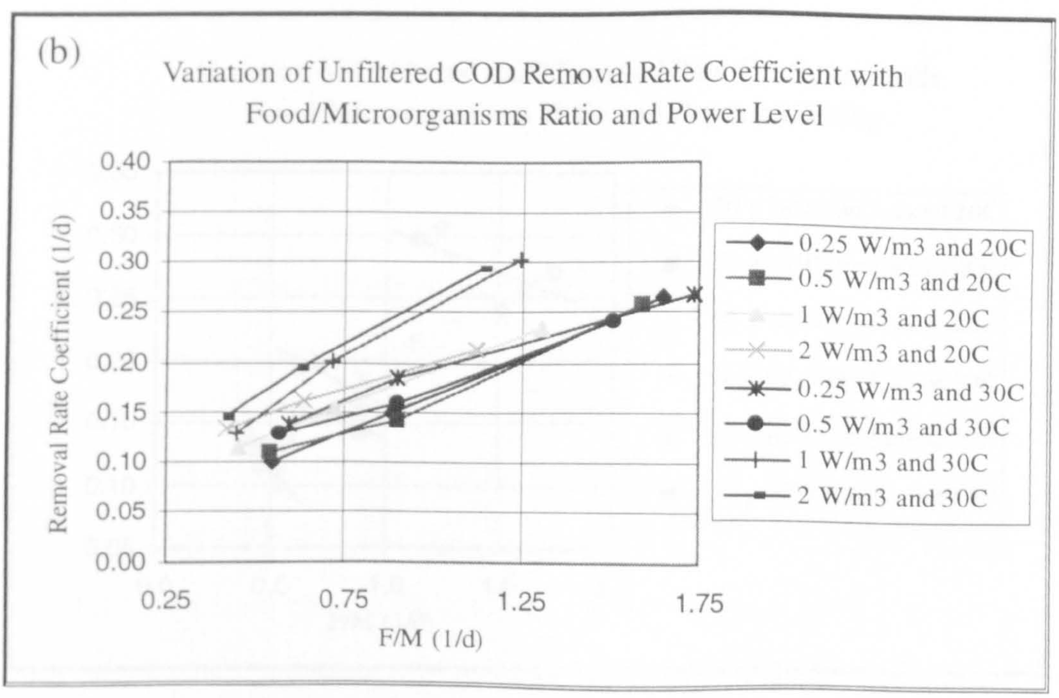
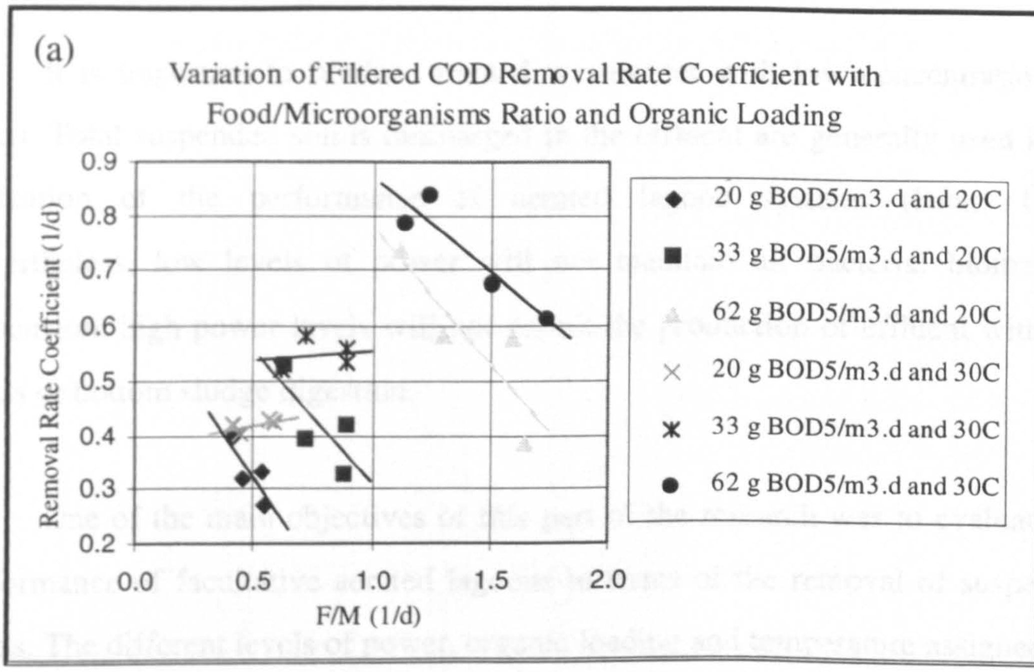


Figure 6.29 Variation of COD Removal Rate Coefficient with Food/Microorganisms Ratio and Power Level.



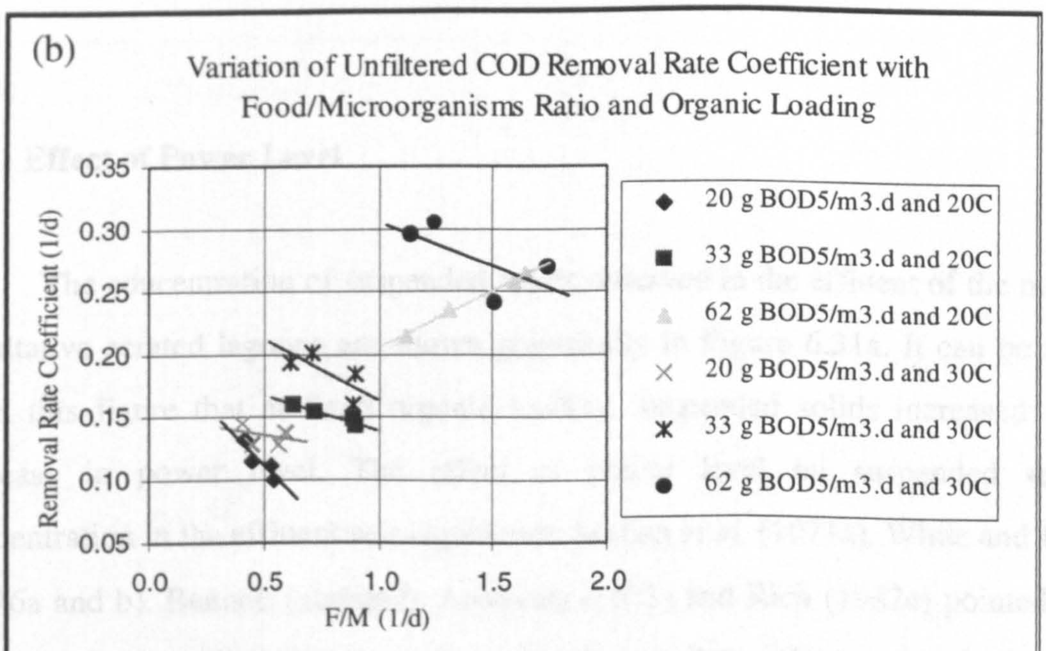


Figure 6.30 Variation of COD Removal Rate Coefficient with Food/Microorganisms Ratio and Organic Loading.

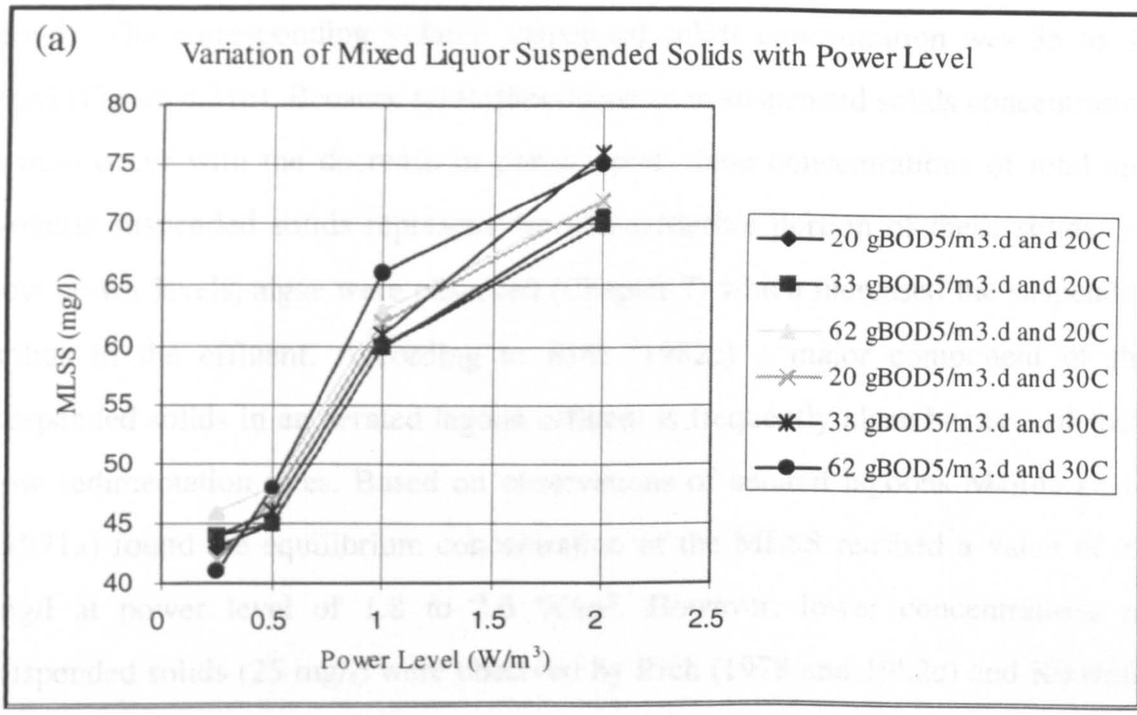
6.5 Solids Production and Removal in Facultative Aerated Lagoons

It is important to produce treated wastewater with low concentrations of solids. Total suspended solids discharged in the effluent are generally used in the evaluation of the performance of aerated lagoon systems (Rich, 1980). Nevertheless, low levels of power will not maintain all bacterial biomass in suspension; high power levels will not permit the production of effluent with low solids or bottom sludge digestion.

One of the main objectives of this part of the research was to evaluate the performance of facultative aerated lagoons in terms of the removal of suspended solids. The different levels of power, organic loading and temperature assigned and applied in this research were considered to determine their effect on such performance.

6.5.1 Effect of Power Level

The concentration of suspended solids observed in the effluent of the model facultative aerated lagoons are shown graphically in Figure 6.31a. It can be seen from this figure that at fixed organic loading, suspended solids increased with increase in power level. The effect of power level on suspended solids concentration in the effluent was significant. Malina *et al.* (1971a), White and Rich (1976a and b), Bennett (undated), Arceivala (1981) and Rich (1982c) pointed out that the degree of mixing is one of the factors that affect suspended solids concentration under steady state. It is clear from the figure that the lowest effluent suspended solids concentration was 41 to 46 mg/l. A similar concentration was observed when the power level was reduced to less than 0.25 W/m^3. Also similar concentration might be expected with extrapolation for zero power level, a



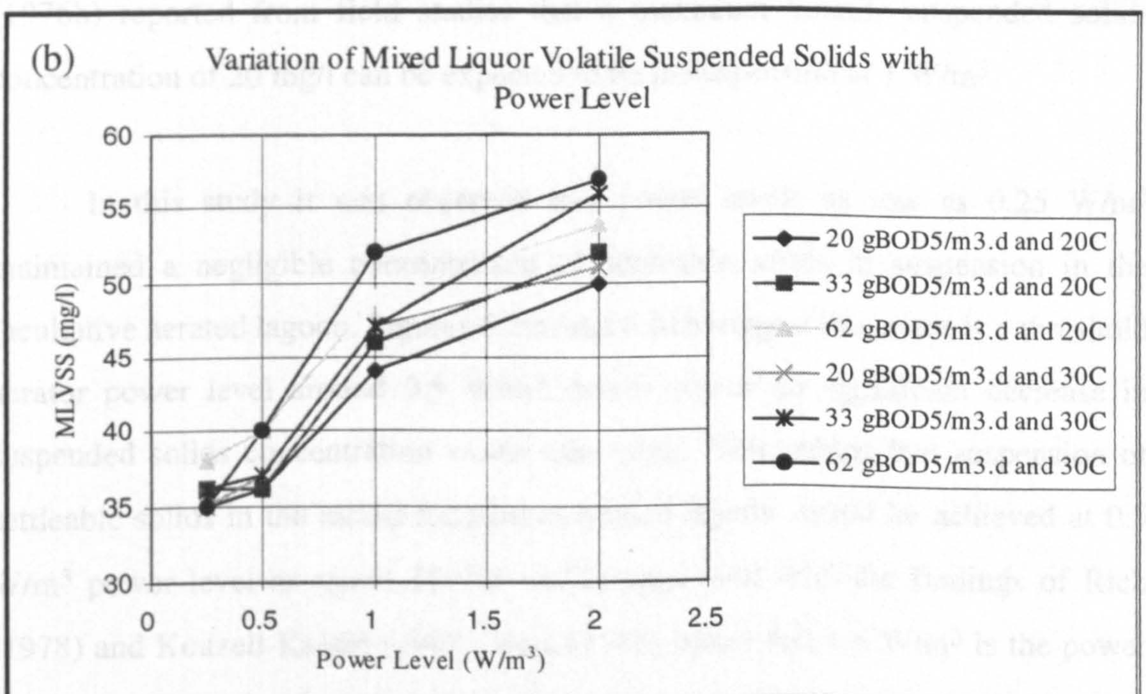


Figure 6.31 Variation of Mixed Liquor Suspended Solids with Power Level.

condition which means no mechanical power input, as in wastewater stabilization ponds. The corresponding volatile suspended solids concentration was 35 to 38 mg/l (Figure 6.31b). Because no further decrease in suspended solids concentration would occur with the decrease in power level, these concentrations of total and volatile suspended solids represent the non-settleable portion of these solids. At low power levels, algae were observed (Chapter 7) which increased the suspended solids in the effluent. According to Rich (1982c) a major component of the suspended solids in an aerated lagoon effluent is frequently algae because of their low sedimentation rates. Based on observations of aerated lagoons Malina *et al.* (1971a) found the equilibrium concentration of the MLSS reached a value of 55 mg/l at power level of 1.8 to 2.6 W/m^3. However, lower concentrations of suspended solids (25 mg/l) were observed by Rich (1978 and 1982c) and Kouzell-Katsiri (1987). The results of the present study are intermediate between the above studies with respect to the concentration of suspended solids. White and Rich (1976b) reported from field studies that a maximum volatile suspended solids concentration of 20 mg/l can be expected to be in suspension at 1 W/m^3.

In this study it was observed that power levels as low as 0.25 W/m^3 maintained a negligible concentration of settleable solids in suspension in the facultative aerated lagoon. Figures 6.31a and 6.31b suggest that there is a threshold aerator power level around 0.5 W/m^3, below which no significant decrease in suspended solids concentration would take place. This means that suspension of settleable solids in the model facultative aerated lagoon would be achieved at 0.5 W/m^3 power level or more. This is not in agreement with the findings of Rich (1978) and Kouzell-Katsiri (1987). Rich (1978) found that 1.6 W/m^3 is the power level at which settleable solids start to be maintained in suspension, while Kouzell-Katsiri (1987) concluded from his study on a laboratory scale aerated lagoon treating domestic wastewater that a value of 1 W/m^3 represents the threshold power level for partial suspension of solids. This inconsistency may be attributable to differences in lagoon size, method of aeration or possibly to a difference in mixing intensity at the same power levels.

The effect of power level on suspended solids concentration was slightly higher than for volatile suspended solids, as can be inferred from Figures 6.31a and 6.31b. Both were observed to increase with power level at higher rates during operational runs at low ranges of power level compared with those at higher ranges.

It was attempted in this research to determine the effect of power level (as an indication of aeration) on the production of biomass, which is represented by the percentage of the suspended solids that were volatile (%VSS). Figure 6.32 shows the variation of %VSS with power level. Over the range of power level, organic loading and temperature used in this research, the percentage of the volatile part of the suspended solids was between 70% and 85%. The typical percentage for facultative aerated lagoons, as cited by Arceivala (1981), is 60% to 80%, while Kouzell-Katsiri (1987) from his laboratory scale facultative aerated lagoon found that a value of 71% was the average percentage of volatile solids. A significant negative effect of power level on the volatile percentage was observed. The decrease in the values of %VSS with power may be the result of an increase in the concentration of inorganic particulates held in suspension by the elevated levels of turbulence occurring in the lagoon. In previous studies, Rich (1978) and Arceivala (1981) reported the marked effect of aeration power upon VSS/SS. The negative effect of power level on this coefficient (VSS/SS) is in agreement with the observations of Rich (1978). Similarly this negative effect could by inferred from the study of McKinney and Benjes (1965). They reported that the volatile fraction in settled sludge of a pilot aerated lagoon increased with distance from the rotor.

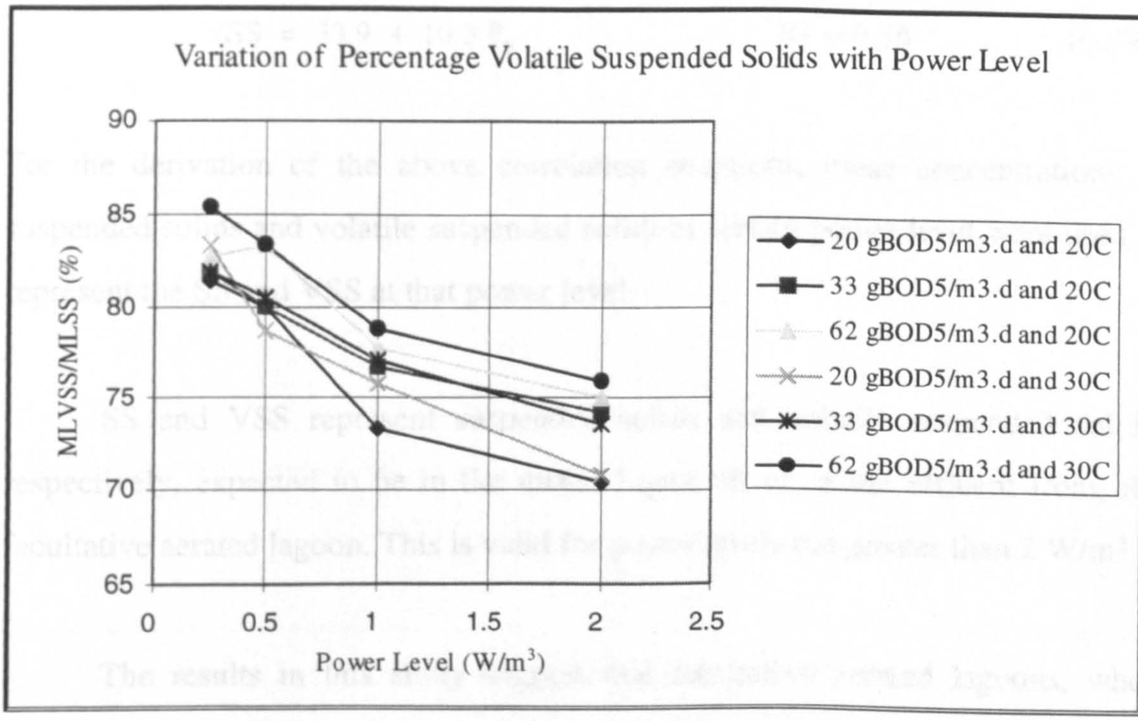


Figure 6.32 Variation of Percentage Volatile Suspended Solids with Power Level.

For the model facultative aerated lagoon used in this study and the diffused aeration method employed, correlation equations and correlation coefficients of the suspended solids with power level are as follows:

$$SS = 40 + 17.2\,P_v \qquad R^2 = 0.92 \qquad (6.38)$$

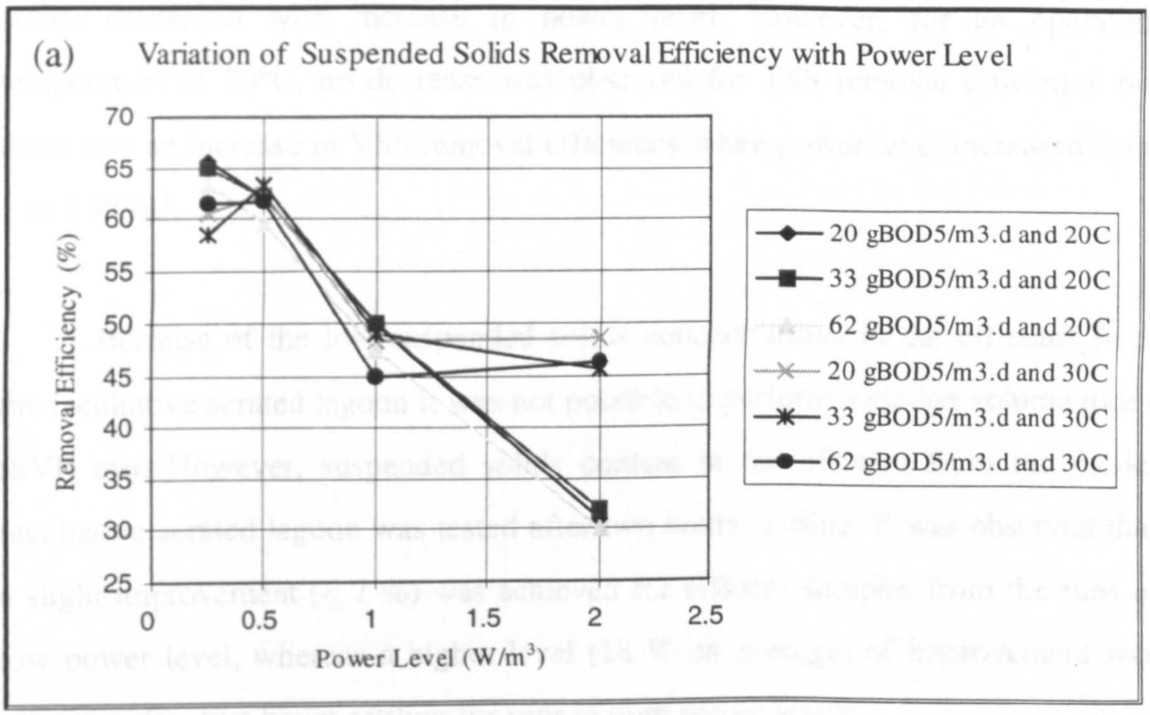
$$VSS = 33.9 + 10.3\,P_v \qquad R^2 = 0.86 \qquad (6.39)$$

For the derivation of the above correlation equations, mean concentrations of suspended solids and volatile suspended solids at certain power level were used to represent the SS and VSS at that power level.

SS and VSS represent suspended solids and volatile suspended solids, respectively, expected to be in the mixed liquor of, or in the effluent from, the facultative aerated lagoon. This is valid for power levels not greater than 2 W/m^3.

The results in this study suggest that facultative aerated lagoons, when treating domestic sewage, can be used as a secondary treatment process for producing effluents with 76 mg/l or less suspended solids provided that the power level introduced is 2 W/m^3 or less. Higher power inputs are expected to increase effluent suspended solids concentration and may render this method less effective in removing solids if used on its own. Malina *et al.* (1971a) reported that a full-scale aerated lagoon operated at 6.4 W/m^3 cannot be used as a single treatment unit because there is no removal of suspended solids and the total effluent substrate concentrations are very high.

Suspended solids (both total and volatile) removal efficiencies were shown to be between 31 and 66 percent over the range of power level, organic loading and temperature applied in this research. Percentage removal of TSS and VSS versus power level are depicted in Figures 6.33a and 6.33b respectively. It can be inferred



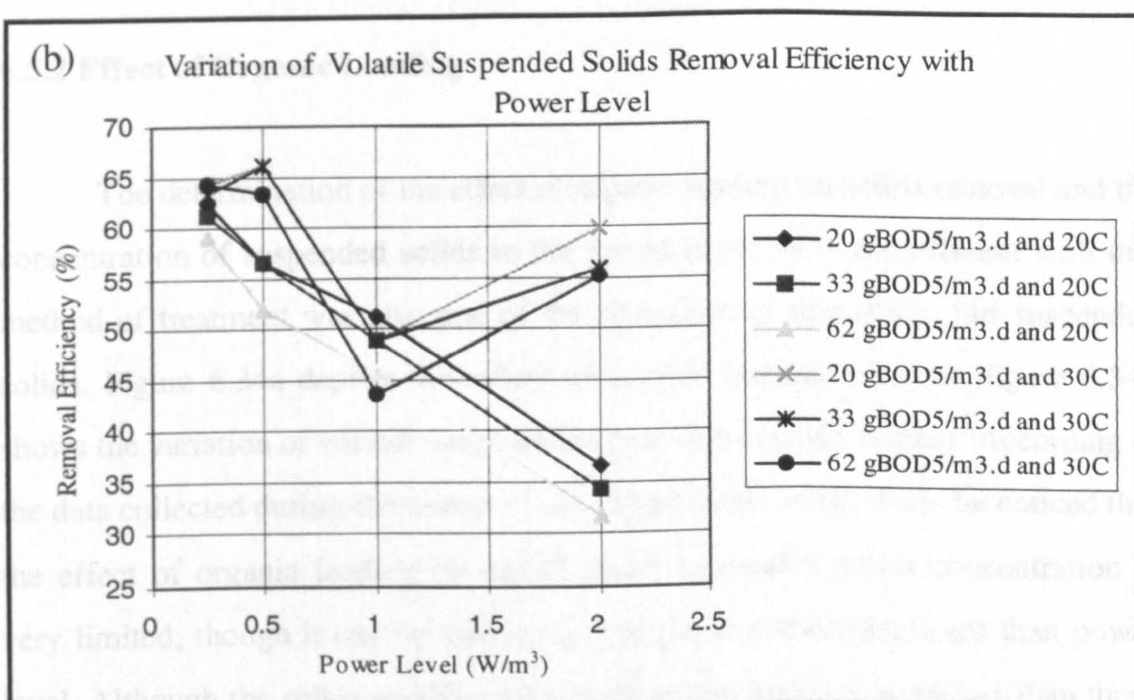


Figure 6.33 Variation of Suspended Solids Removal Efficiency with Power Level.

from these two figures that percentage removal of both total and volatile suspended solids decreased with increase in power level. However, for an operating temperature of 30°C, no decrease was observed for TSS removal efficiency but there was an increase in VSS removal efficiency when power level increased from 1 to 2 W/m^3.

Because of the low suspended solids concentrations in the effluents from the facultative aerated lagoon it was not possible to perform a sludge volume index (SVI) test. However, suspended solids content in the effluent from the model facultative aerated lagoon was tested after two hours settling. It was observed that a slight improvement (< 7 %) was achieved for effluent samples from the runs at low power level, whereas a higher level (18 % on average) of improvement was apparent after two hours settling for runs at high power levels.

6.5.2 Effect of Organic Loading

The determination of the effect of organic loading on solids removal and the concentration of suspended solids in the mixed liquor or in the effluent with this method of treatment was also one of the objectives of this study. For suspended solids, Figure 6.34a depicts this effect of organic loading, whereas Figure 6.34b shows the variation of volatile suspended solids with organic loading. According to the data collected during the course of the experimental work, it can be noticed that the effect of organic loading on mixed liquor suspended solids concentration is very limited; though it can be said to be positive it is less significant than power level. Although the solids entering the system at low loadings were less than those at high loadings, the biological growth at such low loadings was dispersed and tended to be in suspension. Thus the counter effect of reducing organic loading did not greatly decrease the suspended solids. From Balasha and Sperber (1975) data

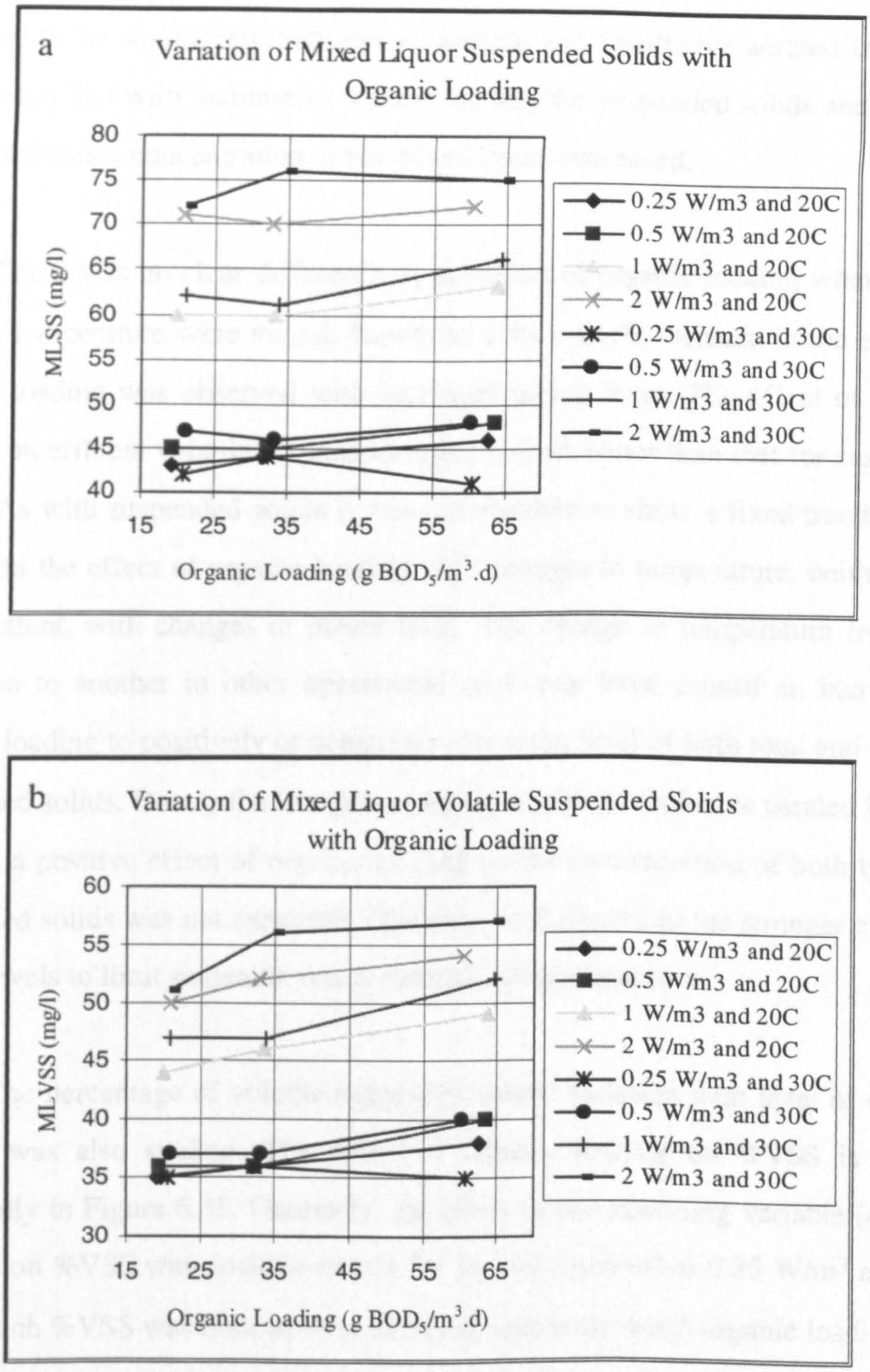


Figure 6.34 Variation of Mixed Liquor Suspended Solids with Organic Loading.

in their study using a full scale aerated lagoon operated at 2.7 W/m^3, which they classified to be somewhere between an aerobic and facultative aerated lagoon, it can be seen that with increase in organic loading the suspended solids and volatile suspended solids concentration in the mixed liquor increased.

There was no clear difference in the effect of organic loading when power level or temperature were varied. However, a very slight increase in the effect of organic loading was observed with increased power level. The effect of organic loading on effluent volatile suspended solids was no better than that for suspended solids. As with suspended solids it was not possible to show a fixed trend for the change in the effect of organic loading with changes in temperature; neither, to a lesser extent, with changes in power level. The change in temperature from one condition to another in other operational runs may have caused an increase of organic loading to positively or negatively affect the level of both total and volatile suspended solids. Due to the low power levels at which facultative aerated lagoons operate, a positive effect of organic loading on the concentration of both types of suspended solids was not expected. This may be attributed to the stronger effect of power levels to limit settleable solids maintained in suspension.

The percentage of volatile suspended solids' variation with level of organic loading was also studied. The effect of organic loading on %VSS is shown graphically in Figure 6.35. Generally, the effect of this operating variable (organic loading) on %VSS was positive except for the run operated at 0.25 W/m^3 and 30° C, at which %VSS was noticed to be affected negatively when organic loading was increased from 20 to 33 g BOD_5 $/m^3.d$. In conclusion it can be said that increase in organic loading led to an increase in the percentage of volatile suspended solids in the model facultative aerated lagoon's mixed liquor and effluent except at high temperature and low level of aeration power, when it (organic loading) was increased from 20 to 33 g BOD_5 $/m^3.d$.

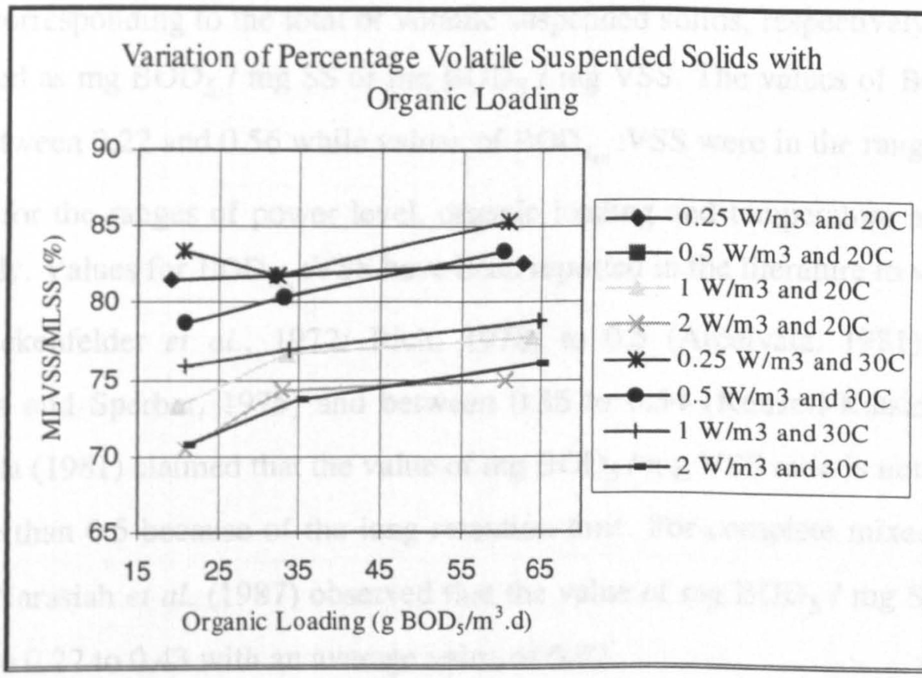


Figure 6.35 Variation of Percentage Volatile Suspended Solids with Organic Loading.

The ratio of effluent particulate (nonsoluble) BOD_5 (total minus soluble BOD_5), $BOD_{5_{NS}}$, to effluent suspended solids (total or volatile) for the various runs was determined for each run. This was achieved by dividing the difference between unfiltered BOD_5 and filtered BOD_5 by total suspended solids ($BOD_{5_{NS}}$:SS) or volatile suspended solids ($BOD_{5_{NS}}$:VSS). These ratios represent the BOD_5 corresponding to the total or volatile suspended solids, respectively, and are expressed as mg BOD_5 / mg SS or mg BOD_5 / mg VSS. The values of $BOD_{5_{NS}}$:SS were between 0.22 and 0.56 while values of $BOD_{5_{NS}}$:VSS were in the range of 0.26 to 0.69 for the ranges of power level, organic loading and temperature applied in this study. Values for $BOD_{5_{NS}}$:VSS have been reported in the literature to vary from 0.42 (Eckenfelder *et al.*, 1972; Rich, 1978) to 0.5 (Arceivala, 1981) to 0.54 (Balasha and Sperber, 1975) and between 0.36 to 0.54 (Kouzell-Katsiri, 1987). Arceivala (1981) claimed that the value of mg BOD_5 / mg VSS ratio is not likely to be more than 0.5 because of the long retention time. For complete mixed aerated lagoon Narasiah *et al.* (1987) observed that the value of mg BOD_5 / mg SS was in the range 0.22 to 0.43 with an average value of 0.32.

The BOD/SS ratios were 1.5 to 2.41 in the influent and 0.42 to 1.7 in the effluent. Similar reductions in the value of this ratio were observed by Narasiah *et al.* (1987) who justified this by the conversion of a considerable fraction of the volatile organic matter in the lagoon (which contributes to the total BOD_5) to non-volatile suspended matter as a result of long periods of hydraulic retention as well as intense aeration.

A slight decrease in removal efficiency for both TSS and VSS was observed when organic loading was increased. This is shown graphically in Figures 6.36a and 6.36b, respectively. The effect of organic loading on the percentage removals of TSS and VSS was not significant.

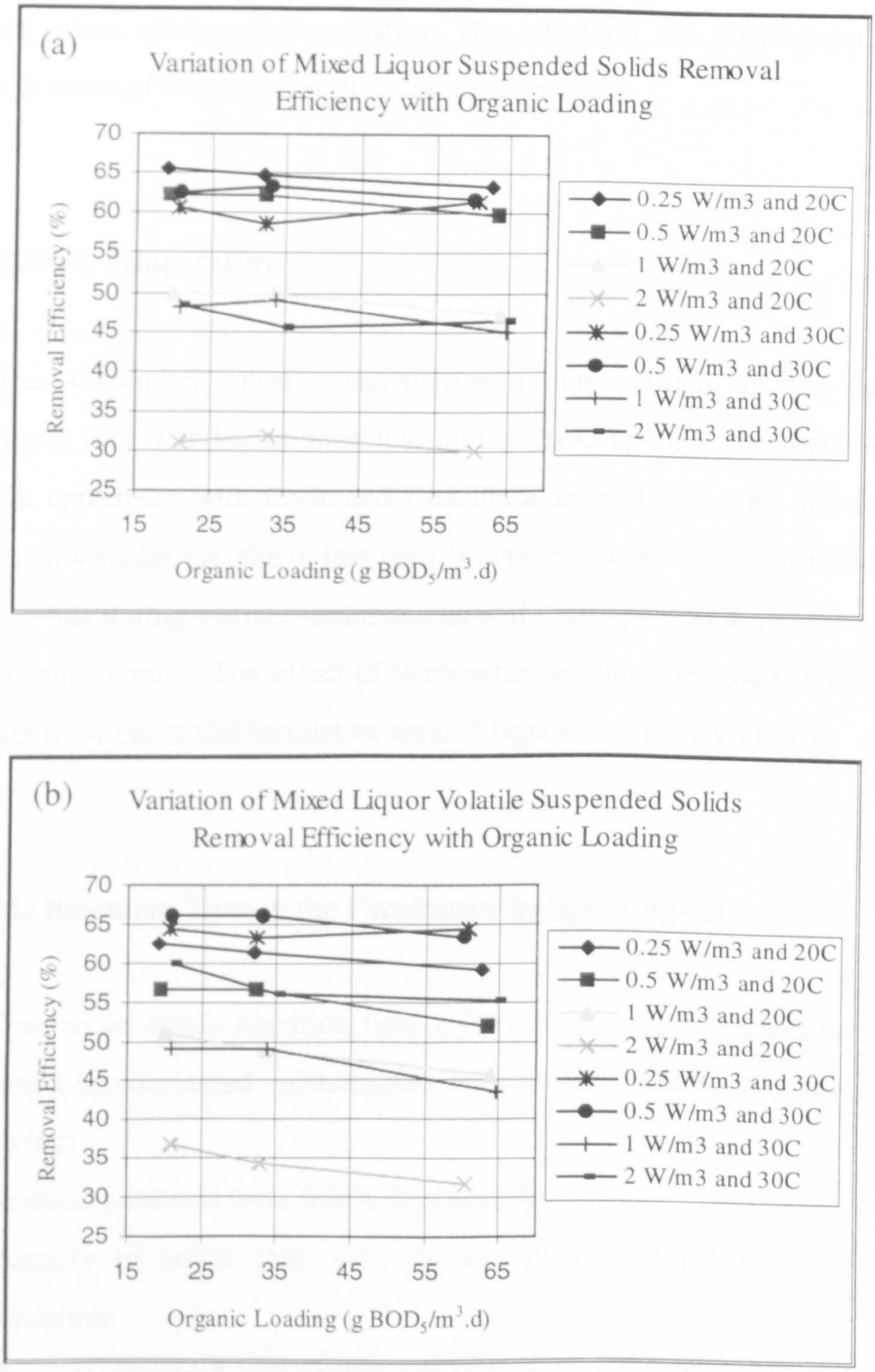


Figure 6.36 Variation of Suspended Solids Removal Efficiency with Organic Loading.

Settling tests for the effluent samples showed that organic loading had little effect on solids settling characteristics. The effect on the improvement of the effluent in terms of suspended solids was not significant.

6.5.3 Effect of Temperature

The suspended solids' concentration in the effluent during the 20°C experimental runs was higher than that during 30°C operating temperature runs. This is in agreement with Cook and Chandrasekaran (1986) who noted that SS discharge in summer months is less than in winter months and is justified by the fact that solids during warmer temperatures will decompose at a quicker rate than in cooler temperatures. The effect of temperature on the suspended solids content of effluent from the model facultative aerated lagoon was not significant.

6.6 Solids Retention Time in the Facultative Aerated Lagoon

True mean solids retention time (SRT) in the model facultative aerated lagoon could be considered indeterminate because of the difficulties in estimating the following:

1. Biomass produced from BOD_5 degradation.
2. Quantity of solids that settle on the lagoon bottom and the rate of deposition.
3. The rate at which the settled solids are removed in the lagoon bottom through biodegradation.
4. Determination of mixed liquor total or volatile suspended solids in the facultative aerated lagoon because of the presence of the sludge layer.

The first obstacle arose from the fact that it is not easy to determine the actual amount of organic matter removed by means of biooxidation because of the difficulty in quantifying substrate benthal release from the settled sludge in the lagoon (Sackellares *et al.*, 1987).

To overcome these obstacles an approximate method was contrived in this research to determine solids retention time. For simplicity, a few assumptions were used for the derivation of the required equations.

6.6.1 Solids Retention Equations Development

In addition to influent suspended solids, the oxidation of BOD_5 requires synthesis of bacteria, which produces biological suspended material (NCASI, 1971). The equilibrium concentration of mixed liquor volatile suspended solids (MLVSS) is used as a common denominator of active biomass in comparing sludge production to BOD_5 removal. Average concentration of the VSS in the reactor, which is comprised of the influent solids and new solids produced due to substrate utilization, is determined by writing the mass balance of biomass around the system described in Figure 6.37:

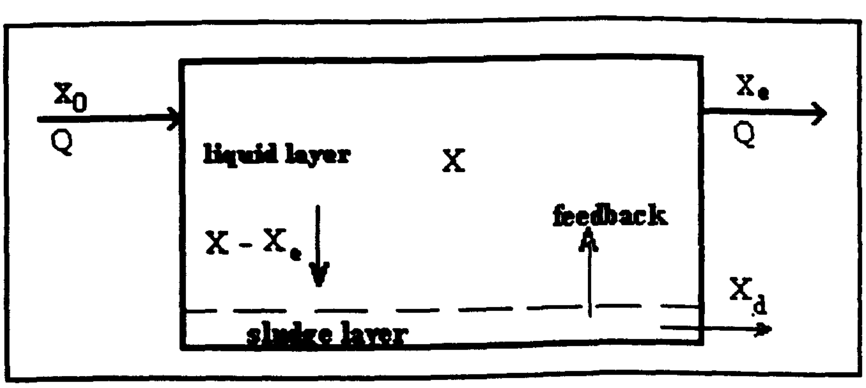


Figure 6.37 Schematic of a facultative aerated lagoon

$$\frac{dX}{dt}=\frac{Q}{V}X_0-\frac{Q}{V}X_e+(\frac{YS_r}{V/Q}-K_dX)-\frac{dX_s}{dt} \tag{6.40}$$

where, dX/dt = rate of change in microorganisms; mg VSS/l.d,

X = average volatile suspended solids in the system; mg/l,

V = volume of reactor; litre,

Q = wastewater flowrate; litre/d,

X_0 = influent volatile suspended solids; mg/l,

X_e = effluent volatile suspended solids; mg/l,

Y = growth yield; mg VSS produced/mg BOD_5 removed,

S_r = substrate utilized for microorganisms growth; mg BOD_5/l,

K_d = specific decay rate; d^{-1}, and

dX_s/dt = rate of degradation of the settled solids; mg VSS/l.d.

At steady state conditions dX/dt equals zero and dX_s/dt will be written as:

$$dX_s/dt = X_d/t_d \tag{6.41}$$

where, X_d = concentration of the degraded solids; mg/l, and

t_d = time taken for X_d to be removed; d.

and Equation 6.40 reduces to:

$$\frac{Q}{V}(X_0-X_e)-\frac{X_d}{t_d}=K_dX-\frac{YS_r}{(V/Q)} \tag{6.42}$$

Rearranging and solving for X:

$$X=\frac{\dfrac{X_0-X_e+YS_r}{V/Q}-\dfrac{X_d}{t_d}}{K_d} \tag{6.43}$$

The concentration of the volatile fraction of the settled deposits of sludge is described according to the following equation:

$$\text{Settled solids} = X-X_e \tag{6.44}$$

The concentration of the part of the settled solids that will be removed daily by degradation in the bottom layer is estimated according the following equation:

$$\frac{X_d}{t_d} = \frac{(X - X_e) \times D}{365} \qquad (6.45)$$

where the coefficient D represents the annual sludge degradation ratio.

The magnitude of BOD_5 released from the settled solids will be diluted in the liquid layer. In calculating the benthal feedback from the settled solids, the volume of the liquid layer is approximated by the total volume of the lagoon. Thus, the soluble BOD_5 feedback from these settled volatile suspended solids would be:

$$\text{Benthal feedback (mg)} = \frac{F}{V}(X - X_e) \qquad (6.46)$$

where, F = Feedback coefficient; g BOD_5/g VSS settled in the lagoon.

The actual substrate removed, which is utilized for microorganisms production, equals the apparent substrate removed plus the sludge feedback fraction and this can be described by the following equation:

$$\text{Utilized substrate for biomass production} = S_a + \frac{F}{V}(X - X_e) \qquad (6.47)$$

where, S_a = Apparent BOD_5 removed (unfiltered influent BOD_5 minus filtered effluent BOD_5).

Substituting Equations 6.45 and 6.47 into Equation 6.43 yield:

$$X = \frac{\dfrac{X_0 - X_e + Y[S_a + \frac{F}{V}(X - X_e)]}{V/Q} - \dfrac{(X - X_e)D}{365}}{K_d} \tag{6.48}$$

Solids retention time is calculated as the total amount of solids in the system divided by the rate wasted. The solids in the system are represented by the solids concentration (X) determined in Equation 6.48 multiplied by the volume of the lagoon. In systems with no sludge recycling, the wasted solids is represented by the solids escape in the effluent. The solids at the sludge layer that are removed from the system by degradation were already considered in Equation 6.48. Therefore the equation for the mean solids retention time (SRT) will be:

$$SRT = \frac{VX}{QX_e} \tag{6.49}$$

6.6.2 Estimating SRT Procedure

For a facultative aerated lagoon operated under steady state conditions, mean solids retention time (SRT) could be determined using the mathematical equations derived above. Kinetic coefficients (Y and K_d) for the wastewater should be determined prior to proceeding with the application of the method. The procedure of the proposed method is as follow:

a. Determine the mean volatile solids concentration (X) in the lagoon before subtracting that part degraded at the bottom using Equation 6.45. This step required the use of a trial and error process.
b. Estimate the SRT using Equation 6.48.

In the above method a feedback coefficient (F) of 0.4 mg BOD_5/mg VSS of settled volatile suspended solids, as quantified by Marais and Capri (1970) as quoted by Eckenfelder *et al.* (1972), could be used. Sludge annual degradation coefficient (D) of 0.4 at 20°C and 0.6 at 30°C, as postulated by Adams and Eckenfelder (1974), is also applicable in this method.

Solids retention times which occurred in this study were estimated for the different experimental runs using the proposed method shown above and are summarised in Table 6.19. The ratio SRT/HRT is also determined and presented in the table. These estimates were based on values of 0.53 mg VSS/mg BOD_5 for Y and $0.061 d^{-1}$ for K_d, which were determined for the settled sewage used in this research in a bench-scale batch reactor in the laboratory. As the table shows, both solids retention time and SRT/HRT ratio in facultative aerated lagoons are highly correlated with power level, organic loading and temperature. As both hydraulic retention time and temperature increased, SRT and SRT/HRT increased,. whereas increase in power level caused both the mean solids retention time and SRT/HRT ratio to decrease. In this research, for the model aerated lagoon operated at the different levels of power, organic loading (or hydraulic retention time) and temperature cited in an earlier chapter, SRT was observed to range from 138 days to as high as 1159 days. The corresponding values of SRT/HRT ratio were 42 to 92.

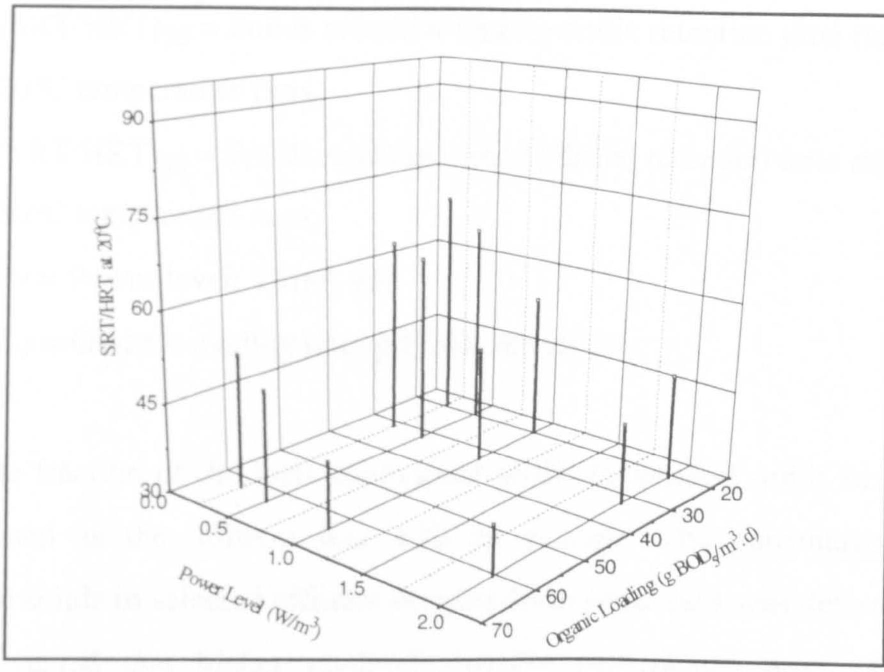
The relationships between SRT/HRT and both power level and organic loading are shown simultaneously in Figures 6.38a and 6.38b for 20°C and 30°C temperatures, respectively. The mathematical models which describe these relationships are shown in the following equations:

$$\mathrm{Ln}\ (\mathrm{SRT/HRT})_{20} = 4.209 \times P_V^{-0.043} - 0.149 \times e^{-0.019 \times L_V} \qquad R^2 = 0.966 \qquad (6.50)$$

$$\mathrm{Ln}\ (\mathrm{SRT/HRT})_{30} = 1.007 \times e^{-1.849 \times P_V} + 4.521 \times L_V^{-0.042} \qquad R^2 = 0.78 \qquad (6.51)$$

Table 6.19: Ratios of Solids Retention Times to Hydraulic Retention Times for Different Levels of the Operating Variables.

Power Level (W/m^3)	Temp. (°C)	Org. Load. g BOD_5 /m^3.d	HRT (days)	SRT (days)	SRT/HRT Ratio
0.25	20	19.18	9.8	683.8	69.8
0.5	20	19.4	10	649.4	64.9
1.0	20	20	10	543.3	54.4
2.0	20	20.91	11	524.3	47.7
0.25	20	31.86	5.9	377	63.9
0.5	20	32.33	6	373	62.2
1.0	20	33.33	6	294.8	49.1
2.0	20	32.86	7	303.1	43.3
0.25	20	62.67	3	157.2	52.4
0.5	20	63.61	3.05	148	48.5
1.0	20	63.9	3.13	128.1	40.9
2.0	20	60.53	3.8	144.8	38.1
0.25	30	20.73	12.3	1112.4	90.4
0.5	30	21.03	12.6	1158.9	92
1.0	30	20.96	10.16	543.2	53.5
2.0	30	21.08	10.39	627.6	60.4
0.25	30	32.28	7.9	675.5	85.5
0.5	30	33.13	8	720.5	90.1
1.0	30	33.7	6.32	327.2	51.8
2.0	30	34.82	6.29	326.8	52
0.25	30	60.71	4.2	353.3	84.1
0.5	30	60.23	4.4	347.5	79
1.0	30	64.55	3.3	138.5	42
2.0	30	64.79	3.38	161.6	47.8



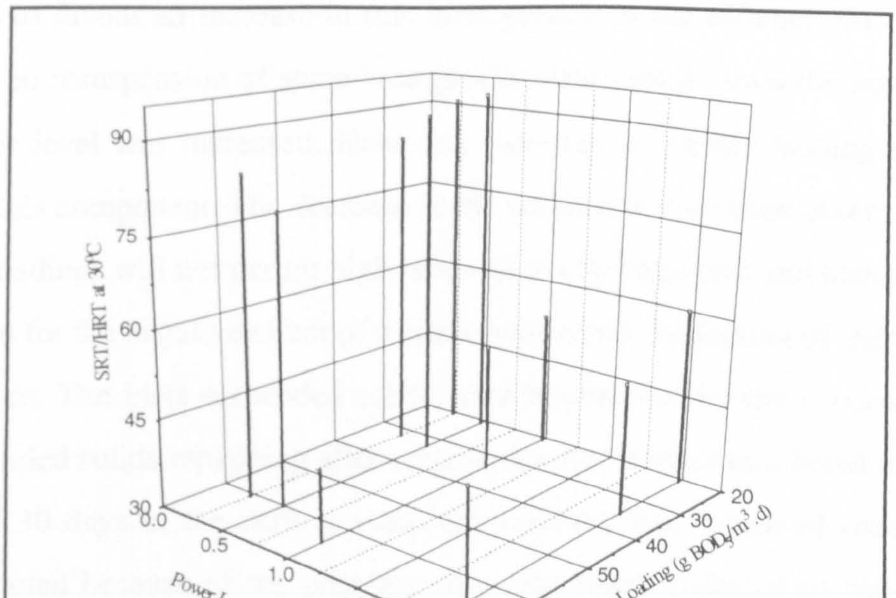


Figure 6.38 Variation of SRT/HRT with Power Level and Organic Loading

where, $(SRT/HRT)_{20}$ = Solids retention time:hydrulic retention time ratio for 20°C temperature runs,

$(SRT/HRT)_{30}$ = Solids retention time:hydrulic retention time ratio for 30°C temperature runs,

Pv = Power level; W/m^3, and

L_V = Organic loading rate; g BOD_5 /m^3.d.

The fraction of the inert component of the suspended solids in the settled sewage used as the influent was 0.23 on average. The percentages of inert suspended solids in selected effluent samples from some runs was determined. The results revealed that higher nonbiodegradable percentages were observed in effluent from the facultative aerated lagoon than in influent. Power level was observed to favour an increase in this inert portion in the effluent; this may have been due to resuspension of some biologically stable solids from the bottom when the power level was increased. However, increase in organic loading negatively affected this component. The decrease in the solids retention time observed at high organic loadings will not permit high levels of sludge digestion and hence could be the reason for the negative effect of organic loading on the faction of inert solids in the effluent. The inert suspended solids were represented by the concentration of the suspended solids remaining after aerating the wastewater in a batch mode for a period of 30 days in the dark at room temperature. The measured concentration was corrected because of the presence of accumulated endogenous residue. This can be described by the following equation:

Inert suspended solids = (remaining SS after 30 days aeration) - (0.23 Y S_0)

The term 0.23 represents the fraction of the biomass produced by the oxidation of the initial substrate (S_0), which was converted to inert material.

ASSESSMENT OF OTHER PERFORMANCE FACTORS OF FACULTATIVE AERATED LAGOONS

7.1 Nutrients Removal in Facultative Aerated Lagoons

7.2.

Nutrients are important constituents of wastewater if it is to be treated biologically, in order to ensure that biological oxidation occurs. The wastewater used in this research, settled sewage, contained abundant nutrients so addition of nitrogen and phosphorus supplements was unnecessary during the course of the experimental work. Substrate utilization rate was not limited by these two elements but their removal during the treatment process should be considered. Removal of nitrogen and phosphorus will reduce the potential of the effluent as a fertilizer if used for irrigation (Heddle, 1982) though it will reduce potential environmental effects (e.g., undesirable side effects of eutrophication) if the effluent is discharged into a receiving body of water. However, the primary function of aerated lagoons is the reduction of BOD (NCASI, 1971). Physico-chemical methods of phosphorus removal are more energy-efficient and cost-effective treatment processes than biological methods (Narasiah *et al.*, 1991), whereas for nitrogen removal Azimi and Horan (1991) stated that biological methods have proved most cost-effective.

7.1.1 Nitrogen Removal

Samples of influent and effluent were tested for ammonia nitrogen (NH_4-N) and total Kjeldahl nitrogen (TKN) during the course of the experimental work. Organic nitrogen was obtained by determining the difference between TKN and NH_4-N. Removal of ammonia nitrogen and TKN in this treatment method were studied under the different operating conditions.

i) Ammonia Nitrogen

The average concentration of ammonia nitrogen in the settled sewage used as influent wastewater to the model facultative aerated lagoons ranged from 29.1 mg/l to 45.3 mg/l for unfiltered samples, while the corresponding concentrations in filtered samples were 27.4 mg/l to 40.6 mg/l. A concentration as low as 18 mg/l of ammonia in the unfiltered effluent samples was achieved, the highest value of 38.7 mg/l being reached. For filtered effluent samples, the corresponding concentrations were 10.3 mg/l and 37.8 mg/l. The effluent quality, with respect to ammonia, from this treatment process was observed to be affected by power level and organic loading as well as liquid temperature in the lagoon. The higher the latter parameter the higher the ammonia nitrogen removal efficiency. Interestingly, at the lower temperature (20°C) some increase in ammonia concentration from influent to effluent at the lowest power level (0.25 W/m^3) was observed. This phenomenon may be due partly to the nitrogen release from bottom deposits which were high at low power levels, as the result of anaerobic degradation and the conversion of organic nitrogen to ammonia within bottom sludge pockets (Eckenfelder *et al.*, 1972; Lewandowski and Bradley, 1981) and endogenous respiration (NCASI, 1982b) of the nitrogen incorporated in cell mass. In the activated sludge process, increase in ammonia concentrations is a sign of insufficient treatment (Billings and Smallhorst, 1971).

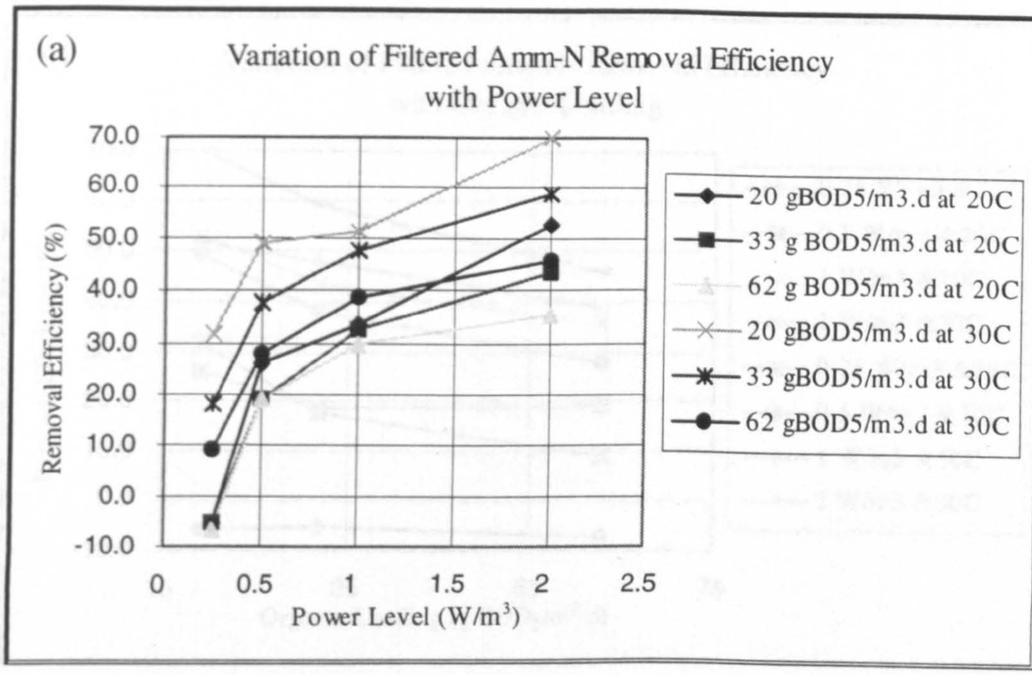
At the 20°C operating temperature, as power level increased, ammonia nitrogen removal efficiency also increased, reaching a maximum removal of 52.6% for filtered samples and 40.1% for unfiltered samples over the range of organic loading and power levels applied in this research. At 30°C, the increase in NH_4-N concentration from influent to effluent was not observed as in the case at 20°C under certain operating conditions. The highest and lowest ammonia nitrogen removal efficiencies at this temperature (30°C) were 70% and 9.4% for filtered samples and 49.6% and 7.9% for unfiltered samples, respectively.

Ammonia nitrogen removal efficiency variations with power level and organic loading are depicted in Figures 7.1 and 7.2. For both temperatures (20°C and 30°C) ammonia removal efficiencies were noticed to be affected positively by power level and negatively by organic loading. For filtered samples, a higher rate of increase was observed as power level increased between 0.25 W/m^3 and 1 W/m^3 than when it was increased from 1 W/m^3 to 2 W/m^3. A decrease was observed in the NH_4-N removal efficiency for unfiltered samples during the runs operated at 30 °C when the power exceeded 0.5 W/m^3. This behaviour was not noticed at 20°C runs. Under similar operating conditions, a positive effect of temperature on NH_4-N removal was experienced.

ii) Total Kjeldahl Nitrogen

Total Kjeldahl nitrogen (TKN) variations under all operating conditions (power level, organic loading and temperature) were also observed in the model facultative aerated lagoons. TKN in the influent wastewater varied from run to run and mean concentrations ranged from 38.4 mg/l to 50.7 mg/l and from 49.3 mg/l to 60.1 mg/l for filtered and unfiltered samples, respectively. The corresponding effluent TKN concentrations were 20 mg/l to 42.1 mg/l and 24.3 mg/l to 47.4 mg/l, respectively.

Because of variations in the influent TKN, comparison of TKN removal efficiencies is used to determine the effect of operating variables. Figures 7.3 and 7.4 show the variations of filtered TKN and total TKN with power level and organic loading at liquid temperature levels of 20°C and 30°C. From these figures



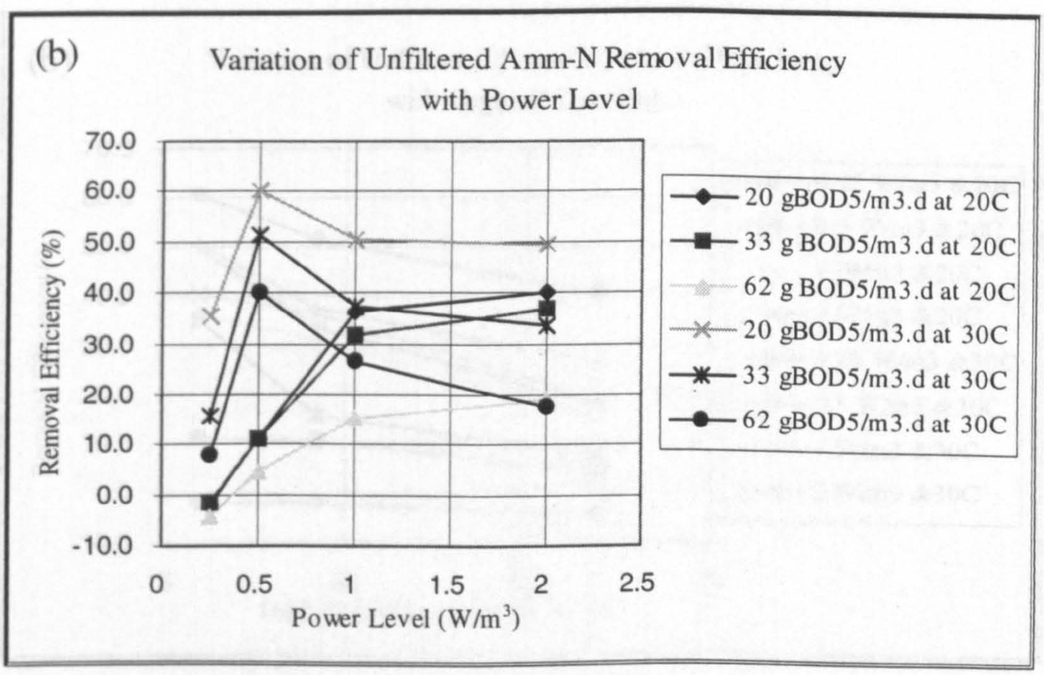


Figure 7.1 Variation of Ammonia Nitrogen Removal Efficiency with Power Level.

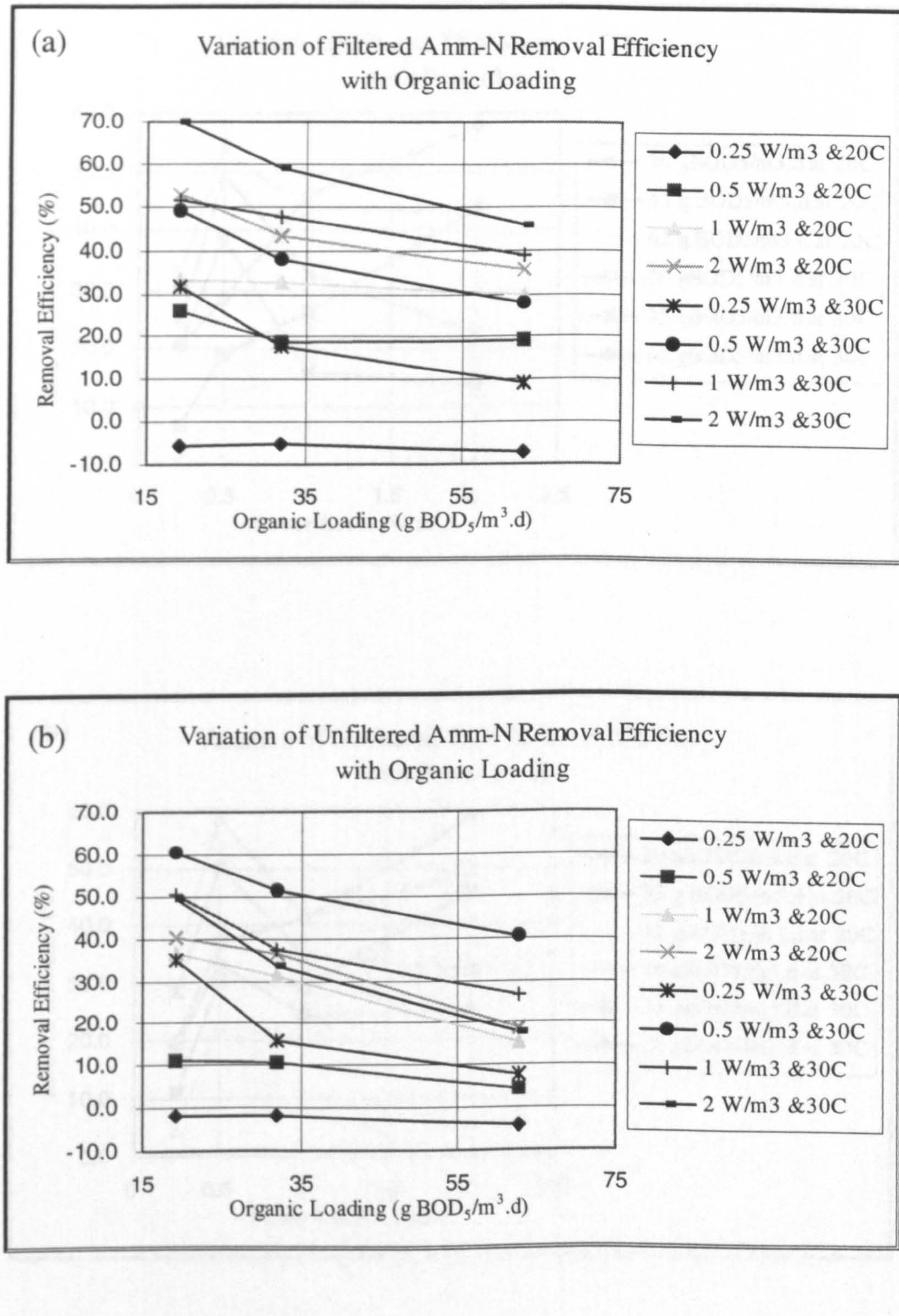


Figure 7.2 Variation of Ammonia Nitrogen Removal Efficiency with Organic Loading.

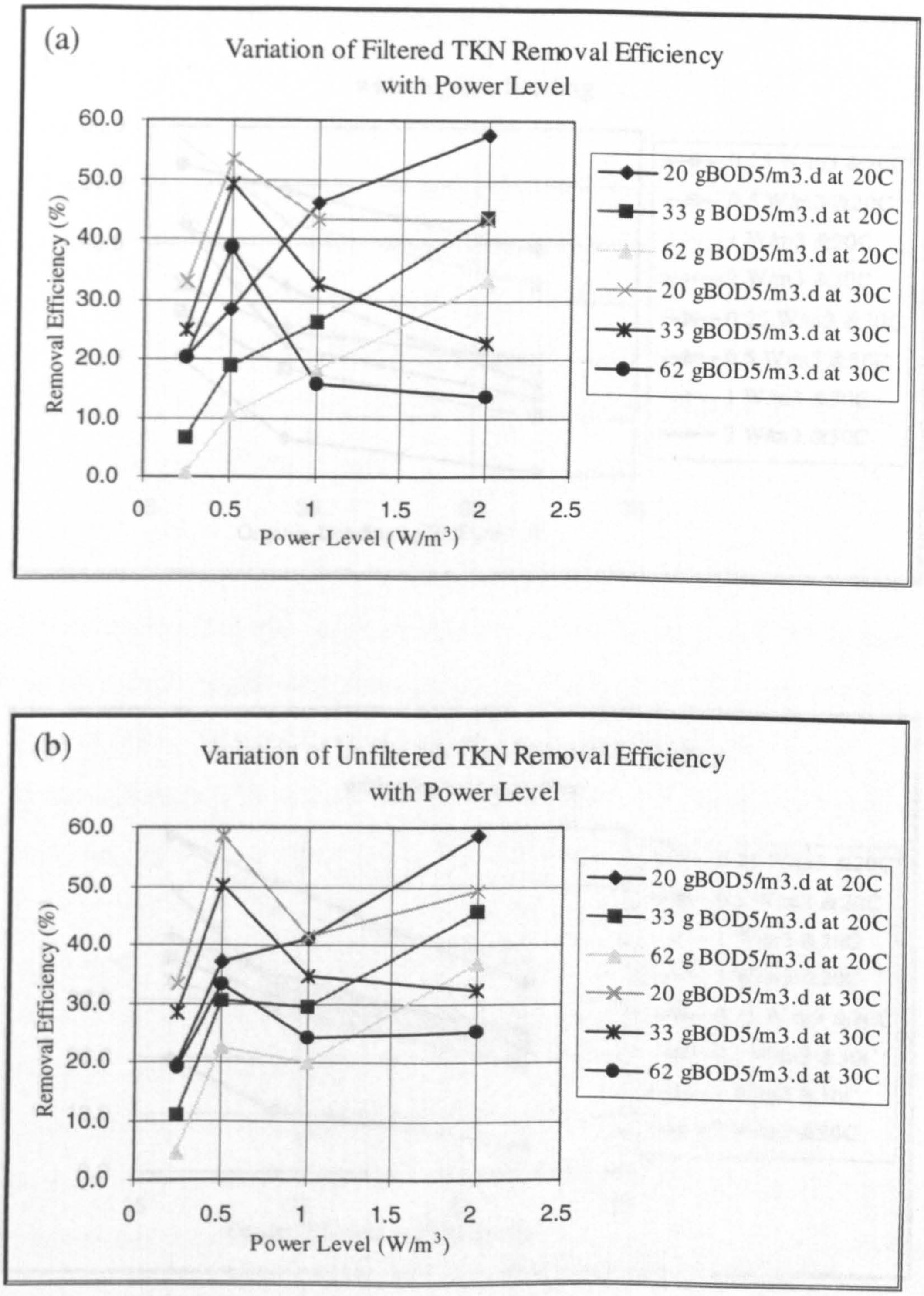
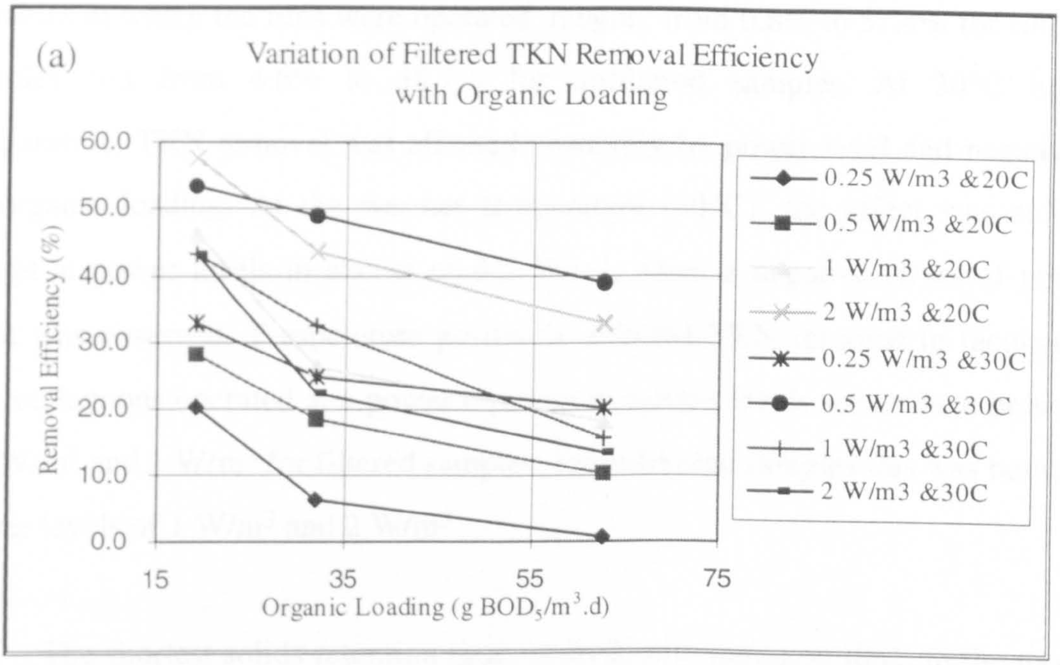


Figure 7.3 Variation of Total Kjeldahl Nitrogen Removal Efficiency with Power Level.



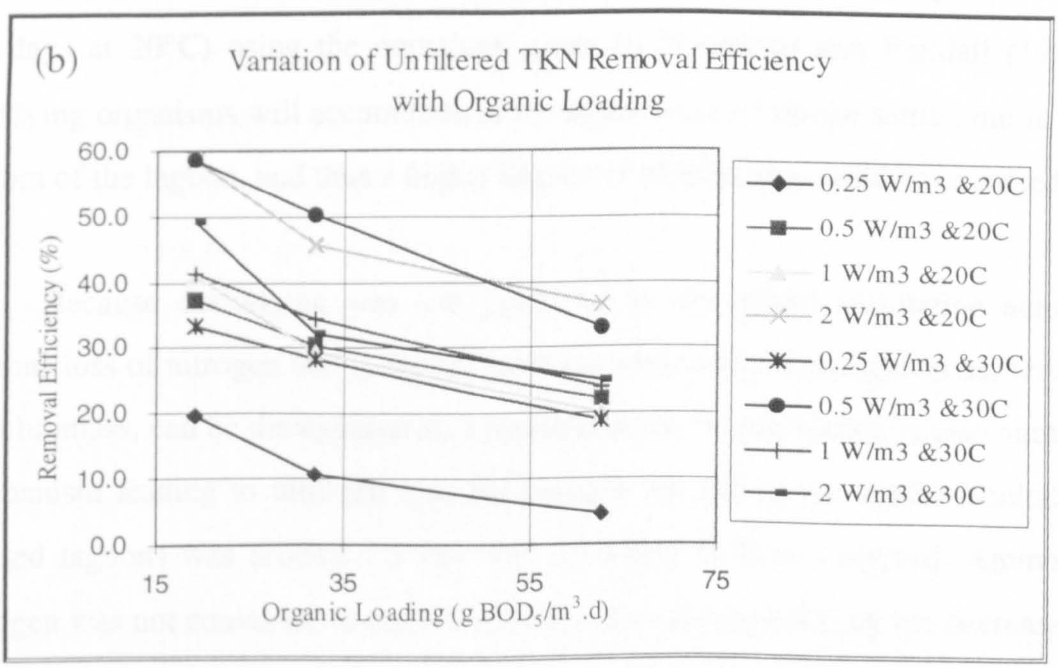


Figure 7.4 Variation of Total Kjeldahl Nitrogen Removal Efficiency with Organic Loading.

it can be inferred that TKN removal varied greatly, depending on the levels of the variables at which the runs were operated, ranging from 0.8% to 57.4% for filtered samples and from 4.6% to 58.7% for unfiltered samples. At 20°C liquid temperature, TKN removal was affected positively by power level and negatively by organic loading. At the warmer temperature (30°C), the effect was similar except at power levels in excess of 0.5 W/m^3, when a negative effect of power input was observed. Temperature positively affected TKN removal in facultative aerated lagoons operated at a power input up to certain levels somewhere between 0.5 W/m^3 and 1 W/m^3 for filtered samples; for unfiltered samples this was between power levels of 1 W/m^3 and 2 W/m^3.

The shortest solids retention time, or hydraulic retention time, in the model facultative aerated lagoons was more than the minimum estimated solids retention time below which nitrification will not occur (1.3 days at 30°C temperature and 2.8 days at 20°C) using the equations given by Benefield and Randall (1980). Nitrifying organisms will accumulate in the upper layer of sludge settled out in the bottom of the lagoon, and thus a higher degree of nitrification could be expected.

Because desludging was not practiced in the model facultative aerated lagoons loss of nitrogen due to sludge wasting, within which nitrogen is associated with biomass, can be disregarded as a possible cause. Volatilisation is also another mechanism leading to nitrogen loss but because the pH in the model facultative aerated lagoons was around 7.3 this was not likely to have occurred. Ammonia nitrogen was not converted to organic nitrogen; this is supported by the decrease in TKN. The assimilatory uptake of nitrogen by bacteria and by algae when it is present (at low power levels), followed by settling may have occurred as a step before denitrfication took place in the bottom deposits. Nitrification is expected only at higher power levels at which ammonia nitrogen is converted to nitrite and thence to nitrate. This is inconsistent with Arceivala (1981) who claimed that no nitrification occurs in facultative aerated lagoons.

Thus it could be concluded that the postulated major cause of appreciable losses of nitrogen is the nitrification/denitrification mechanism and at low power levels is the uptake by biomass. The two processes (nitrification/denitrification) occur concomitantly in the presence of oxygen in the bulk liquid and in its absence at bacterial sites in the sludge layer.

7.1.2 Phosphorus Removal

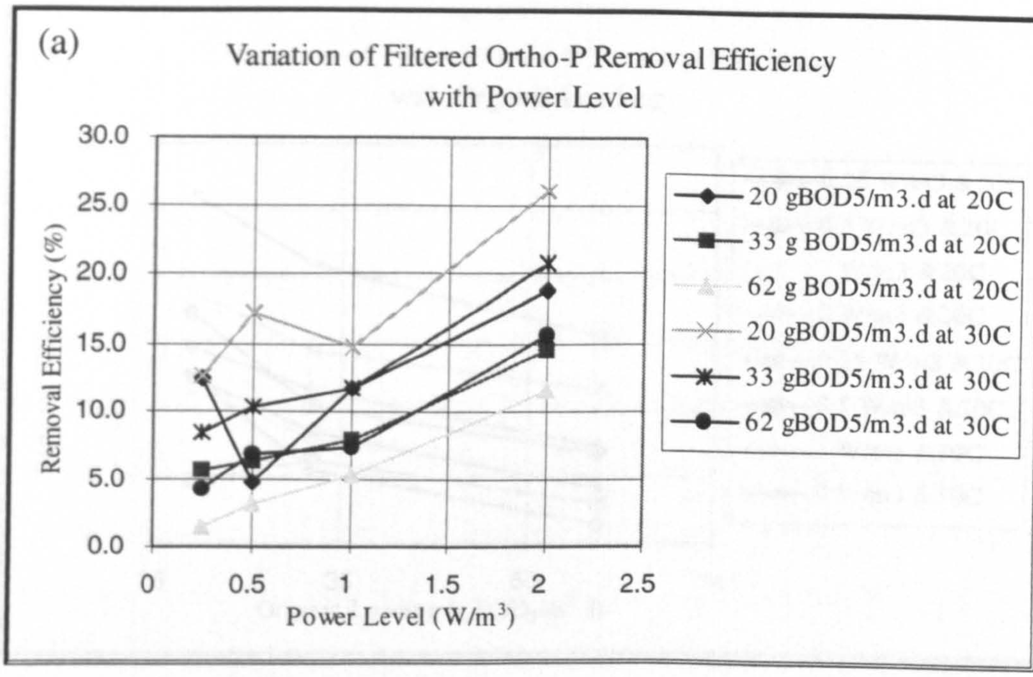
Orthophosphate, which is the soluble form of phosphorus, was measured for both influent and effluent samples. Orthophosphates are readily assimilated by bacteria but other forms of phosphates must first undergo enzymatic hydrolysis to orthophosphates before they can be assimilated. The average orthophosphate concentration in the influent wastewater was 7.2 mg/l to 9.5 mg/l and 7.9 mg/l to 9.8 mg/l for filtered and unfiltered samples, respectively. Over the range of variable parameter values investigated the corresponding effluent concentrations were 5.7 mg/l to 9.1 mg/l and 6 mg/l to 9.6 mg/l, respectively. Filterable orthophosphate removal efficiencies ranged from 1.4% to 18.8% during runs at a temperature of 20°C while for runs operated at 30°C removal was in excess of 4.2% and ranged as high as 26%. Corresponding orthophosphate removal for unfiltered samples was 2.9% to 13.5% and 2% to 25.9%, respectively. Generally, as temperature increased orthophosphate removal efficiency increased, except for unfiltered samples at very low power level (0.25 W/m^3) in which case removal at 20°C was higher than at 30°C. The increase in filterable orthophosphate removal with temperature in the model facultative aerated lagoons was unlike that observed in aerobic (completely mixed) aerated lagoons as reported by Narasiah *et al.*

(1987). However, the positive effect of temperature at high power level input is in agreement with that reported by Narasiah (1983) who observed higher phosphate removal in summer than in winter in completely mixed aerated lagoon. This similarity is most probably because at high power levels the facultative aerated lagoon resembles an aerobic aerated lagoon.

Orthophosphate removal efficiency variations were affected by the power level and organic loading at which the runs were operated. These variations are shown graphically in Figures 7.5 and 7.6 indicating, aside from some operational conditions, that as a general trend there is a positive effect of power level and a negative effect of organic loading. Neither uptake by algae nor the precipitation of hydroxyapatite ($Ca_5(PO_4)_3OH$) were causes of the appreciable losses in phosphate. The former was because algal concentration was not significant in the model facultative aerated lagoons, whereas the latter was because pH in the model facultative aerated lagoons was not high. According to the present study, phosphorus removals were considerable at high power levels, whereas in aerobic (completely mixed) aerated lagoons some researchers (Loehr and Schulte, 1970) assert that the bacterial synthesis of phosphates is very poor.

7.2 Pathogen Removal in Facultative Aerated Lagoons

The numbers of faecal coliforms are used to indicate the bacteriological quality of the final effluent and efficiency of the removal process. Samples of influent to and effluent from the model facultative aerated lagoons were tested for coliform bacteria as typified by *Escherichia. coli (E. coli)* and *faecal streptococci*. Four tests on samples of both influent and effluent were tested during each run and the mean values used in this assessment. Two analyses were carried out on composite samples collected throughout a 24-hour time period.



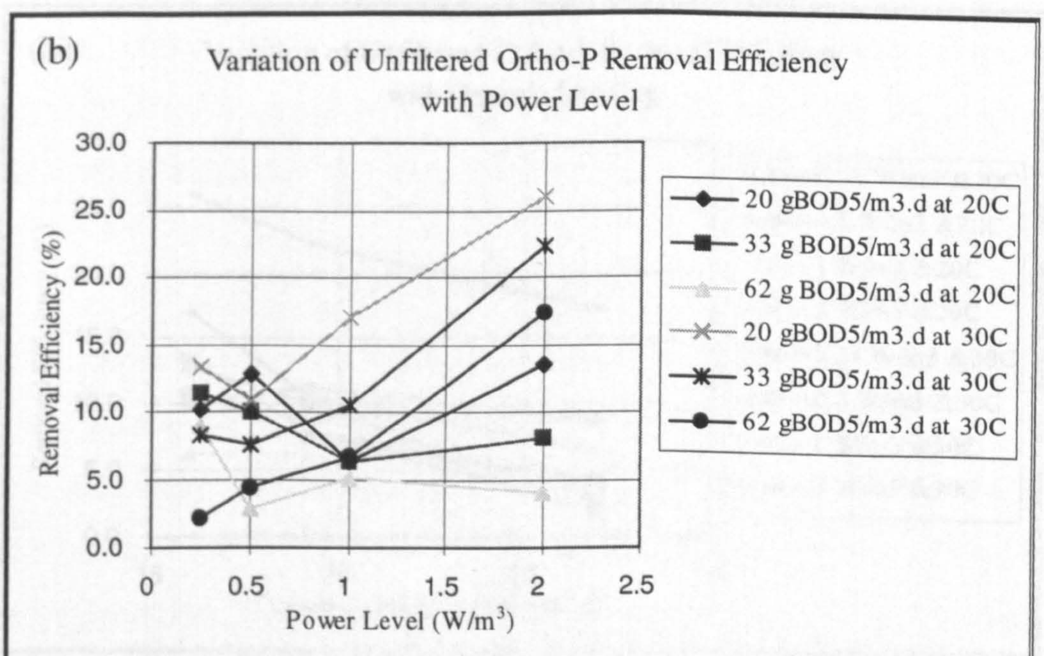
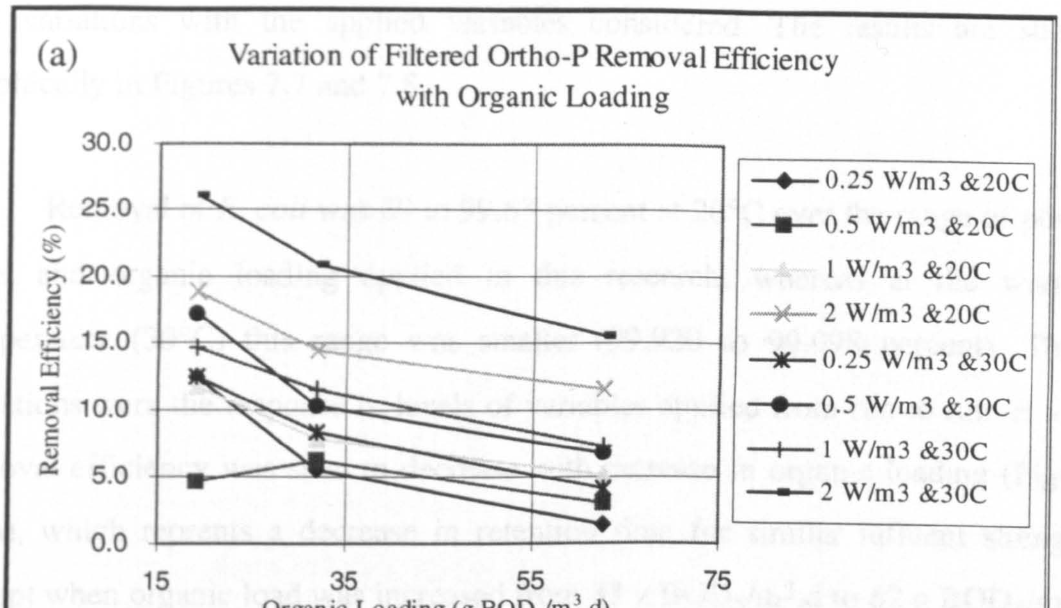


Figure 7.5 Variation of Phosphorus Removal Efficiency with Power Level.



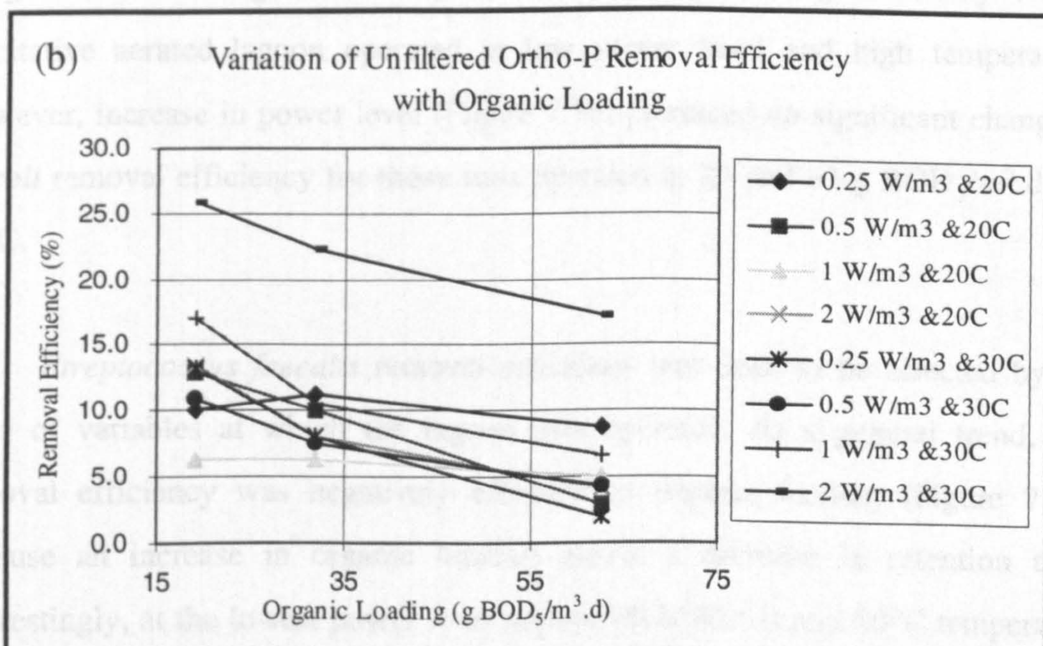
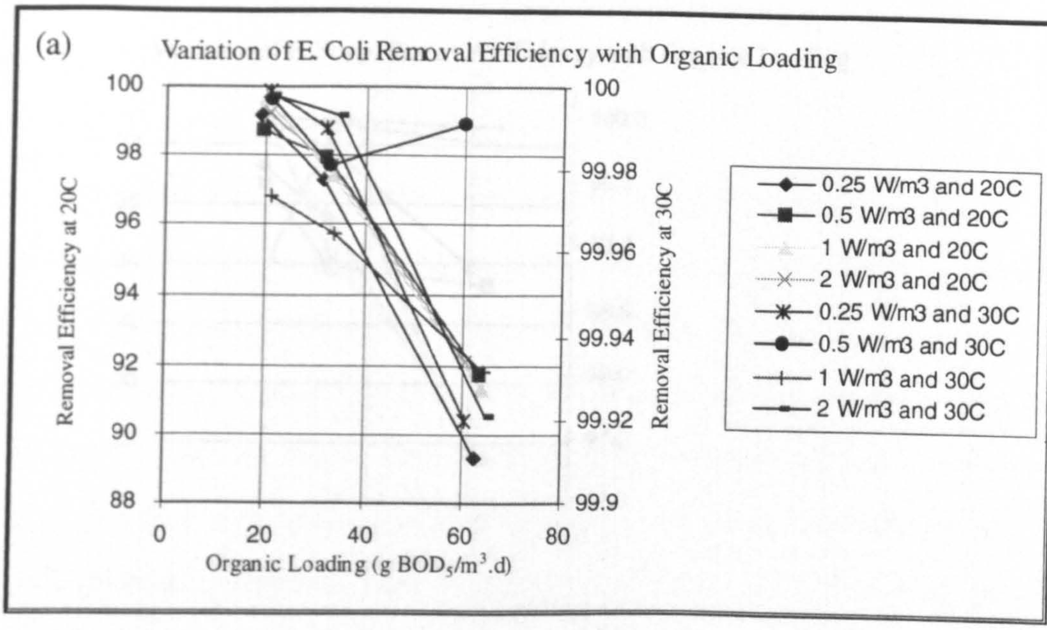


Figure 7.6 Variation of Phosphorus Removal Efficiency with Organic Loading.

The removal efficiencies of these two pathogen organisms were determined and variations with the applied variables considered. The results are shown graphically in Figures 7.7 and 7.8.

Removal of *E. coli* was 89 to 99.63 percent at 20°C over the range of power level and organic loading applied in this research, whereas at the warmer temperature (30°C) this range was smaller (99.920 to 99.998 percent). These variations were the response to levels of variables applied from run to run. *E. coli* removal efficiency was seen to decrease with increase in organic loading (Figure 7.7a), which reprents a decrease in retention time for similar influent strength, except when organic load was increased from 33 g BOD_5/m^3.d to 62 g BOD_5/m^3.d at 0.5 W/m^3 power level and 30°C temperature. This might suggests that facultative aerated lagoons are efficient in removing *E. coli* at 0.5 W/m^3, 30°C and 62 g BOD_5/m^3.d organic loading, a condition representing a heavily loaded facultative aerated lagoon operated at low power level and high temperature. However, increase in power level (Figure 7.7b) produced no significant change in *E. coli* removal efficiency for those runs operated at 20 and 33 g BOD_5/m^3.d and 20°C.

Streptococcus faecalis removal efficiency was seen to be affected by the level of variables at which the lagoon was operated. As a general trend, this removal efficiency was negatively affected by organic loading (Figure 7.8a), because an increase in organic loading means a decrease in retention time. Interestingly, at the lowest power level applied (0.25W/m^3) and 30°C temperature the increase in organic loading from 20 to 33 g BOD_5/m^3.d, which is a decrease in retention time from 9.8 to 5.9 days, was accompanied by an increase in *faecal streptococci* removal efficiency. This suggests that facultative aerated lagoons are also efficient in removing *Streptococcus faecal* at o.25 W/m^3, 30°C and 33 g BOD_5/m^3.d. This represents a medium organic loaded facultative aerated lagoon



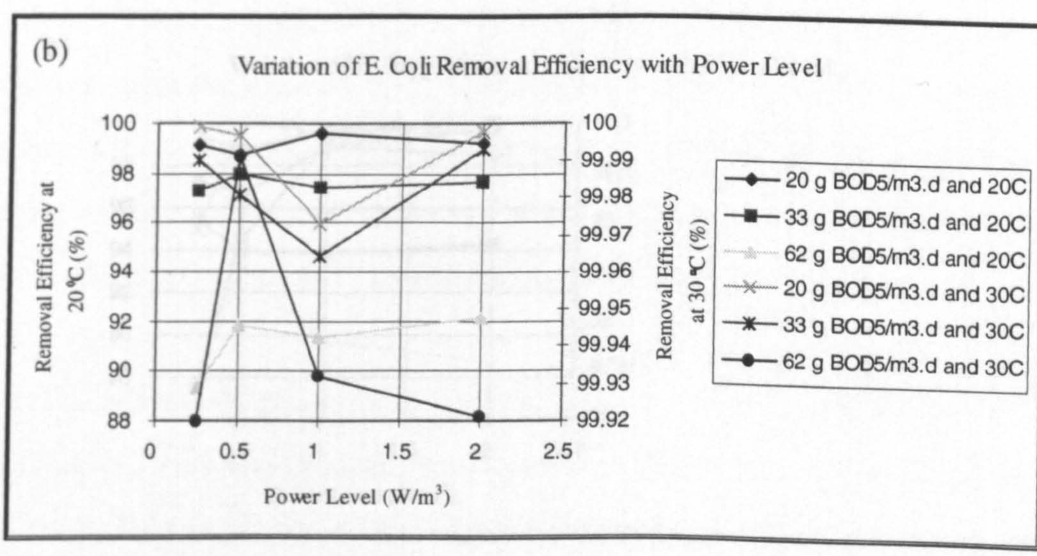
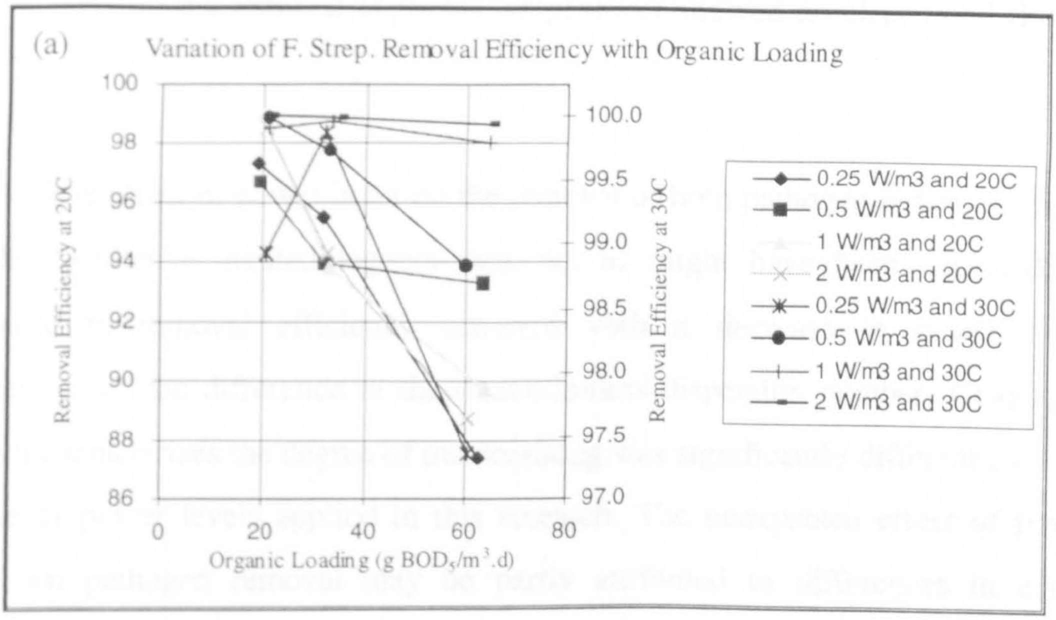


Figure 7.7 Variation of *Escherichia Coli* Removal Efficiency with Organic Loading (a) and Power Level (b).



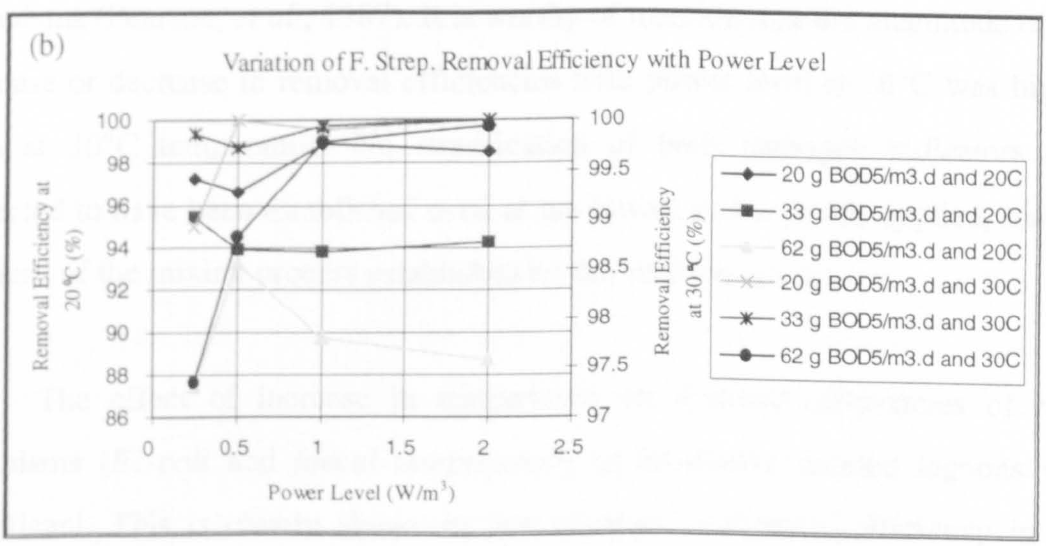


Figure 7.8 Variation of *Faecal Streptococci* Removal Efficiency with Organic Loading (a) and Power Level (b).

operated at low power level and high temperature. As for *E. coli* , the effect of power level on the removal of *faecal streptococci* showed no clear trend (Figure 7.8b).

The effect of power input on the removal of both pathogen indicators in the model facultative aerated lagoons was not as might have been expected; an increase in removal efficiency occurred with a decrease in power level. Nevertheless, the difference in the dimensionless dispersion number (Chapter 5) which characterizes the degree of macromixing was significantly different over the range of power levels applied in this research. The unexpected effect of power level on pathogen removal may be partly attributed to differences in actual hydraulic retention times for runs operated at similar organic loading rates and temperatures or to the positive effect of dissolved oxygen on the removal of these organisms (Pearson, *et al.*, 1987). It is worthy of mention that the magnitude of the increase or decrease in removal efficiencies with power level at 20°C was higher than at 30°C temperature. No stratification of both pathogen indicators was expected to have been established even at the lowest power levels applied, mainly because of the mixing process established by the aeration system.

The effect of increase in temperature on removal efficiencies of both organisms (*E. coli* and *faecal streptococci*) in facultative aerated lagoons was significant. This is clearly shown by the increase in removal efficiency in the model lagoons when temperature was increased from 20°C to 30°C. Increase in removal efficiency with increase in temperature may possibly be due to acceleration of the die-off as a result of increased metabolic activity and greater susceptibility to toxicants as well as accelerated substrate utilization and the consequent starvation conditions (Pearson *et al.*, 1987).

Mean concentrations of *E. coli* and *faecal streptococci* (per 100 ml) in the influent wastewater throughout the period of the experimental work were 2.74×10^7 and 3.19×10^6, respectively, with standard deviations of 1.63×10^7 and 2.33×10^6. Residual counts of *E. coli* in effluents were 1.07×10^6 (with standard deviation of 1.26×10^6) and 6.9×10^3 (S.D. of 8.49×10^3) at 20°C and 30°C temperatures, respectively. The corresponding *Streptococcus faecalis* densities in the effluent were 2.53×10^5 (S.D. of 1.88×10^5) and 1.25×10^4 (S.D. of 2.2×10^4).

Sedimentation at low power levels (in the form of flocs) and turbidity (suspended solids) at higher power levels are expected factors that can improve or deteriorate, respectively, pathogen removal efficiency. The latter factor would reduce light penetration into the facultative aerated lagoon. The higher retention times at low organic loadings than at higher loadings are likely to be the main cause of the negative effect of organic loading on pathogen removal efficiency. Several environmental conditions and factors would affect the survival and viability of pathogens in any system including solar radiation, predation, bacteriophages, nutrient deficiency, algal and bacterial toxins, and physico-chemical factors (Gray, 1989). Mixed liquor pH values were not high (7-7.5) hence the commonly held view that the removal of pathogens in some treatment processes is caused by high pH values was not applicable for facultative aerated lagoons as supported by this study.

The findings of this research would suggest that in warm climates this type of treatment process is effective in reducing bacterial pathogens. Facultative aerated lagoons stand intermediately between aerated lagoons and facultative stabilization ponds. However, effluent from the model facultative aerated lagoon shows that further microorganisms destruction should be provided to meet the bacteriological quality standards required for different forms of effluent reuse and disposal. This can be achieved by either increasing retention time or by further treatment. Maturation ponds or disinfection can be used to reduce pathogen numbers in the effluent from facultative aerated lagoons. However, this is also the same case for complately mixed aerated lagoons for which Mara (1976) pointed out that they are not particularly effective in removing faecal coliforms and further treatment may be necessary. Peat moss filters (Narasiah and Hains, 1988), slow sand filters (Horan, 1990) and microfiltration with a dynamic membrane (Al-Malack, 1993) are examples of efficient tertiary treatment processes for additional removal of these pathogens.

7.3 Accumulated Sludge Quality

In draining the lagoons at the end of the experimental work, two months after the last run operated at fixed levels of power, organic loading and temperature, a distinct layer of sludge was observed in the facultative aerated lagoons. Horan (1990) indicated that the agglomeration of flocs would be relatively heavy and would settle at a suitable power level. The low power levels at which these facultative aerated lagoons were operated led to the deposition of some settleable particulate solids to the bottom of the lagoon. These settled solids formed deposits in which the biodegradable materials would be decomposed if conditions permit. Benthal feedback is an extra load which is often omitted in simple analysis of aerated lagoons (Bryant, 1993). Feedback from deposited solids results from liquefaction and, if conditions are favourable, these solids will undergo gasification converting organic carbon to methane and carbon dioxide. Dissolved oxygen consumption by the chemical oxidation of reduced inorganic compounds released from the bottom is another factor to be considered in the design and analysis of aerated lagoons.

Analysis of sludge samples showed that the organic portion of the solids was in the range 64% to 84%. Inert solids are represented by the inorganic solids as well as part of the organic solids. The higher organic ratio is associated with high organic loading; this is mainly because of a decrease in the mean solids retention time. With increase in solids retention time, sludge stabilization will be higher. Supriyanto *et al.* (1988) cited the Primary Production Department Government of Singapore (1982), who postulated that total volatile solids to total solids ratio of 70 percent represent the standard below which the sludge will be considered stabilized. Thus from the organic percentage in the sludge removed from the model aerated lagoons it can be said that the sludge in the facultative aerated lagoon seems to be stabilized or nearly stabilized, depending on the organic loading. The low power levels at which these lagoons were operated permited the settleable fraction of suspended solids to settle and form deposits in which the biodegradable fraction aerobically and anaerobically decomposed and stabilized. Hurwitz (1963) observed that the inorganic solids and stabilized sludge in a full scale aerated lagoon installed with diffused aeration system settled out in the slower circulating mid-cells.

The sludge at the bottom of the lagoon consists of settleable inorganic solids (e.g. grit), settleable inert organics and inert residue from endogenous degradation of biological solids and the remainder is water. The wet volume of sludge in the model facultative aerated lagoons have a moisture content of 96% on average (Balasha and Sperber, 1975; Supriyanto *et al.*, 1988). With increase in accumulated solids in the bottom of the lagoon, the water volume will decrease and, as a consequence, power level will increase. When the threshold level is reached, some of the settleable solids will remain in suspension. After several years of operation, sludge depth in facultative aerated lagoons operated at power levels higher than the threshold level will not be affected by power level. For the model facultative aerated lagoons studied, it can be said that different sludge depths might be expected at power levels higher than the threshold level located between 0.5 W/m^3 and 1 W/m^3 (Chapter 6). Therefore, as power level increases sludge depth decreases. The Engineering News Record (1965) pointed out that the sludge build-up in a 5 ft aerated oxidation pond treating normal domestic wastewater is negligible because of the continuous aeration process and anaerobic digestion of the settled sludge. Organic loading affects sludge depth only for the first period after commencing operation of a facultative aerated lagoon when the steady state with respect to solids is not yet reached and sludge depth will increase with time till it reaches steady state conditions.

To summarise, the depth of sludge in a facultative aerated lagoon will depend mainly upon power level, provided a steady state condition has been reached. The accumulated sludge in facultative aerated lagoons can be expected to be highly stable and ready for land disposal, thus avoiding the expense of anaerobic sludge digestion or sludge drying beds required for stabilization of sludges from completely mixed aerated lagoons.

7.4 Algae in Facultative Aerated Lagoons

Light, carbon dioxide, nitrogen and phosphorus are the main factors that limit the growth of algae in facultative aerated lagoons (Rich, 1982b). Other physical factors, such as retention time, turbidity and temperature, and biological factors, principally grazing by herbivores, can control the growth of algae (Water Pollution Research Laboratory, 1973). In this research the wastewater used was domestic wastewater, therefore only light and carbon dioxide could be the controlling factors in the growth of algae. In facultative aerated lagoons, the latter (carbon dioxide) results from aerobic oxidation and from the benthal deposits and endogenous respiration of suspended hetrotrophic organisms. Light intensity at the surface of the model facultative aerated lagoon was approximately 6250 lux, as was estimated elsewhere (Almasi, 1994) under similar conditions. Predominant species of algae in the model facultative aerated lagoons were identified in the samples collected during each of the runs when algae were present.

Algal biomass in the model facultative aerated lagoon was assessed by determining the chlorophyll "a" concentration as an algal biomass indicator. This is because all photosynthetic components of algae contain chlorophyll "a" (Boney, 1989). Chlorophyll "a" determination (by spectrophotometric method) was used, on a relative basis, for algal biomass concentration because it is more accurate than the cell count method. The reasons for the low accuracy of the latter method are given elsewhere (Boney, 1989; APHA, 1989). APHA (1989) assumed that 1.5% of the dry weight of the organic matter of algae is constituted of chlorophyll "a". Based on this assumption, algal biomass can be estimated by multiplying the chlorophyll "a" concentration by a factor of 67. The method utilized in this study for determining chlorophyll "a" content is based on the extraction of the photopigments with aqueous acetone followed by measurement of the optical density of the extract.

Algae were almost absent in the model facultative aerated lagoons except at low organic loadings and low power levels, except at the corners (edges) where traces were detected at all times. At 20 g $BOD_5/m^3.d$ organic loading and 0.25 W/m^3 power level the algal biomass in the effluent was measured in terms of chlorophyll "a" and found to be 29 μg/l on average. The predominant algal genera were *Euglena, Oscillatoria* and *Chlamydomonas* at this organic loading and power level. At the same loading but with higher power level (0.5 W/m^3) this value decreased to 25 μg/l. Higher power level (1 and 2 W/m^3) operational runs produced a lower concentration of chlorophyll "a" (12 μg/l and 9 μg/l) in the model facultative aerated lagoon. With an increase in organic loading to 33 g

BOD_5/m^3.d *Euglena* and *Oscillatoria* were predominant. Chlorophyll "a" was 28 µg/l at this organic loading and a power level of 0.25 W/m^3. Considerable decrease in chlorophyll "a" concentration, to 7 µg/l, was observed when the power level was increased to 0.5 W/m^3. Only 4 µg/l and 3 µg/l of Chlorophyll "a" concentration were found when this power level was increased to 1 and 2 W/m^3, respectively. At 62 g BOD_5/m^3.d organic loading the chlorophyll "a" was negligible or almost zero, indicating that there was essentially no algae present in the model facultative aerated lagoon. Chlorophyll "a" concentrations at 20°C were slightly lower than those at the warmer temperature (30°C). No significant difference was observed between the pH values when the lights were on or during darkness. This indicates that either no algae were present or that there was very low algal concentration or activity. The relationship between the power level and chlorophyll "a" concentration in the effluent from the model facultative aerated lagoons is shown graphically in Figure 7.9. The charp decrease in the chlorophyll "a" concentration can be noticed as the power level increases from 0.25 to 0.5 W/m^3.

Chlorophyll "a", as a measure of algae biomass, was generally low in the model facultative aerated lagoon, especially at high power levels. It was negatively affected by power level and organic loading. Algae will disappear and be replaced by mixed heterotrophic bacteria if a pond is aerated artificially (Horan, 1990). Organic loading is one of the factors upon which the presence of a particular algal group depend.

Increase in power level increases the level of turbulence and hence the turbidity concentration, which reduces the depth of light penetration and the resultant algal growth. A high loading means low retention time, short time for growth and generation of algae, as well as more turbidity and consequent

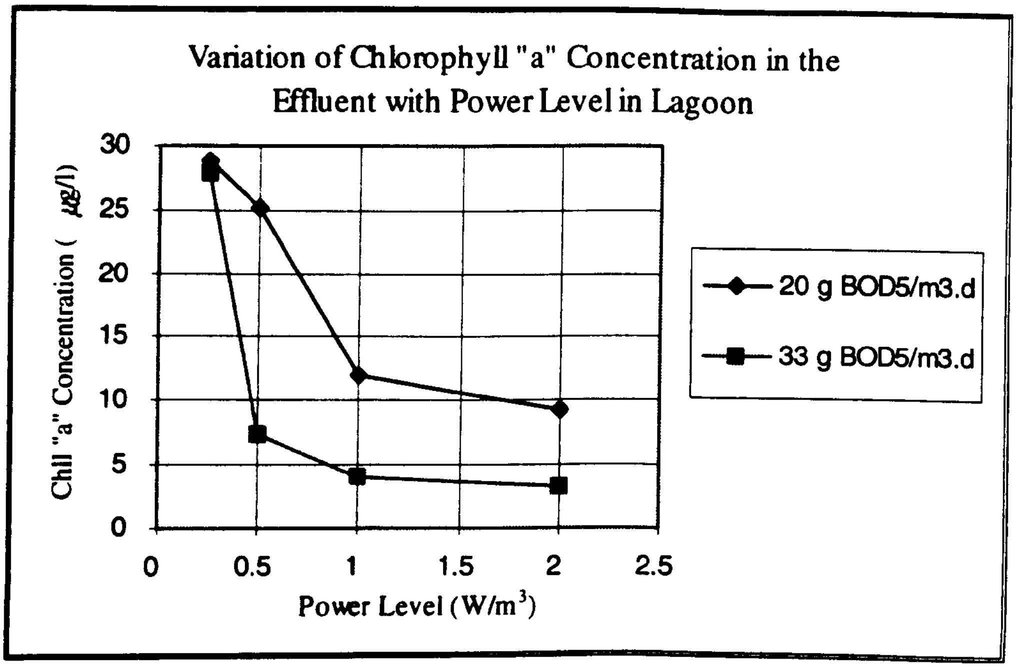


Figure 7.9 Variation of Chlorophyll "a" Concentration with Power Level.

hampered light penetration. Thus the major cause of low algae concentrations at high power levels was reduced light penetration resulting from mixing. Malina *et al.* (1971b) observed that algae are present in the facultative aerated lagoon at hydraulic detention times of about 3.5 days. Mixing intensity at a power level of 5 W/m^3 (McKinney and Edde, 1961) and 5 to 6 W/m^3 (Rich, 1978) was sufficient to suppress the growth of algae. Thermostratification, which provides an optimal environment for algae to become established and grow, will not exist if mixing is sufficient. In lagoons of 1 m depth and with a retention time of less than 50 hours, little (or negligible) growth of algae is expected (Water Pollution Research Laboratory, 1973).

Algae contributes to both TSS and BOD_5 and their control in facultative aerated lagoons is highly recommended. The increase in effluent BOD_5, COD and solids concentrations at low power levels could be partially accounted for by the presence of algae in the effluent. White and Rich (1976a) gave a correlation between chlorophyll "a" and TSS which estimated that each gram of chlorophyll "a" contributed 143 grams to the TSS. Algae exert high BOD in the standard laboratory BOD test involving dark room incubation (Khandual, 1987). An oxygen demand equivalent to about 1 mg BOD_5/mg algal suspended solids will be exerted in the traditional BOD test, whereas if the BOD test is completed under optimum lighting conditions, the algae will produce, on average, about ten times as much oxygen as they consume in the dark (Water Pollution Research Laboratory, 1973). Also Brayent (1988) quoted from Naehle (1987) that algal samples yielded 10 g COD/100 mg chlorophyll "a".

7.5 Relationship Between Mixing and Performance of Facultative Aerated Lagoons

Significant effects of mixing conditions on the performance of the model facultative aerated lagoons have been proved in this study. Therefore, a relationships between the parameters of the simulation model developed in this study (Chapter 5) and the performance parameters, both removal efficiencies and removal rate coefficients, are attempted in this section. Multiple regression analysis between simulation model parameters and process performance was carried out. The performance of the model facultative aerated lagoon was taken as the dependent variable and both simulation model parameters and temperature were treated as the independent variables. Temperature was included because of its importance in affecting performance (Chapter 6). The data were regressed linearly and the empirical equations are shown in Table 7.1.

It can be observed from the empirical equations that there is a positive relationship between the BOD_5 and COD removal efficiencies and the volume percentages of the segments by which the lagoon was simulated. BOD_5 removal rate coefficients, both filtered and unfiltered, are positively proportional to volume percentages of the segements whereas, COD removal rate coefficients, are negatively proportional to this these simulation model parameters. The degree of the backmixing and the recycle flow between the segments of the simulation model, as were represented by the backmix and recycle ratios, respectively, have showed a relationship with the performance of the lagoon. As have been shown in the Chapter 6 and this chapter, the performance of the facultative aerated lagoon is positively proportional to the in-lagoon temperature level.

Table 7.1 Relationships between the Model Facultative Aerated Lagoon Performance and Simulation Model Parameters of the EAD Curves.

Relationship	R^2
i) Biochemical oxygen demand	
$BOD_5\ FPR = -2460.9 + 24.9PV_1 + 25.1PV_2 + 23.8PV_3 + 25.2PV_4 - 0.16BM_{12} + 18.8BM_{23} - 23BM_{34} - 2.3RE_2 + 3.9RE_3 + 9RE_4 + 1.72Temp$	0.74
$BOD_5\ UPR = -1299 + 13.5PV_1 + 13.6PV_2 + 13.1PV_3 + 13.9PV_4 - 0.52BM_{12} + 2.2BM_{23} - 6.6BM_{34} - 4.6RE_2 + 2RE_3 - 11.5RE_4 + 1.1Temp$	0.77
$BOD_5\ K_f = -13.2 + 0.132PV_1 + 0.13PV_2 + 0.119PV_3 + 0.128PV_4 + 0.014BM_{12} + 0.266BM_{23} - 0.24BM_{34} - 0.011RE_2 + 0.023RE_3 + 0.339RE_4 + 0.049Temp$	0.97
$BOD_5\ K_u = -1.68 + 0.017PV_1 + 0.015PV_2 + 0.022PV_3 + 0.013PV_4 + 0.014BM_{12} - 0.123BM_{23} + 0.14BM_{34} + 0.029RE_2 - 0.029RE_3 + 0.092RE_4 + 0.023Temp$	0.94
ii) Chemical oxygen demand	
$COD\ FPR = -1700.2 + 17.2PV_1 + 17.3PV_2 + 16.37PV_3 + 17.51PV_4 - 0.394BM_{12} + 13.53BM_{23} - 16.89BM_{34} - 2.89RE_2 + 3.21RE_3 - 1.62RE_4 + 1.23Temp$	0.73
$COD\ UPR = -1151.5 + 11.82PV_1 + 11.89PV_2 + 11.72PV_3 + 12.1PV_4 - 0.29BM_{12} - 0.345BM_{23} - 2.4BM_{34} - 2.55RE_2 + 1.14RE_3 - 10.57RE_4 + 0.8Temp$	0.76
$COD\ K_f = 5.29 - 0.051PV_1 - 0.053PV_2 - 0.049PV_3 - 0.065PV_4 + 0.023BM_{12} + 0.082BM_{23} + 0.035BM_{34} + 0.16RE_2 - 0.037RE_3 + 0.62RE_4 + 0.012Temp$	0.71
$COD\ K_u = 2.96 - 0.028PV_1 - 0.029PV_2 - 0.024PV_3 - 0.033PV_4 + 0.008BM_{12} - 0.06BM_{23} + 0.1BM_{34} + 0.053RE_2 - 0.019RE_3 + 0.155RE_4 + 0.003Temp$	0.81

Symbols and Abbreviations

FPR	Filtered percentage removal	percent
UPR	Unfiltered percentage removal	percent
K_f	Filtered removal rate coefficient	d^{-1}
K_u	Unfiltered removal rate coefficient	d^{-1}
PV_n	Percentage of the segment volume in the whole lagoon volume	percent
Temp	Temperature	°C
BM	Backmix ratio	unitless
RE	Recycle ratio	unitless

7.6 Application of the Research Results to Full-Scale

In wastewater treatment research different scales of experimental equipment are frequently used; however, processes change as the scale changes. It will be important to establish relationships between these processes occurring at different scales. Performance of a full-scale facultative aerated lagoon process might be expected to differ from that of a laboratory-scale lagoon. Thus it is important to know what the performance would be if such change in size applied. The data collected in this study describes the performance of the model facultative aerated lagoons used in the experimental work when operated at different levels of power input, organic loading and temperature. Model performance does not necessarily reflect the performance of a full scale facultative aerated lagoon operated with similar operating variables. Upon scale-up mixing levels and oxygen transfer rate are not necessarily would be similar to that in the model lagoon at same power level. Sackellares (1985) pointed out that because of the slow growth rate of the anaerobic community and the complexity of the fluid flow field, scale-up of the results for full scale application is difficult. However, it gives, at similar operating conditions, an approximate performance level as well as a prediction of the trend of such performance of the full scale lagoon when the levels of the controlled variables are changed.

Developing design equations and kinetic constants are strongly dependent on data collected from well operated and well controlled full-scale systems (Middlebrooks, 1987). Prediction of the equilibrium operating conditions in full-scale facultative aerated lagoons is an important objective of laboratory treatment studies (Blane *et al.*, 1984). The effects of variables are usually studied on pilot scale equipment followed by establishment of a scale-up criterion to predict process results at larger scale. These pilot and laboratory scales are important tools for obtaining reliable design figures for full-scale treatment plants. The rate of biological removal in facultative aerated lagoons is affected by substrate concentration, temperature, pH value, degree of dilution, dissolved oxygen concentration, presence of inhibitory substances and the need for adequate trace concentrations of nutrients. None of these parameters is scale-dependent, therefore the biological kinetics are not affected by scale (Cooper and Boon, 1983). However, as the size of the facultative aerated lagoon increases from laboratory scale to full scale, the shear force near the aerator will increase in the vicinity of the diffusers (diffused air aeration) or impeller (mechanical aeration) if a similar power level is to be maintained. Maintaining similar shear and hydraulic characteristics that affect the bacterial biomass is not possible as scale increases (Oldshue, 1983). Furthermore, increase in oxygen transfer efficiency (Chapter 5) at full-scale is another important factor with increasing lagoon size. Similarity between the model and prototype are important to model the operation correctly.

The low algal biomass concentrations in the model facultative aerated lagoons indicates the insignificant role of algae in oxygen production via the photosynthesis process. Therefore, the dissolved oxygen in the lagoon comes mainly from the diffused air aerators located at the bottom of the model lagoons, with a low percentage being transferred from the atmosphere through the induced surface aeration. Light penetration will be limited to only the first few centimetres below the liquid surface.

Full scale facultative aerated lagoons performance are expected to be not highly different from the model facultative aerated lagoon studied in this research. The results of this study are applicable to a facultative aerated lagoon following a primary anaerobic pond or to a facultative aerated lagoon treating primary treated wastewater. High organic loading (short retention time) in facultative aerated lagoons operated at low power levels proved to remove more organic material per day per unit of power introduced into the lagoon. According to the results of this research, if the facultative aerated lagoon is to be used to treat wastewater without using secondary settling tank, the optimum conditions of organic loading and power level are 0.25 W/m^3 for power level and 62 g $BOD_5/m^3.d$ for organic loading (3 to 4 days retention time).

7.7 Effluent from Facultative Aerated Lagoons

7.7.1 Effluent Disposal and Reuse

The effluents from facultative aerated lagoons and many other treatment processes are either discharged to rivers, seas, or lakes which local people use for drinking water or recreation. An alternative is the reuse of these effluents, provided that minimum standards required by each particular user are met, for agricultural (irrigation), aquacultural (e.g. fishponds), municipal (e.g. drinking water), industrial, recreational and navigational (e.g. ponds for boating) and groundwater recharge uses. The latter can either be considered as a disposal method for such effluents or a method for conserving water resources. Aquifers in some areas are used to store sewage effluent intended as irrigation waters. The reuse of treated effluent is considered as one of the available disposal methods. However, the amount of effluent that can be reused is affected by the availability and cost of fresh water, transportation and treatment costs, water-quality standards, and the reclamation potential of the wastewater.

In the previous parts of this research the performance of a facultative aerated lagoon used as a single treatment process was studied intensively at various operating levels of power, organic loading and temperature. In this part of the discussion, consideration of the above alternatives for facultative aerated lagoon effluents disposal and reuse will be discussed. Suggestions for upgrading of the effluent quality, if needed, by applying some additional treatment processes will be pointed out.

Based on the performance results of this treatment process as observed in this study, the degree of treatment, if required, before such application for the effluent will be presented for the following uses or applications of the effluent:

i) Agricultural Irrigation

The quality of the effluent from facultative aerated lagoons as well as the health regulations will control the types of crops that can be irrigated. Guidelines recommended by WHO (1989), proposed guidelines by WHO (1987) and recommended standards by the World Bank (Arthur, 1983) are summarized in Table 7.2. The effluent quality from facultative aerated lagoons (FAL) is also presented in Table 7.2. As can be observed from the table, the effluent from facultative aerated lagoons has a suspended solids concentration higher than the limits for such usage, therefore additional removal of these solids would be needed. In addition, for certain operating conditions, faecal coliform concentration in the effluent from facultative aerated lagoon was higher than the guideline limit by WHO (1989) and WHO (1987) as well as World Bank limits. So, for these conditions additional removal of faecal coliform is required.

ii) Fish Rearing

For fishponds the guideline limits, as proposed by WHO (1987), are also summarized in Table 7.3. By comparing the figures in the table it can be observed that there is a need for further treatment of facultative aerated lagoon effluents for

Table 7.2 Effluent Quality from FAL and Effluent Standards for Agricultural Irrigation Reuse

Parameter	Effluent from FAL*	Recommended guidelines (WHO, 1989)	Reuse Limits (WHO, 1987)	Reuse limits (World Bank)+
BOD_5	39-72		No limits	60 mg/l
Suspended solids	41-76		< 30 mg/l	
Total Kjeldahl nitrogen	24-47		No limit	
Ammonia nitrogen	18-38		No limit	
Total phosphorus	6-9.6		No limit	
Faecal coliform	2.14×10^{2}-5.56×10^{6}	≤ 1000/100 ml	< 1000/100 ml	50,000/100ml

* For the ranges of operating parameters applied in this research.
+ For irrigation of trees, cotton, and other non-edible corps.

Table 7.3 Effluent Quality from FAL and WHO (1987) Effluent Guideline limits for Fish Rearing Reuse

Parameter	Effluent from FAL*	Guideline Limits
BOD_5	39-72	< 10 mg/l
Suspended solids	41-76	Low
Total Kjeldahl nitrogen	24-47	
Ammonia nitrogen	18-38	< 0.5 mg/l
Total phosphorus	6-9.6	No limit
Faecal coliform	2.14×10^{2}-5.56×10^{6}	< 1000/100 ml

* For the ranges of operating parameters applied in this research.

removing suspended solids in the effluent. Removal of faecal coliforms is also needed for all operating levels of power, temperature, and organic loading (Chapter 6). Nitrogen removal should also be considered.

iii) Groundwater Recharge

Groundwater recharge is one of the most common methods for combining water reuse and effluent disposal. Recharge has been used to replenish groundwater supplies in many areas. Stopping or preventing saltwater intrusion into the freshwater aquifers is also accomplished by groundwater recharge.

Proposed guidelines by WHO (1987) are summarized in Table 7.4. The effluent quality from facultative aerated lagoons is also presented in Table 7.4. Suspended solids removal is also required. Faecal coliform removal requirements are similar to the preceding types of effluent reuse.

Table 7.4 Effluent Quality from FAL and Effluent Standards for Reuse for Groundwater Recharge

Parameter	Effluent from FAL*	Limits
BOD_5	39-72	< 5 mg/l
Suspended solids	41-76	< 30 mg/l
Total Kjeldahl nitrogen	24-47	Low
Ammonia nitrogen	18-38	Virtually none
Total phosphorus	6-9.6	< 10 mg/l
Faecal coliform	2.14×10^2-5.56×10^6	< 1000/100 ml

* For the ranges of operating parameters applied in this research.

iv) Discharge to water bodies

If facultative aerated lagoon effluent is not to be reclaimed for further use, it will have to be disposed of somewhere, most often into a stream or a lake. The self-purification and dilution capacity of a receiving water under conditions of critical flow should be known in order to determine the degree of treatment to be provided for such effluent. If the receiving body is a non-eutrophic lake, the nutrient content of the facultative aerated lagoon effluent is often the most critical factor which needs to be taken into account to prevent eutrophication. According to the standards set by U.S. Environmental Protection Agency (Metcalf and Eddy, 1979) and recommended standards by the World Bank (Arthur, 1983) as well as Modified Waste-water Quality Standards in Jordan For discharge in wadis, rivers, or reservoirs (Table 7.5), it can be said that there is a need for suspended solids and faecal coliform removal for certain operational conditions.

If the effluent is to be discharge to a turbid river, as the case in tropical countries, suspended solids level in the effluent from facultative aerated lagoons will not cause any problem.

7.7.2 Effluent Upgrade

According to the above comparisons of the different reuse standards with the effluent quality from the facultative aerated lagoon, the effluent might need to be upgraded to comply with the standards for some uses. Effluent quality improvement methods include microstraining, rapid sand filtration, flotation, chlorination, ozonation, and treatment with activated carbon. These are not appropriate for use in developing countries and are too costly. Maturation (polishing) ponds following the facultative aerated lagoon is the most economical

Table 7.5 Effluent Quality from FAL and Effluent Standards Promulgated by the U.S. Environmental Protection Agency and Modified Wastewater Quality Standards in Jordan

Parameter	Average concentration limits (U.S. Env. Pro. Agency)		Reuse limits (World Bank)	Modified Waste-water Quality Standards in Jordan For discharge in wadis, rivers, or reservoirs*	Effluent from FAL**
	Monthly	Weekly			
BOD_5	30 mg/l	45 mg/l	25		39-72 mg/l
Suspended solids	30 mg/l	45 mg/l		50 mg/l	41-76 mg/l
Faecal coliform			5,000		2.14×10^{2}-5.56×10^{6}

* Source (Miqdadi, 1992).

** For the ranges of operating parameters applied in this research.

alternative to these effluent upgrading processes. Improving the microbiological quality of the effluent as well as removing the solids will be simultaneously accomplished in such a pond. BOD_5 concentration of the facultative aerated lagoon effluent will also be improved.

Considering the faecal coliform removal at the worst operating conditions, three maturation ponds each with 6.5 days retention time would be sufficient to produce an effluent suitable for different reuses. This design was performed using the equation presented by Mara *et al.* (1992a) as follows:

$$\theta_m = \{[N_i/N_e]^{(1/n)} - 1\}/R_T$$

where, θ_m= Retention time required in the maturation pond; day,

R_T = First order removal rate for faecal coliform removal at T temperature; d^{-1}, and

N_i and N_e = number of faecal coliform per 100 ml of influent and effluent, respectively.

the value of k used was 2.6 d^{-1} for 20°C climates.

It should be noted that for certain operating conditions and effluent uses, as discussed earlier, there is no need for such polishing ponds.

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

The results of this study confirm that facultative aerated lagoons are effective in removing both organic and inorganic matter. Also, this research has revealed the effects of different variable parameters on the performance of facultative aerated lagoons and determined the performance of this wastewater treatment process at different levels of organic loading, power level, and temperature and how performance changes with changes in the operating variables. The following conclusions can be drawn:

1. Removal of organic substrate was always found to be relatively high. For BOD_5, as much as 87 percent for unfiltered samples and 91 percent for filtered samples was removed over the range of variable parameters applied. The corresponding COD removals were less, at 62 percent and 67 percent, respectively. However, these levels of removal are less than for other advanced treatment processes, such as activated sludge, but the high-maintenance requirements and the mechanical complexity of those processes make facultative aerated lagoons an appropriate method to achieve high removals at low cost.

2. Organic loading (or its inverse equivalent, retention time) significantly, at the 5% significance level, affected the filtered BOD_5 removal rate coefficient positively and the unfiltered BOD_5 removal rate coefficient negatively. These two coefficients varied between 0.588 d^{-1} and 1.585 d^{-1} for filtered BOD_5 removal rate coefficients and between 0.318 d^{-1} and 0.841 d^{-1} for unfiltered BOD_5 removal rate coefficients over the range of power levels, organic loading, and temperatures applied in this study.

3. Both COD removal rate coefficients, filtered and unfiltered, were positively and significantly affected by the organic loading level applied in the facultative aerated lagoon. The filtered COD removal rate coefficient ranged from 0.270 d^{-1} to 0.838 d^{-1} whereas the unfiltered COD removal rate coefficient ranged from 0.1 d^{-1} to 0.304 d^{-1}.

4. Removal efficiencies of BOD_5 and COD were negatively affected by organic loading (or positively by retention time) and the effect was significant.

5. The performance of facultative aerated lagoons was affected by power level. In terms of removal rate coefficients the effect of power level was insignificant for unfiltered removal rate coefficients for both BOD_5 and COD, whereas it was positive and significant for filtered removal rate coefficients, except for COD in lagoons operated at low loadings (20g BOD_5 /m^3.d or 52 g COD /m^3.d) and high temperature (30°C).

6. Removal efficiencies for BOD_5 and COD were positively affected by power level except for COD in facultative aerated lagoons operated at high temperature (30°C). The effect of power level on filtered removal efficiency was higher than on unfiltered removal efficiency.

7. The temperature effect on the removal rate coefficients, both filtered BOD_5 and COD as well as unfiltered BOD_5, in facultative aerated lagoons was significant and positive. However, the unfiltered COD removal rate coefficient was not significantly affected by temperature. The effect of temperature on the removal rate coefficients, except the unfiltered COD removal rate coefficient, was higher at higher organic loadings (shorter retention times).

8. BOD_5 and COD removal efficiencies were positively and significantly affected by temperature. This effect was higher on removal efficiencies for filtered samples than for unfiltered samples. The effect was higher on those facultative aerated lagoons operated at lower power levels than at higher levels.

9. Effluent quality from these lagoons was considered satisfactory for operations with 2 W/m^3 power levels and low loadings (20g BOD_5 /m^3.d). Filtered effluent sample BOD_5 concentrations of 17 mg/l and 13 mg/l were obtained for runs at 20°C and 30°C temperatures, respectively, when treating domestic sewage.

10. Temperature correction coefficients ranged from 1.03 to 1.06 for BOD_5 and 1 to 1.05 for COD. The temperature correction coefficients for BOD_5 at low power levels were higher than at higher levels of power. The effect of power levels on temperature correction coefficients was significant, whereas the organic loading (or retention time) effect was insignificant.

11. Modelling the combined effects of power level and organic loading on BOD_5 and COD removal rate coefficients and removal efficiencies at two temperature levels was performed and resulted in empirical equations that can be used to predict performance for any particular conditions, within the range of variables studied in this research.

12. High organic loading (short retention time) in facultative aerated lagoons operated at low power levels proved to remove more organic material per day per unit of power introduced into the lagoon. Based on this research, the optimum conditions of organic loading and power level at which facultative aerated lagoons should be operated, if it is used solely, are considered to be 0.25 W/m^3 for power level and 62 g BOD_5 /m^3.d for organic loading (3 to 4 days retention time).

13. Mixing studies utilizing the response to a pulse tracer input showed that the number of tanks in series necessary to simulate the model facultative aerated lagoon was in the range of between two and four. The dispersion number for these model facultative aerated lagoons was found to be between 0.17 and 0.49, which was positively affected by power level. The C-curves of the facultative aerated lagoons were simulated by a C-curve of a model consisting of unequal-tanks-in-series with backmixing and recycle flows between tanks. The simulation model derived is described by a set of parameters. Empirical equations relating the performance of facultative aerated lagoons and the parameters of the simulation model and temperature were developed.

14. Reduction of ammonia nitrogen concentration in the facultative aerated lagoons operated at high power level (2 W/m^3) and low organic loading (20 g BOD_5 $/m^3.d$) was very high (as high as 70 percent at 30°C temperature). However, there was an increase in the concentration between the influent and effluent at low power levels and low temperatures, which is attributed to the release of nitrogen from bottom deposits as a result of anaerobic degradation and conversion of organic nitrogen to ammonia.

15. The effect of power level on ammonia nitrogen removal was significant and positive, whereas organic loading (or its inverse equivalent, retention time) had a negative effect.

16. Effluent total Kjeldahl nitrogen concentrations as low as 20 and 24 mg/l for filtered and unfiltered samples, respectively, were obtained from the facultative aerated lagoons treating domestic wastewaters with influent levels of 47 and 59 mg/l, respectively. The removal efficiency of TKN in these facultative aerated lagoons was observed to be positively affected by power level for 20°C operation, whereas for 30° C operation this effect was positive only for power levels less than 0.5 W/m^3, beyond which a negative effect was experienced.

17. Nitrification/denitrification mechanisms were the main causes of nitrogen removal in facultative aerated lagoons.

18. Phosphorus removal in the facultative aerated lagoons was significant. Removals as high as 26 percent at 30°C for orthophosphate were obtained at high power levels and low loadings (long retention times). As a general trend, aside from some operational conditions, the removals were positively affected by power level and negatively by organic loading (or positively by retention time). The probable removal of phosphorus in facultative aerated lagoons is solely by incorporation into bacterial cell tissue.

19. The facultative aerated lagoons effectively removed coliform bacteria as typified by *Escherichia. coli (E. coli)* and *faecal streptococci*. Removal of 99.999 percent were achieved under certain conditions; however, further removals from the effluent of this treatment process would be needed before reuse for irrigation.

20. Algal concentration, assessed by chlorophyll "a" concentration, in the facultative aerated lagoons decreased sharply with increase in power level. Increase in organic loading (or decrease in retention time) reduced the concentration of algal biomass. When present, the predominant algal genera were *Euglena, Oscillatoria* and *Chlamydomonas* at low power levels and low loadings.

21. Solids concentration in the effluent from the facultative aerated lagoons, considered to be solids in suspension, was positively and significantly affected by power level whereas the organic loading (or hydraulic retention time) effect were less significant. Effluent suspended solids as low as 41 mg/l were observed at low power levels and these were due partly to the presence of algae in the effluent. A relationship between power level and suspended solids concentration was determined.

22. A power level of around 0.5 W/m^3 was the threshold for settleable solids suspension, below which no significant decrease in effluent suspended solids concentration took place.

23. A mathematical equation was developed in this study to determine mean retention time of the solids (SRT) in facultative aerated lagoons. For the operational runs performed in this study, the ratio between SRT and hydraulic retention time (HRT) was determined and found to be affected negatively by power level and organic loading. An empirical equation was developed between SRT/HRT and ,simultaneously, power level and organic loading.

24. Relationships between the individual operating parameters and performance parameters are presented in the form of empirical equations, using the performance data collected in this study.

8.2 Recommendations

Resulting from this research further investigations are recommended. The following points summarize these recommendations:

1. The effects of facultative aerated lagoon depth on performance need to be investigated to define the optimum depth for certain operating conditions.

2. The performance of this treatment process when treating raw domestic sewage and other types of wastewater need to be evaluated for both scales, laboratory and full scale, at different levels of operating variables.

3. Full scale studies of the effects of the variable parameters applied in this study and other uncontrollable factors, such as actual sunlight and wind effects, are recommended.

4. The rate of soluble nutrients feedback rate released from the bottom deposits and its effect on facultative aerated lagoon performance should also be investigated.

5. The effects of higher and lower levels of organic loading and power level, as well as lower temperatures that might be inherent in some geographical areas, require investigation.

6. Investigating the effect of organic loading at fixed retention time on the performance of facultative aerated lagoons is also recommended. This will require a variable strength influent.

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APPENDIX A

Determination of the Over-all Oxygen Transfer Coefficient (K_La) and Aeration Efficiency

The non-steady state deoxygenation-reoxygenation approach was used for the evaluation of the aeration system used. This evaluation include the determination of the over-all oxygen transfer coefficient (K_La) and aeration efficiency. Deoxygenation of the tap water in the aeration tank was conducted by using sodium sulfite ($Na_2 SO_3$) solution and cobalt chloride ($CoCl_2$) catalyst. As $Na_2 SO_3$ dissolved in the test water, it will immediately ionize to sodium and sulfite ions which will react with dissolved oxygen to form $Na_2 SO_4$ according to the reaction:

$$2\ Na_2 SO_3 \rightarrow 4\ Na^+ + 2\ SO_3^=$$

$$2\ SO_3^= + O_2 \rightarrow 2\ SO_4^=$$

Therefore, by adding sufficient sodium sulfite the dissolved oxygen concentration in the aeration tank become zero.

The procedure is as follows:

1. Add sufficient cobaltous chloride ($CoCl_2.6H_2O$) in solution form to provide a cobalt-ion concentration of 2 mg/l.

2. Add sufficient sodium sulfite ($Na_2 SO_3$) dissolved in water to reduce the DO level to zero. The amount required is 9-12 mg $Na_2 SO_3$/l per one mg/l DO initially present in the water.

3. Run the aeration device and at specific time intervals measure the DO concentration in different points of the tank and terminate the test when the basin dissolved oxygen approaches within about 0.5 mg/l of saturation.

4. A plot of $\ln[(C_S-C_1)/(C_S-C_2)]$ versus time will give a linear trace of slope K_La . Also K_La is determined using the following equation:

$$K_La = \frac{\ln\left(\frac{C_S-C_{t_1}}{C_S-C_{t_2}}\right)}{t_2-t_1}$$

$$C_S = C_{S_C}\left(\frac{P_b}{29.4}+\frac{O_t}{42}\right)$$

where, P_b = The absolute pressure at the depth of air release, and

O_t = The volume-percent-concentration of oxygen in the air leaving the test unit.

5. Rate of total oxygen transferred is estimated as follows:

$$R = K_La \times C_S \times W$$

where, R = Rate of total oxygen transferred; g/hr, and

W = weight of the water in the aeration tank; g,

6. Calculate the aeration efficiency (E) as follows:

$$E = \frac{R}{\text{Rate of oxygen supplied}} \times 100$$

APPENDIX B

Determination of Chlorophyll "a" Concentration

The method used for the determination of Chlorophyll "a" concentration in the this research is the methanol extract technique. This method is described by Mara *et al.* (1992a). In this method 1% (W/V) aqueous suspension of magnesium carbonate (Mg CO_3) is used to aid retention of algae and *maintain alkaline conditions to prevent* denaturation of the Chlorophyll during extraction. The procedure of this method is as follows:

1. Filter 2.5 ml of 1% (W/V) aqueous suspension of magnesium carbonate (Mg CO_3) through a glass fibre filter paper.

2. Filter a known volume, V_S, (10-50 ml) of the sample to be tested for Chlorophyll "a" through the same filter paper. This subsample should be taken from a well-stirred effluent or in-lagoon sample.

3. Place the filter paper in a centrifuge tube and add 10 ml of 90% (V/V) methanol.

4. Place the centrifuge tube in a 65°C water bath and boil for two minutes to extract the Chlorophyll.

5. Make up the extract volume to 10 ml by adding 90% methanol

6. To remove suspended materials use either of the following
   a. Centrifuge the centrifuge tube contents at 500 g for 10 minutes
   b. If a centrifuge in not available, leave the extract for 15 minutes in the dark to allow most of the debris to settle out.

7. Transfer a known volume of the clarified extract, V_e ,to a cuvette of L light path length.

8. Measure the absorbency at 663 nm and 750 nm against a 90% methanol blank (i.e. zero the absorbency reading using the blank sample before measuring the absorbency of the actual sample.

9. Chlorophyll "a" concentration is calculated using the following equation:

$$\text{Chla } (\mu g/l) = [A_{663}\text{-}A_{750}] \times [V_e/(V_S \times L \times E] \times 10^6$$

where, Chla = Chlorophyll "a" concentration; μg/l,

A_{663} = Absorbency reading at 663 nm wavelength

A_{750} = Absorbency reading at 750 nm wavelength

V_e = Solvent extract volume; ml,

V_S = Original sample volume; ml,

L = Light path length of the cuvette; cm, and

E = Extinction coefficient for chlorophyll "a" in a 90% methanol and equal 77 l/g.cm.

APPENDIX C-I

```
   PROGRAM HYDCH
C   THIS PROGRAM IS WRITTEN IN FORTRAN LANGUAGE AND
C   USED TO:
C   1. DETERMINE THE ACTUAL HYDRULIC RETENTION TIME OF
C      A SYSTEM (FROM ITS C-CURVE)
C   2. DETREMINATION OF VARIANCE AND DISPERSION NO.
C   3. NUMBER OF TANKS-IN-SERIES, ANTA (USING EQUATION 4.6)
C   4. DEAD SPACE VOLUME
C
C   THRT= THEORITICAL HYDRULIC RETENTION TIME (DAYS) (V/Q)
C   N= NUMBER OF T-C DATA
C   T= TIME (MIN)
C   C= CONCENTRATION
C
   DIMENSION T(1000),C(1000)
   DOUBLE PRECISION SC,STC,ST2C
   DOUBLE PRECISION TT,DMN,SS,DDD
   DOUBLE PRECISION ATC,AC,AT2C,ANTA,DV
C
   OPEN(5,FILE='S25-3')
   OPEN(6,FILE='S25-3s')
   READ(5,*) THRT,N
C
   DO 100 I=1,N
   READ(5,*) T(I),C(I)
C
100 CONTINUE
C
C===================***************=====================
C
   ST2C=0.
   SC=0.
   STC=0.
   AT2C=0.
   AC=0.
   ATC=0.
   TT=0.
   DMN=0.
   SS=0.
   DDD=0.
C
   DO 150 KA=1.,N
C
    ST2C=ST2C+((T(KA)**2)*(C(KA)))
    SC=SC+(C(KA))
    STC=STC+((T(KA))*(C(KA)))
```

```
 150  CONTINUE
     DO 155 KZ=1.,N
C
    AT2C=AT2C+((T(KZ)**2)*(C(KZ)))
    AC=AC+(C(KZ))
    ATC=ATC+((T(KZ))*(C(KZ)))
    DHRT=2*24*THRT
    IF(T(KZ).GT.DHRT) GOTO 160
 155    CONTINUE
C
 160  TT=(ATC/AC)/(0.68696)
     SS=((ST2C/SC)-((STC/SC)**2))/((STC/SC)**2)
     DMN=SS/2
165  DDD=(2.*DMN)-((2.0*DMN**2.0)*(1.0-EXP(-(1.0/DMN))))
C
    IF(DDD.LT.SS) THEN
C
    DMN=DMN+0.0001D0
C
    PRINT*,'DMN=',DMN
    GOTO 165
C
    END IF
    DV=((TT/(24*THRT))-1)*81.0
C   ====================================================
C            NUMBER OF TANKS IN SERIES (Equation 4.6)
    ANTA=1./SS
C   +++++++++++++++++++++++++++++++++++++++++++++++++++
C                PROGRAM OUTPUT
   WRITE(6,740) ANTA
740  FORMAT(T2,'NUMBER OF TANKS IN SERIES (MODEL A)=',F15.6)
C
    WRITE(6,750) TT
750  FORMAT(T6,'Act. Calc. Hyd. Ret. Time in hours=',F10.4)
C
    WRITE(6,752) THRT*24
752  FORMAT(T6,'Theoretical Hyd. Ret. Time in hours=',F10.4)
C
     WRITE(6,755) DMN
755  FORMAT(T6,'Calc. Dispersion No.=',F10.4)
C
    WRITE(6,757)DV
757  FORMAT(T6,'Calc. Dead Space Volume=',F10.4)
C
    STOP
    END
```

APPENDIX C-II

```
   PROGRAM TRISM
C
C    THIS PROGRAM IS WRITTEN IN FORTRAN LANGUAGE AND USED C        TO
SIMULATE C-CURVE OF CERTAIN SYSTEMS WHICH IS DIVIDED C        TO
IMAGINARY SEGMENTS WITH BACKMIXING BETWEEN THESE C
     SEGMENTS AND FLOW RECYCLE
C
C    V= TOTAL VOLUME OF THE SYSTEM (L)
C
C    V1,V2,V3,V4= THE VOLUMES OF THE SEGMENTS ASSUMED IN THE C
     SYSTEM (L)
C
C    BM12,BM23,BM34= BACKMIXING:NET FLOW RATIOS BETWEEN
C    THE SEGMENTS
C
C    RE2, RE3, RE4= RECYCLE RATIO (e.g; 0.3,...etc.)
C
C    CU = UNIFORM TRACER CONC. IN THE RECTOR IF INITIAL
C    COMPLETE MIXING (mg/l) (e.g; 1,2,...etc.)
C
C    F= (NET) FLOWRATE THROUGH THE SYSTEM (L/HR)
C
C   CZ= TRACER CONCENTRATION ENTERING THE SYSTEM (mg/L)
C
C    HRT2= ACTUAL HYDRULIC RETENTION TIME (HR)
C
   DIMENSION C1(50000),C2(50000),C3(50000),C4(50000),T(50000)
C
   DOUBLE PRECISION FLV1,FLV2,FLV3,FLV4
   DOUBLE PRECISION AK1,AK2,AK3,AK4
   DOUBLE PRECISION Z1,Z2,Z3
   DOUBLE PRECISION DCF,DCS,DCT,DCH
   DOUBLE PRECISION FN,FNN
   DOUBLE PRECISION CF,CS,CT,CH
C
   INTEGER N,LJ,ITP
C
   OPEN(8,FILE='recres1')
C
C    DT= ITS UNIT IS HR
C    N*DT = Should be not les than 2 THRT
C    F = Is expressed in l/hr
   DT=0.010D0
   N=50000
   F=0.33750D0
```

```
      ITP=49
 C
      V1=65.000D0
      V2=9.0000D0
      V3=5.0000D0
      V4=2.0000D0
 C
      BM12=1.0D0
      BM23=1.0D0
      BM34=1.0D0
 C
      RE2=0.5D0
      RE3=0.5D0
      RE4=0.5D0
 C
      CU=1.0D0
 C
      CZ=0.0D0
      CF=0.0D0
      CS=0.0D0
      CT=0.0D0
      CH=0.0D0
 C
      AK1=0.0D0
      AK2=0.0D0
      AK3=0.0D0
      AK4=0.0D0
 C
      Z1=0.0D0
      Z2=0.0D0
      Z3=0.0D0
      Z4=0.0D0
 C
 C    FOR THE FIRST DT
      CZ=(CU*(V1+V2+V3+V4)/(F*DT))
      FRE=(F*RE2*CS)+(F*RE3*CT)+(F*RE4*CH)
      FLV1=((F*CZ)+(F*BM12*CS)- (((F*(1+RE2+RE3+RE4))+F*BM12)*CF)+
      FRE)/V1
      AK1=FLV1*DT
      FE=(F*(1+RE2+RE3+RE4))
      Z1=(-(FE+F*BM12)*(CF/V1))+(AK1/2)
      AK2=((((F*CZ)+(F*BM12*CS)+FRE)/V1)+Z1)*DT
      Z2=(-(FE+F*BM12)*(CF/V1))+(AK2/2)
      AK3=((((F*CZ)+(F*BM12*CS)+FRE)/V1)+Z2)*DT
      Z3=(-(FE+F*BM12)*(CF/V1))+(AK3)
      AK4=((((F*CZ)+(F*BM12*CS)+FRE)/V1)+Z3)*DT
      DCF=(AK1+2*AK2+2*AK3+AK4)/6
 C
      FN=FE+(F*BM12)+(F*BM23)-(F*BM12)-(F*RE2)
```

```
      FRT=(F*RE2*CS)
      FLV2=((F*CF)+(F*BM12*CF)+(F*BM23*CT)-(F*BM12*CS)-(FN*CS)-
      FRT)/V2
      AK1=FLV2*DT
      Z1=-(((F*BM12)+FN+(F*RE2))*(CS/V2))+(AK1/2)
      AK2=(((((FE+F*BM12)*CF)+(F*BM23*CT))/V2)+Z1)*DT
      Z2=-(((F*BM12)+FN+(F*RE2))*(CS/V2))+(AK2/2)
      AK3=(((((FE+F*BM12)*CF)+(F*BM23*CT))/V2)+Z2)*DT
      Z3=-(((F*BM12)+FN+(F*RE2))*(CS/V2))+(AK3)
      AK4=(((((FE+F*BM12)*CF)+(F*BM23*CT))/V2)+Z3)*DT
      DCS=(AK1+2*AK2+2*AK3+AK4)/6
C
C
      FNN=FN+(F*BM34)-(F*BM23)-(F*RE3)
      FLV3=((FN*CS)+(F*BM34*CH)-(F*BM23*CT)-(FNN*CT)-
      (F*RE3*CT))/V3
      AK1=FLV3*DT
      Z1=-(((F*BM23)+FNN+(F*RE3))*(CT/V3))+(AK1/2)
      AK2=(((FN*CS+F*BM34*CH)/V3)+Z1)*DT
      Z2=-(((F*BM23)+FNN+(F*RE3))*(CT/V3))+(AK2/2)
      AK3=(((FN*CS+F*BM34*CH)/V3)+Z2)*DT
      Z3=-(((F*BM23)+FNN+(F*RE3))*(CT/V3))+(AK3)
      AK4=(((FN*CS+F*BM34*CH)/V3)+Z3)*DT
      DCT=(AK1+2*AK2+2*AK3+AK4)/6
C
C
      FLV4=((FNN*CT)-(F*BM34*CH)-(F*CH)-(F*RE4*CH))/V4
      AK1=FLV4*DT
      Z1=-(((F*BM34)+F+(F*RE4))*(CH/V4))+(AK1/2)
      AK2=((FNN*CT/V4)+Z1)*DT
      Z2=-(((F*BM34)+F+(F*RE4))*(CH/V4))+(AK2/2)
      AK3=((FNN*CT/V4)+Z2)*DT
      Z3=-(((F*BM34)+F+(F*RE4))*(CH/V4))+(AK3)
      AK4=((FNN*CT/V4)+Z3)*DT
      DCH=(AK1+2*AK2+2*AK3+AK4)/6
C
C
      CF=CF+DCF
      CS=CS+DCS
      CT=CT+DCT
      CH=CH+DCH
C
C
      C1(1)=CF
      C2(1)=CS
      C3(1)=CT
      C4(1)=CH
      T(1)=DT
C
```

```
C   FOR THE NEXT DTs
C
      CZ=0.0D0
      DO 20 J=2,N
C
      FRE=(F*RE2*CS)+(F*RE3*CT)+(F*RE4*CH)
      FLV1=((F*CZ)+(F*BM12*CS)-(((F*(1+RE2+RE3+RE4))+F*BM12)*CF)+
       FRE)/V1
      AK1=FLV1*DT
      FE=(F*(1+RE2+RE3+RE4))
      Z1=(-(FE+F*BM12)*(CF/V1))+(AK1/2)
      AK2=((((F*CZ)+(F*BM12*CS)+FRE)/V1)+Z1)*DT
      Z2=(-(FE+F*BM12)*(CF/V1))+(AK2/2)
      AK3=((((F*CZ)+(F*BM12*CS)+FRE)/V1)+Z2)*DT
      Z3=(-(FE+F*BM12)*(CF/V1))+(AK3)
      AK4=((((F*CZ)+(F*BM12*CS)+FRE)/V1)+Z3)*DT
      DCF=(AK1+2*AK2+2*AK3+AK4)/6
C
      FN=FE+(F*BM12)+(F*BM23)-(F*BM12)-(F*RE2)
      FRT=(F*RE2*CS)
      FLV2=((F*CF)+(F*BM12*CF)+(F*BM23*CT)-(F*BM12*CS)-(FN*CS)-
      FRT)/V2
      AK1=FLV2*DT
      Z1=-(((F*BM12)+FN+(F*RE2))*(CS/V2))+(AK1/2)
      AK2=(((((FE+F*BM12)*CF)+(F*BM23*CT))/V2)+Z1)*DT
      Z2=-(((F*BM12)+FN+(F*RE2))*(CS/V2))+(AK2/2)
      AK3=(((((FE+F*BM12)*CF)+(F*BM23*CT))/V2)+Z2)*DT
      Z3=-(((F*BM12)+FN+(F*RE2))*(CS/V2))+(AK3)
      AK4=(((((FE+F*BM12)*CF)+(F*BM23*CT))/V2)+Z3)*DT
      DCS=(AK1+2*AK2+2*AK3+AK4)/6
C
C
      FNN=FN+(F*BM34)-(F*BM23)-(F*RE3)
      FLV3=((FN*CS)+(F*BM34*CH)-(F*BM23*CT)-(FNN*CT)-(F*RE3*CT))/
      V3
      AK1=FLV3*DT
      Z1=-(((F*BM23)+FNN+(F*RE3))*(CT/V3))+(AK1/2)
      AK2=(((FN*CS+F*BM34*CH)/V3)+Z1)*DT
      Z2=-(((F*BM23)+FNN+(F*RE3))*(CT/V3))+(AK2/2)
      AK3=(((FN*CS+F*BM34*CH)/V3)+Z2)*DT
      Z3=-(((F*BM23)+FNN+(F*RE3))*(CT/V3))+(AK3)
      AK4=(((FN*CS+F*BM34*CH)/V3)+Z3)*DT
      DCT=(AK1+2*AK2+2*AK3+AK4)/6
C
C
      FLV4=((FNN*CT)-(F*BM34*CH)-(F*CH)-(F*RE4*CH))/V4
      AK1=FLV4*DT
      Z1=-(((F*BM34)+F+(F*RE4))*(CH/V4))+(AK1/2)
      AK2=((FNN*CT/V4)+Z1)*DT
```

```
      Z2=-(((F*BM34)+F+(F*RE4))*(CH/V4))+(AK2/2)
      AK3=((FNN*CT/V4)+Z2)*DT
      Z3=-(((F*BM34)+F+(F*RE4))*(CH/V4))+(AK3)
      AK4=((FNN*CT/V4)+Z3)*DT
      DCH=(AK1+2*AK2+2*AK3+AK4)/6
C
C
      CF=CF+DCF
      CS=CS+DCS
      CT=CT+DCT
      CH=CH+DCH
C
      C1(J)=CF
      C2(J)=CS
      C3(J)=CT
      C4(J)=CH
      T(J)=J*DT
C
 20   CONTINUE
C
      DO 80 LJ=1,N
        ITP=ITP+1
        IF(ITP.EQ.50)THEN
      WRITE(8,40)T(LJ),C1(LJ),C2(LJ),C3(LJ),C4(LJ)
 40   FORMAT(T2,F20.3,T25,F10.5,T45,F10.5,T65,F10.5,T75,F10.5)
        ITP=0
        END IF
 80   CONTINUE
      STOP
      END
```