REMOVING ALGAE FROM STABILIZATION POND EFFLUENTS BY USING TRICKLING FILTERS

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ABSTRACT

REMOVING ALGAE FROM STABILIZATION POND EFFLUENTS BY USING TRICKLING FILTERS

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The objective of this study is to remove turbidity originating from algae present in oxidation ponds effluents by an easy and inexpensive method. For this reason, a novel lab-scale **S**tep **F**eed **D**ual **T**reatment (SFDT) process was constructed and the efficiency of trickling filter (TF) to remove algae and organic matter was investigated. SFDT process developed in this study is the unique, inexpensive and new system to scavenge algae from oxidation pond effluents. In this system, influent is first treated in a stabilization pond, and subsequently they directed to a TF, so as to provide a dual treatment. Moreover, some fraction of the raw influent was directly sent to TF to maintain a steady biofilm on the TF medium. Stabilization pond was not simulated in the experimental set-up as the main objective of the study is to observe TF ability to scavenge algae from pond effluent. To determine the magnitude of the effect of individual operational parameters (hydraulic loading rate, influent COD and chlorophyll-a concentration) and of their combinations on organics and particle removal efficacy an experimental design was followed. Experiments consistent with two-level factorial design with three variables (2^3) were performed. Hydraulic loading rate (HLR) (0.5-2 m^3/m^2.day), influent COD (150-550 mg/l) and influent chlorophyll-a concentrations (Chl-a) (250-600 μg/l) were selected as independent variables. The COD and algae removal (as Chl-a) were selected as dependent variables. Data obtained from the experiments showed that when HLR (m^3/m^2.day) was increased from 0.5 to 2, Chl-a, NTU, SS and COD removals were decreased, however, more than 85 % removal was attained in each case, except for COD. The lowest removal efficiencies were obtained for all the quality parameters when hydraulic loading was increased to 4 m^3/m^2.day. It was observed that in general removal percentages for turbidity, Chl-a, SS and COD increased considerably with the decreasing hydraulic loading rate. Highest removals were obtained at lowest HLR. The removal of algae in TF was presumably due to both flocculation (due to algal and bacterial EPS production) and degradation (through bacterial activity) of algae. In conclusion, trickling filter produced clear effluents, with less than 2 NTU, for most of the cases.

Key words: algae removal, trickling filter, stabilization ponds,

ÖZ

DAMLATMALI FİLTRE KULLANARAK STABİLİZASYON HAVUZU ÇIKIŞ SULARINDAN ALG GİDERİMİ

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Bu çalışmada stabilizasyon havuzu çıkış sularında algden kaynaklanan bulanıklığı kolay ve ucuz bir şekilde gidermek amaçlanmıştır. Bu amaçla **Ç**ift **K**ademe **B**eslemeli **A**rdışık **A**rıtma (ÇKBAA) prosesi geliştirilmiş ve proses içinde damlatmalı filtrenin (DF) oksidasyon havuzu çıkış suyundan alg ve organik partikülleri gidermedeki verimliliği araştırılmıştır. ÇKBAA prosesi oksidasyon havuzu çıkış sularından alg giderimi amacıyla kullanılmak üzere bu çalışmada geliştirilen ucuz ve yeni bir sistemdir. Bu sistemde, stabilizasyon havuzuna gelen atıksu önce stabilizasyon havuzunda daha sonra da DF'de arıtılmakta, yani ardışık arıtıma tabii tutulmaktadır. Ayrıca, DF'de biyofilm oluşumunu sağlamak amacıyla, ham atıksuyun bir kısmı doğrudan DF'ye yönlendirilmektedir. Stabilizasyon havuzu deney düzeneğinde simule edilmemiştir çünkü çalışmanın amacı DF'nin havuz çıkış sularından alg giderim verimini araştırmaktır. Deneylerde, işletim parametrelerinin ayrı ayrı ve birlikte etkilerini gözlemek amacıyla, iki seviyeli, 3 değişkenli, (2^3), 8 deney setinden oluşan, tam faktoriyel deney matriksi oluşturulmuştur. Hidrolik yükleme hızı (HYH) (0,5 - 2 m^3/m^2.gün), giriş KOI (150-550 mg/l) ve klorofil-a konsantrasyonu (Chl-a) (250-600 µg/l) bağımsız değişkenler olarak; KOI ve alg giderimi ise bağımlı değişkenler olarak seçilmiştir. Elde edilen verilere göre; HYH 0,5'den 2 m^3/m^2.gün'e yükseltildiğinde Chl-a, NTU ve KOI giderimlerinde düşüş olduğu gözlenmekle birlikte, tüm yükleme hızlarında, KOI giderimi hariç diğer parametrelerde genellikle % 85 civarında bir giderim elde edilmiştir. HYH 4 m^3/m^2.gün' e yükseltildiğinde, tüm kalite parametreleri açısından en düşük verim elde edilmiştir. Genelde, HYH'nın azalmasıyla bulanıklık, Chl-a, AKM, ve KOI giderim verimlerinde önemli bir artış gözlenmiştir. En yüksek giderim verimleri en düşük HYH'ında elde edilmiştir. DF'de alg gideriminin hem flokulasyon (alg ve bakteri kaynaklı EPS'den dolayı) hem de bozulma ile sağlandığı düşünülmüştür. Sonuç olarak, ÇKBAA sistemi denemelerinin çoğunda berrak bir çıkış suyu, <2 NTU bulanıklık değerinde, sağlanmıştır.

Anahtar kelimeler: alg giderimi, damlatmalı filtre, stabilizasyon havuzları

To my family…

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ABBREVIATIONS

HLR	: Hydraulic loading rate, m^3/m^2.day
COD_{inf}	: Influent chemical oxygen demand, mg/L
Chl-a_{inf}	: Influent Chlorophyll-a concentration, μg/L
EFF	: Effluent
Inf	: Influent
SS	: Suspended solids concentration, mg/L
AVG	: Average
TF	: Trickling filter
SFDT	: Step Feed Dual Treatment
BOD	: Biochemical oxygen demand, mg/L

CHAPTER 1

INTRODUCTION

1.1. General

Waste stabilization ponds, or lagoons, provide an inexpensive alternative to conventional processes where the main objective of a wastewater treatment is not the removal of BOD but the removal of excreted pathogenic microorganisms which pose health threat due to a wide range of water-related diseases that wastewaters may harbour. However, ponds are still capable of producing an effluent with a low BOD and nutrient concentration (Mara and Pearson, 1998).

Lagoons, or stabilization ponds, treat wastewater through the use of sunlight, wind, algae, and oxygen. In stabilization ponds, wastewater enters the pond at a single point, either in the middle or at the edge. Algae grow in the pond by getting energy from the sunlight and using up the carbon dioxide and inorganics released by bacteria. Algae release oxygen as a by-product of photosynthesis for the use by bacteria (Tchobanoglous and Burton, 1991).

Waste stabilization or oxidation ponds have many advantages which can be summarized as:

- simple to operate -- effective operation with minimum of monitoring
- relative ease of commissioning -- given a relatively close source of suitable viable sludge

- low cost -- per volume of water treated wastewater or on capital and operating costs basis
- odour impact can readily be controlled by covering the anaerobic areas, or by designing aerobic or mildly facultative conditions, or by applying aeration
- sludge harvested from desludging operations is relatively stable and can be dried atmospherically without significant odor generation
- Wetlands can readily be coupled with lagoon systems for effluent polishing
- Lagoons can achieve high levels of disinfection.

On the other hand, some disadvantages of waste stabilization ponds can be listed as:

- Large foot step: require large land area for implementation
- Only about 80% maximum BOD removal is possible
- Lagoons are strongly affected by atmospheric conditions -- temperature is the greatest factor. The atmospheric temperature generally has a noticeable effect on the operating temperature of the ponds. The effect of temperature is most significant where the winter temperatures are very low.
- Unsightly and/or odorous scum accumulations may occur. Steps such as surface skimming or the addition of microbiological products can be taken to attempt to combat the problem.
- Some limitations appear to exist regarding nutrient removal in lagoons. In particular nitrogen removal in cold climates is a concern
- Effluents are extremely turbid character due to algal growth (Graae et al, 1998).

The major limitation of this treatment is the high concentrations of total suspended solids (TSS) in their effluent which is mainly due to high concentrations of algal cells in the finished effluent. The presence of such algae can impose serious constraints on effluent reuse potential, which is particularly important in water-scarce regions. Agriculture is perhaps the sector where effluent reuse is most widely practiced. The presence of algae in large quantities in the effluent is bound to create undesirable effects in water receiving bodies and in the irrigation networks, drip systems in particular (Saidam et al., 1995). Moreover, current re-use standards call for almost SS-free waters for agricultural reuse.

Since the pond effluents contain considerable amount of algae, they are generally unable to satisfy stringent water-quality standards for disposal or reuse for irrigation (Esen et al., 1991). As a yardstick of the effect that algae in suspension have on the COD of a pond effluent the following equation could be used as rough approximation: 100 μg/L of chlorophyll-a give rise to 5.6 mg COD/L (Meiring and Oellerman, 1995). Similarly, Shipin et al. (1999b) reported that every 100 μg/L in chl-a values represents 10 mg/L COD (3 mg/L BOD) values and 20 mg/L SS. The concentration of algae in a healthy facultative pond depends on loading and temperature, but is usually in the range 500-2000 μg chlorophyll-a per liter (Ramadan and Ponce, 2003) representing an effluent COD concentration of 28-200 mg/L. Therefore, it is obvious that algae need to be removed before discharge or reuse.

Upgrading of stabilization pond effluents by algal removal is a topic that received considerable attention in recent years. Algae may be removed by several methods, each of which is doubtful in economics and operation from practical point of view. The techniques, processes and operations, such as centrifugation, microstraining, coagulation-flocculation, rock filters, and etc. to scavenge algae from water, wastewater effluent and lakes have been discussed by Middlebrooks et al. (1974), Golueke and Oswald (1965), Berry (1961), Tenney et al. (1969), Friedman et al. (1977), Folkman and Wachs (1973) and by other researchers. Brief information about these techniques are given in the

Section 2.4. However none of these techniques found application in practice as they were either complicated, too costly or unreliable.
An additional, promising alternative for the removal of algae from waste stabilization pond effluents could be the trickling filter (TF) although they are conventionally designed for the removal of organic matter from wastewater. In contrast to systems where microorganisms are sustained in a liquid, and are thus known as suspended growth processes (Solomon et al., 1998); TF is an aerobic treatment system utilizing microorganisms attached to a media to remove organic matter from wastewater that passes over, around, through, or by the media. These systems are known as attached growth processes. TF system is a wastewater treatment technology that couples biological and mechanical filtration to effectively reduce BOD and TSS. TFs are capable of achieving BOD and TSS removal efficiencies greater than 80%, producing an effluent suitable for reclamation (landscape irrigation and soil conditioning). At an incremental cost, addition of other treatment components (e.g. wetlands, ponds and sand filters) boosts overall removal rates of BOD and TSS to more than 90%, creating a water source acceptable for human contact (Shipin et al., 1999a).

Therefore, potential use of TF as a system removing algae from the effluents of the stabilization ponds seemed worth studying.

1.2. Aim and scope of the Study

The aim of the study can be stated as:

- To test the ability of TF to scavenge algae from oxidation pond effluents.

To this purpose, an experimental set up was designed as to simulate a system with both TF and oxidation pond. This system was named as Step Feed Dual Treatment (SFDT). SFDT process is the unique, inexpensive and newly developed system to scavenge algae from oxidation pond effluents. In this system, raw wastewater (influent) firstly is treated in oxidation pond, and then effluent is directed to TF, hence, the name dual treatment. Moreover, using step feed application, a portion of oxidation pond influent, to help forming and maintaining biofilm on TF medium, is directly sent to TF together with oxidation pond effluent containing algae. Stabilization pond was not simulated in the experimental set-up as the main objective of the study is to observe TF ability to scavenge algae from pond effluent. For this purpose, an influent tank was used to feed TF within this system (Figure 3.1). Synthetic wastewater was put in this tank to simulate oxidation pond influent and algae were added for simulation of oxidation pond effluent.

Within the scope of this study, the trickling filter performance in SFDT was monitored in terms of turbidity, chlorophyll-a (Chl-a), suspended solids (SS) and chemical oxygen demand (COD) removals, under various operational conditions. The basis for their selection is given below.

Chl-a concentration in the effluent is important as it brings an important loading to receiving bodies and it reduces potential uses of these waters. Shipin et al. (1999b) found that every 100 μg/L in chl-a values represents 10 mg/L (3 mg/L) COD (BOD) values and 20 mg/L SS.

COD is parameter used to measure the content of organic matter of both wastewaters and natural waters. A significant oxygen demand is imposed on receiving waters by organic matters discharged through wastewaters

SS are discrete particles in suspension ranging from those which are easily settleable to the colloidal. Suspended material may be objectionable in water for several reasons (Peavy et al., 1985). High concentrations of suspended solids can cause many problems for aquatic life. High TSS in the water column can reduce light from reaching phytoplankton and rooted submerged vegetation. As the amount of light passing through the water is reduced, the rate of photosynthesis is reduced. Reduced rates of photosynthesis cause less dissolved oxygen to be released into the water by plants. If light is completely blocked from bottom dwelling plants, the plants will stop producing oxygen and will die. As the plants decompose, bacteria will consume even more oxygen from the water. Low dissolved oxygen can lead to fish kills. High TSS can also cause an increase in surface water temperature because the suspended particles absorb heat from sunlight. This can cause dissolved oxygen levels to fall even further and can harm aquatic life in many other ways (Mitchell and Stapp, 1992).

Criteria have been established for solids and turbidity primarily because of their effect on primary productivity by reducing light penetration, as well as for drinking water supplies (USEPA, 1987). The criterion for suspended solids (TSS) is that the depth of the compensation point for photosynthetic activity should not be reduced by more than 10 percent by TSS from its seasonally established norm.

The operational conditions studied were hydraulic loading rate (HLR), influent wastewater characteristics such as COD_{inf} and Chl-a_{inf} concentrations. Accordingly, an operating strategy for SFDT was tried to be developed.

CHAPTER 2

REVIEW OF LITERATURE

2.1. General

Wastewaters need to be treated prior to discharge or reuse to protect the public health and prevent ecological damage. In agricultural and aquacultural reuse final disinfection is mandatory to produce "microbiologically safe" effluents.

Both aerated lagoons and waste stabilization ponds are commonly used as efficient means of wastewater treatment relying on little sophisticated technology and minimal, albeit regular, maintenance. Their low capital and operating costs and ability to handle fluctuating organic and hydraulic loads have been valued for years in rural regions and in many tropical countries wherever suitable land is available at reasonable cost (Nameche and Vasel, 1998).

The micro-algae make a significant synergistic contribution to successful effluent treatment in oxidation ponds (Abeliovich, 1986; Oswald, 1988; Rose et al, 1992). Algae producing oxygen thus facilitate organics breakdown by bacteria and other components of the microbial consortium. Carbon dioxide and low molecular organics consumed by algae result in a photosynthetic conversion of a substantial portion of the organic load into algal biomass (Abeliovich and Weisman, 1978). The COD of the final effluent may be high with a large contribution by algal biomass in the form of filterable solids. Removal is problematic with a potential for nuisance in the form of secondary pollution by algal wastes and decay products (Shipin et al., 1998).

2.2. Waste Stabilization Ponds (WSP)

Waste stabilization ponds are the simplest of all waste treatment techniques available for sewered wastewaters. Adopting as low a level of treatment as possible is especially desirable in developing countries, not only from the point of view of cost but also in acknowledgement of the difficulty of operating complex systems reliably. In many locations it will be better to design the reuse system to accept a low-grade of effluent rather than to rely on advanced treatment processes producing a reclaimed effluent which continuously meets a stringent quality standard (Ramadan and Ponce, 2003).

Waste stabilization ponds (WSP), often referred to as oxidation ponds or lagoons, are include basins used for secondary wastewater (sewage effluents) treatment where decomposition of organic matter is processed naturally, i.e. biologically. The activity in the WSP is a complex symbiosis of bacteria and algae, which stabilizes the waste and reduces pathogens. As a result of this biological process the organic content of the effluent is converted to more stable and less offensive forms. WSP are used to treat a variety of wastewaters, from domestic wastewaters to complex industrial waters, and they function under a wide range of weather conditions, i.e. tropical to arctic. They can be used alone or in combination with additional treatment processes (Ramadan and Ponce, 2003).

A WSP is a relatively shallow body of wastewater contained in an earthen man-made basin into which wastewater flows and from which, after certain retention time elapsed a treated effluent is discharged. Many characteristics make WSP substantially different from other wastewater treatment. This includes design, construction and operation simplicity, cost effectiveness, low maintenance requirements, low energy requirements, easily adaptive for upgrading and high efficiency (Ramadan and Ponce, 2003).

A World Bank Report (Shuval et al., 1986) endorsed the concept of stabilization pond as the most suitable wastewater treatment system for effluent use in agriculture.

WSP can be classified in respect to the type(s) of biological activity occurring in a pond. There are three types of WSPs: anaerobic, facultative and maturation ponds. Anaerobic and facultative ponds are designed for BOD removal and maturation ponds for pathogen removal, although some BOD removal occurs in maturation ponds and some pathogen removal in anaerobic and facultative ponds. In many instances only anaerobic and facultative ponds are required. In general, maturation ponds are required only when stronger wastewaters (BOD > 150 mg/l) are to be treated prior to surface water discharge and when the treated wastewater is to be used for unrestricted irrigation (irrigation for vegetable crops). Generally, in WSP systems, effluent flows from the anaerobic pond to the facultative pond and finally, if necessary, to the maturation pond (Ramadan and Ponce, 2003).

2.2.1. Aerobic Ponds

They can be either naturally aerobic or artificially aerated. Naturally aerobic ponds are shallow ponds of about 0.3 m depth or less so designed as to maximize light penetration and the growth of algae through photosynthetic action (Arceivala, 1998). Dissolved oxygen is present throughout much of the depth of aerobic lagoons. They tend to be much shallower than other lagoons, so that sunlight and oxygen from air can better penetrate into the wastewater. In general, they are better suited for warm, sunny climates, where they are less likely to freeze. Wastewater usually must remain in aerobic lagoons from 3 to 50 days to receive adequate treatment. Such ponds are useful where ultimate harvesting of algae is desired, but their use in waste treatment has not been widespread (Arceivala, 1998).

Aerated lagoons are common in small communities. These systems use aerators to mix the contents of the pond and add oxygen to the wastewater. They are sometimes referred to as partial-mix or complete-mix lagoons depending on the extent of aeration. Aeration makes treatment more efficient, which offsets energy costs in some cases. Aerated lagoons require smaller foot print and shorter detention times, as compared to the algal ponds.

2.2.2. Anaerobic Ponds

Anaerobic ponds (AP) are deep treatment ponds that exclude oxygen and encourage the growth of bacteria, which break down the effluent, releasing methane and carbon dioxide. Sludge is deposited on the bottom and a crust forms over the surface (Ramadan and Ponce, 2003).

Anaerobic ponds are commonly 2-5 m deep and receive such a high organic loading (usually > 100 g BOD/m^3.d equivalent to > 3000 kg/ha/d for a depth of 3 m). They receive an organic loading which is much higher relative to the amount of oxygen entering the pond, which maintains anaerobic conditions. Anaerobic ponds don't contain algae, although occasionally a thin film of mainly *Chlamydomonas* can be seen at the surface. They work extremely well in warm climate (can attain 60-85% BOD removal) and have relatively short retention time (for BOD of up to 300 mg/l, one day is sufficient at temperature > 20^{o}C) (Ramadan and Ponce, 2003).

Anaerobic ponds (AP) reduce N, P, K and pathogenic microorganisms by sludge formation and the release of ammonia into the air. Arridge et al. (1995) found a one log removal in the APs for each of the following indicators: faecal coliforms, faecal streptococci and Clostridium perfringens. The Salmonellae were reduced from 130 to 70 MPN/100 ml and Vibrio cholerae 01 was reduced from 40 to 10 MPN/l respectively. Anaerobic ponds appear to be essential for high levels of V. cholerae removal. In another study, Arridge et al. (1995) reported the removal of one log unit for rotaviruses in the anaerobic pond in an experimental WSP complex.

Physical as well as chemical factors affect the habitat of microorganisms and consequently the anaerobic sewage treatment process. The most important environmental factors to take into consideration are: temperature, pH, degree of mixing, nutrient requirements, ammonia and sulphide control and the presence of toxic compounds in the influent (Ramadan and Ponce, 2003).

In order to have a reasonable methane production rate, the temperature should be maintained above 20°C. Methane production rates are doubled for each 10°C temperature increase in the mesophilic range (Droste, 1997).

The optimum pH range for all methanogenic bacteria is between 6 and 8, but the optimum value for the group as a whole is close to 7 (Zehnder et al., 1982).

2.2.3. Facultative Ponds

Facultative waste stabilization ponds are partly aerobic and partly anaerobic. They are often about 1-2 m deep and are of two types: primary facultative ponds, which receive raw wastewater, and secondary facultative ponds, which receive settled wastewater (usually the effluent from anaerobic ponds) (Arceivala, 1998). They are designed for BOD removal on the basis of a relatively low surface loading, 100-400 kg BOD/ha.d at temperatures between 20°C and 25°C) to permit the development of a healthy algal population as the oxygen for BOD removal by the pond bacteria is mostly generated by algal photosynthesis. Due to the algae, facultative ponds are coloured dark green, although they may occasionally appear red or pink (especially when slightly overloaded) due to the presence of anaerobic purple sulphide-oxidizing photosynthetic bacteria. The algae that tend to predominate in the turbid waters of facultative ponds are the motile genera, such as *Chlamydomonas, Pyrobotrys* and *Euglena*, as these can optimize their vertical position in the pond water column in relation to incident light intensity and temperature more easily than non-motile forms (such as *Chlorella*, although this is also fairly common in facultative ponds). The concentration of algae in a healthy facultative pond depends on loading and temperature, but is usually in the range 500-2000 µg chlorophyll-a per litre (Ramadan and Ponce, 2003).

Aerobic pond is more accurately termed "facultative", as in practice the pond usually has an aerobic upper layer and anaerobic lower layer (Figure 2.1). This facultative condition occurs because high oxygen levels cannot be maintained throughout the total depth of aerobic ponds. So a fully aerobic surface layer develops, along with an aerobic/anaerobic intermediate layer, and a fully anaerobic layer on the pond bottom (Ramadan and Ponce, 2003).

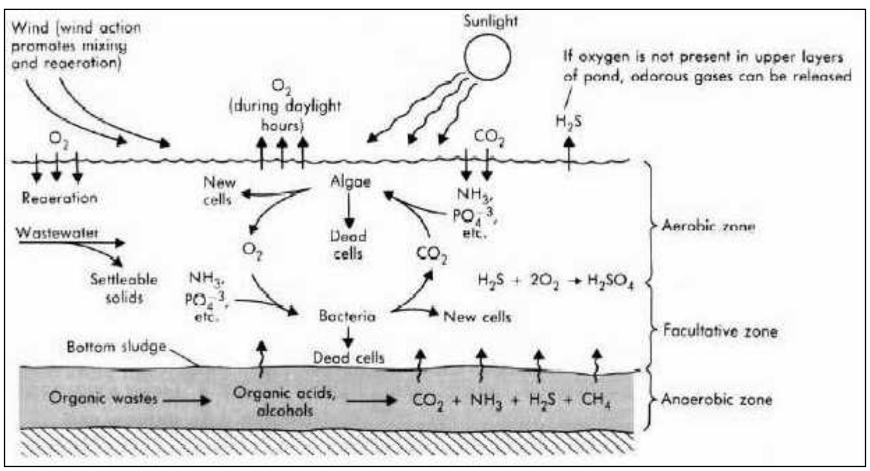


Figure 2.1. Schematic representation of a facultative waste stabilization pond (Tchobanoglous and Burton, 1991).

Faecal bacteria are mainly removed in facultative and especially in maturation ponds whose size and number determine the numbers of faecal bacteria in the final effluent. The principal mechanisms for faecal bacterial removal in facultative and maturation ponds are now known to be, time (retention time as pathogen attenuation occurs over time); temperature (faecal bacteria die off increases with temperature); high pH (> 9) and high light intensity together with high dissolved oxygen concentration.

Regarding viruse removal, little is known about the mechanisms of viral removal in WSPs, but it is generally recognized that it occurs by adsorption on to settleable solids, including the pond algae, and consequent sedimentation (Ramadan and Ponce, 2003). Some parasites can be removed as well. Protozoan cysts and helminth eggs are removed by sedimentation (Ayres et al., 1992).

2.2.4. Problems of Waste Stabilization Ponds

Recently, analyses of ponds effluents have pointed out that effluent containing algae can impose a significant oxygen demand on receiving waters. Theoretical calculation of the oxygen demand created by algae destruction suggests that 1.58 mg O_2 are required to oxidize 1 mg of algae (dry weight) to CO_2 and H_20 (Friedman et al., 1977). Bare et al. (1975) found that 1.11 mg O_2 was required per mg of algae and obtained from laboratory studies and 0.81 mg O_2 per mg algae obtained from field studies. Varma and Digiano (1968) found that 0.67 mg O_2 was required for each mg of algae destroyed. Data resulting from algae obtained in both laboratory and field experiments during this study indicate an average oxygen demand of 1.19 mg O_2 per mg algae.

The coliform organisms which may be present in the effluent are another problem. Routine disinfection practices can lower the bacterial population to acceptable levels. However, when algae cells are present the use of chlorine as a disinfectant is counterproductive, because the side reactions which occur increase both the chlorine demand and the concentration of soluble organic material present in the effluent (Echelberger et al., 1971).

Friedman et al. (1977) reported that despite the cited differences in oxygen demand per unit weight of algae one can conclude that a significant oxygen demand is imposed on receiving waters by algae discharged from waste stabilization ponds. ***Only the removal or destruction of algae prior to discharge can sufficiently improve effluent quality so that waste stabilization ponds can be incorporated into low cost wastewater treatment systems***. Some means of algae removal will have to be employed with existing ponds or they will have to be replaced with expensive conventional wastewater treatment systems to meet current effluent standards.

2.3. Algae in Stabilization Ponds

To understand the nature of algae is very important for ensuring the effective performance of waste stabilization ponds.

Microalgae are microscopic, single-celled photosynthetic organisms, growing in aqueous environment. Microalgae converts sunlight, CO_2 and nutrients like nitrates and phosphates into proteins, lipids, and carbohydrates, pigments and specific bioactive substances.

Microalgae represent an immense range of genetic diversity and can exist as unicells, colonies and extended filaments. They are ubiquitously distributed throughout the biosphere and grow under the widest possible variety of conditions.

Algae can be described as hydrophilic biocolloids with apparent negative surface charges (Ives, 1959; Tenney et al., 1969). In addition, their small size, 3 to 15 μ, and low specific gravity further complicate physical removal processes (Friedman et al., 1977).

Algal biomass is usually measured by the amount of chlorophyll-a in the water. Chlorophyll-a is a photosynthetic pigment that serves as a measurable parameter for all phytoplanktonic production. On average, 1.5% of algal organic matter is chlorophyll-a (Raschke, 1993).

The green (*Chlorophyta*) and blue-green (*Cyanophyta*) algae are commonly found in waste stabilization. Typical of the green algae in stabilization ponds are: *Chlorella, Scenedesmus, Chlamydomonas, Chlorococcum, Chlorogonium, Coelastrum, Gonium, Ankistrodesmus, Micractinium, Actinastrum, Eudorina, Pandorina* (Arceivala et al., 1970).

Among the blue-green algae common to waste stabilization ponds are: *Oscillatoria, Spirulina (Arthospira*), *Phormidium, Merismopedia, Anabaena, Anacystis* (*Microcystis*), *Aphanizomenon.*

Some flagellates commonly found are: *Euglena*, *Phacus, Trachelomonas.*

The most common yellow-green algae (diatoms) are: *Navicula, Cyclotella, Asterionella, Synedra, Tabellaria, Melosira, Fragilaria.*

Blue-green algal mats frequently develop in ponds during summer months. *Euglena* show a high degree of adaptability to various pond conditions and are present during all seasons and under most climatological conditions. Probably next in the adaptability are *Chlamydomonas*, *Ankistrodesmus, Scenedesmus*, and *Chlorella*. *Chlorella* is the most desirable alga in waste stabilization ponds as it has the maximum oxygen donation capacity (Arceivala et al., 1970).

Algal photosynthetic rate is heavily dependent on light climate, which varies with pond depth. Some blue-green algae contain gas vacuoles within their cells and can therefore control their buoyancy and position in the water column and thus have a further growth advantage over other genera. Other algae that are motile can swim to adjust their position; however non-motile genera inevitably sink because their densities are greater than water. The rate of sinking is directly related to algal cell size, with larger cells sinking faster. It has been demonstrated that increased surface area due to cellular features increases friction and therefore the likelihood of entrainment within the water column (South and Whittick, 1987).

Green algae generally reach higher maximum growth rates than *Cyanobacteria*, however in low light intensities blue-green algae can achieve higher growth rates because they require little energy to maintain cell function and structure (Mur et al., 1999). The small size of the *Cyanobacteria* and some green algae improve their nutrient assimilation capacity and thus survival. The rapid growth of green algae, allows them to out-compete other algae, however the environmental conditions can become limiting; with increased numbers algae induce self-shading and faster depletion of nutrients (Sandgren, 1988). This is evident in the lagoons, with green algae dominating the genera, specifically *Chlamydomonas* and *Scenedesmus*, which are both very small cells, and presumed to have fast growth rates. *Cyanobacteria* have a competitive advantage over green algae in poor growth conditions, such as high turbidity.

The algal cell contains on an average 50 to 60 % protein, 20 to 30 % fat and 10 to 20 % carbohydrates besides amino acids and vitamins (Arceivala et al., 1970). The empirical formula of *Chlorella* is $C_7H_{8.1}O_{2.5}N_1$ and the empirical formula of *Euglena* is $C_{7.62}H_{8.08}O_{2.53}N_1$. Besides the elements indicated in the formulae, algae also contain small amounts of phosphorus, sulphur and traces of other elements (Arceivala et al., 1970).

During photosynthesis, energy from light is absorbed by the chlorophyll, a green pigment in algae, and through a series of reactions, is transformed in chemical energy which is stored in the molecules of algae. In other words, the production of new algae by photosynthesis is accomplished by the utilization of energy and the release of oxygen (Varma and Digiano, 1968).

The overall equation representing the synthesis of algal cell material and release of oxygen can be represented in Equation 2.1 as follows:

$$aCO_2 + (0.5b\text{-}1.5d)\ H_2O + dNH_3 \longrightarrow \underset{\text{(Algae)}}{C_aH_bO_cN_d} + (a\text{+}0.25b\text{-}0.75d\text{-}0.5c)\ O_2 \qquad (2.1)$$

The production of oxygen during photosynthesis depends, therefore, on the coefficients a, b, c and d and vary from one species to another. If the ratio of weight of oxygen released to the weight of algae synthesized is designated as p, then

$$p = \frac{(a + 0.25b - 0.75d - 0.5c)O_2}{C_a H_b O_C N_d} \qquad (2.2)$$

The values of p reported by different workers vary from 1.25 to 1.75 for algae in waste stabilization ponds. In other words, the formation of one gram of algal material is associated with the production of 1.25 to 1.75 g of oxygen.

In the absence of sunlight, algae are able to carry on metabolism like bacteria. Algae require oxygen for respiration. The demand by algae for oxygen in the absence of sunlight is of as much importance as their production of oxygen in its presence. There seems to be a mechanism in the algae for conserving energy by alternating the dominance between photosynthesis during the day and respiration at night (Arceivala et al., 1970).

High intensity of sunshine is not always necessary to increase algal production. For a species like *Anacystis*, the saturation level is below 1000 foot-candles or less than 1/10 of full sunlight. The light saturation intensity for *Chlorella* is 600 foot-candles only, whereas for *Euglena* the optimum is 2000 foot-candles. It can, therefore, be stated that very high intensity light is not needed for successful waste treatment (Arceivala et al., 1970).

Temperature affects photosynthetic oxygenation as well as bacterial decomposition of organic matter. The favourable range of temperature for photosynthesis appears to be about 20-40 °C. If the water temperature approaches 35 °C, the growth of beneficial algae tends to decline. Green algae belonging to the group of *Chlorophyta* will decrease or disappear. The dominant algae remaining in the ponds when temperature exceeds 35°C are blue-green algae and the euglenoid algae (Arceivala et al., 1970).

2.4. Alternative Removal Techniques of Algae

There are a lot of techniques, processes and operations to remove algae from water, wastewater effluent and lakes. Algae can be removed by many methods, and every method of algae removal from ponds has specific advantages and disadvantages, but the methods selected must be specific to the particular treatment situation (population, land costs, discharges, water uses, and so forth) (Middlebrooks, 1975). The processes and operations discussed by various researchers are: centrifugation, microstraining, coagulation-flocculation-floatation-oxidation, in-pond removal of particulate matter, oxidation ditches, dissolved air flotation, granular media filtration, intermittent sand filtration, algae removal with clays, and rock filters. Brief information about these processes especially specifying their disadvantages are below.

2.4.1. Centrifugation

Pilot plant experiments on wastewater-grown algae indicate that centrifugation is an effective means for dewatering algal sludge (Golueke and Oswald, 1965). However, there are some disadvantages of this process. The principal disadvantage of centrifugation is the temperamental nature of the equipment. Abrasive solids in the water can cause rapid deterioration of the scroll. Operating problems associated with the relatively sophisticated equipment take up a considerable amount of a skilled operator's time (Middlebrooks et al., 1974). Since one of the advantages of stabilization ponds is the low operating expense, it would appear to be impractical to couple these advantages with a unit process having major disadvantage of high operating costs (Middlebrooks et al., 1974; Sim et al, 1988). Finally, centrifuge performance responds sensitively and rapidly to changes in loading and sludge properties; major performance problems occur rapidly and without warning. (Rittmann and McCarty, 2001)

2.4.2. Microstraning

Microstraining is a form of simple filtration (Middlebrooks et al., 1974). Microstrainers being low speed (up to 4 to 7 rpm) rotating drum filters operating under gravity conditions have been used to remove algae from water in reservoirs before treatment, to remove solids from industrial wastes, and to polish activated sludge effluents (Berry, 1961; Turre and Evans, 1959; Golueke and Oswald, 1965)

However, microstraining has proven notably unsuccessful in a number of tests on oxidation pond effluent, primarily because desirable species of pond algae tends to be smaller than 10 μ (such as Chlorella and Scenedesmus), whereas the smallest-sized microstrainer opening that has been commercially available is 23 μ (Parker and Uhte, 1975). However, a polyester fabric with a 1 μm mesh size has since been developed, and it appears that microstrainers equipped with this fabric are capable of producing an effluent with BOD and SS concentrations of less than 30 mg/l. Microscreen manufacturers are promoting the use of the 1 μm screen with the returned of the filtered algae to the pond. The service life of the screens is reported to be about 1.5 years, which is considerably less than the manufacturer's prediction of 5 years. Difficulty with screen binding and short run times was experienced with Camden system, South Carolina in December 1981 (Middlebrooks, 1995).

The main problems of the process are incomplete solids removal and difficulty in handling solids fluctuations. These problems may be partially overcome by varying the speed of rotation. In general, drum rotation should be at the slowest rate possible, that is, consistent with the throughput, and should provide an acceptable head differential across the fabric (Middlebrooks et al., 1974). Another problem associated with microstrainers is the build up bacterial and algal slime on the microfabric. This growth may be inhibited by installing high intensity ultraviolet irradiation equipment. Therefore, microstrainers may require periodic cleaning (Middlebrooks et al., 1974; Berry, 1961). *The majority of evidence supports that conventional microstraining has no practical application for algae removal from oxidation pond effluents* (Parker and Uhte, 1975).

2.4.3. Coagulation-Flocculation-Floatation-Oxidation

The coagulation of wastewater has been studied by various investigators. A review of the literature suggested that chemical coagulation and flocculation followed by sedimentation or flotation would be the most likely methods of achieving algae removal under the conditions encountered in most wastewater lagoons. Lime (Folkman and Wachs, 1973; Friedman et al., 1977; Ayoub et al., 1986), alum (Abo-Elela et al., 1988), and ferric salts are the most commonly used coagulating agents. Floc formation is sensitive to parameters such as pH, alkalinity, turbidity and temperature. Most of these parameters have been studied, and their effects on removal of turbidity have been evaluated (Golueke and Oswald, 1965; Tenney et al., 1969; Divakaran and Pillai, 2002).

Ma and Liu (2002) conducted jar tests to evaluate the effectiveness of potassium ferrate preoxidation on algae removal by coagulation. In their studies, they used algae-bearing lake water and cultured algae solution and demonstrated that pre-treatment with potassium ferrate obviously enhanced the algae removal by coagulation-sedimentation process with alum [$Al_2(SO_4)_3.18H_2O$]. Ferrate preoxidation inactivated algae, and also induced coagulant aid secreted by algal cells.

Floating algae blankets have been reported in some cases in the presence of chemical coagulants (Montiel and Welte, 1998; Shindala and Stewart, 1971;

Friedman et al., 1977; Van Vuuren et al. 1965; Van Vuuren and Van Duuren, 1965). Van Vuuren et al. (1965) and Van Vuuren and Van Duuren (1965) separated algae from pond effluents by gas bubble flotation using alum as the flotation aid. About 50 percent of the algae removed were skimmed from the surface.

The effect of the combining use of contact coagulation with flotation (with or without preozonation of the water) to remove algae was also investigated by various researcher (Montiel and Welte, 1998; Betzer et al., 1980; Richard and Dalga, 1993; Richard, 1993). They found that preozonation increased the removal of algae.

However, treatment chain used in the studies contains several processes which are very complicated and expensive such as ozonation, GAC filtration and disinfection. Moreover, coagulation-flocculation is not easily controlled and requires expert operating personnel at all times. It produces a large volume of sludge, and this introduces an additional operating problem that would very likely be ignored in a small community that is accustomed to a minimum operation and maintenance of a wastewater lagoon. *For these reasons, coagulation-flocculation appears infeasible for application in small communities* (Middlebrooks et al., 1974).

2.4.4. In-Pond Removal of Particulate Matter

Specific in-pond mechanisms of particulate removal include complete containment, biological disks, baffles or raceways, chemical additions for precipitation (Golueke and Oswald, 1965), autoflocculation (Humenik and Hanna, 1970), and biological harvesting (Middlebrooks et al., 1974).

Precipitation of algae may be accomplished by adding chemicals which bring about the formation of settleable insoluble gelatinous hydroxide particles in which algae are enmeshed, or by inducing a change in the surface characteristics of the individual algal cells so that the cells agglomerate into settleable floc (Golueke and Oswald, 1965).

Autoflocculation of algae has been observed during some studies. The phenomenon consists of a naturally caused flocculation and settling of the algae under certain conditions. The required conditions for the occurrence of this phenomenon are an actively photosynthesizing shallow culture, a relatively warm day, and sunlight (Gloueke and Oswald, 1965; McGriff and McKinney, 1971; McKinney 1971). Laboratory studies have indicated that it is possible to bioflocculate algae and remove the flocculated mass so that relatively solids-free effluent is produced (Humenik and Hanna, 1970)

Regarding the biological harvesting of algae, the suggestion that fish and/or higher plant-consuming vertebrates might effectively remove algal material has proven largely unsuccessful. One of the most significant problems with the use of these organisms to harvest particulate matter is the excretion of their own fecal matter, which in turn later decomposes and results in higher BOD in the effluent than would be permitted. The practical aspects of keeping the biological harvesters in good growing conditions under varying temperature and flow and oxygen regimes are further reasons why this technique has not been acceptable; it is a practical concept; as a treatment step, however, it seems unusable. Biological harvesting is also costly and relatively ineffective and requires considerable management and operating skill (Middlebrooks et al., 1974).

None of these biological systems, however, functions perfectly. They still produce a certain amount of particulate matter from the breaking off and fractioning of filamentous microorganisms. The biological systems have an added disadvantage in that they are relatively expensive; the disks and baffles require a considerable expenditure for construction, although this might be compensated for in part by the added rate of treatment and, hence, higher loadings. The raceway require either pumping or gravity flow construction. Both methods require considerable capital investment, although the former has lower operating costs. Whereas an attached growth system has the advantage of requiring little maintenance in terms of the biological operations, its initial cost, subsequent treatment requirement, and unproven capability seem to preclude serious consideration (Middlebrooks et al., 1974).

In general, Middlebrooks et al. (1974) presented several problems related to the in-pond removal of particulate matter;

1) The subsequent decay of settled matter and degradation by microorganisms to produce dissolved BOD, which would then have an effect on the receiving water;
2) The possibility that settled material will not remain settled;
3) The lack of positive control of effluent particulate matter;
4) The problem of eventually filling in the oxidation pond; and
5) The possibility that anaerobic reactions within the settled material will produce malodours.

2.4.5. Oxidation ditches

Oxidation ditches may be considered as a method of in-pond removal of particulate matter. Generally, the oxidation ditch does not develop a significant algal growth that remains in suspension; hence, algal removal is not a problem. Although this method of waste treatment may be feasible, two disadvantages noted are that operational costs are involved, as well as significant space requirements; and little operating experience is available to predict performance. Therefore, this process does not appear to be a good choice for lagoon effluent polishing. Oxidation ditches also require considerable operation and maintenance (Middlebrooks et al., 1974).

2.4.6. Dissolved Air Flotation

Another method, namely, dissolved air flotation involves the mechanical saturation of dissolved air in a portion of the liquid stream (influent or effluent recycle). The release of the dissolved gases to form fine bubbles in the influent stream while adding alum or other coagulants allows the separation of suspended materials to take place by floatation (Parker et al., 1973; Bare et al., 1975; Nurdogan, 1985; Wang and Krofta, 1985; Chen et al., 1998; Martyn et al., 2004). To increase the particulate size of algal biomass, which is important for this method, flocculants are added to bind the cells together to facilitate settling. Air bubbles, passed into the solution, will adhere themselves onto the particulate mass, thus increasing the buoyancy and causing the algal particles to float to the surface where a compaction zone is formed. A common problem encountered with dissolved air flotation systems is that oversized bubbles break up the floc. To achieve the required size of air bubbles, a saturation tank is necessary to obtain a supersaturated solution of air in water. The algal float which is formed on the surface of the floater and which would be allowed to stand for a period of time is intermittently scraped into a collection through, while the clarified water usually flows out via a weir discharge (Sim et al., 1988).

Dissolved air floatation does not satisfy the basic requirements of simplicity and ease of operation that would be necessary in small communities. Also, the process is relatively expensive, and addition of coagulants is required to obtain successful algae removal (Middlebrooks et al., 1974). In terms of waste treatment, the DAF system for harvesting algae combined with chitosan as flocculant appeared to be superior to that of centrifugation. Comparison of operating cost indicated that DAF system using chitosan or alum as the flocculant was less expensive than centrifugation (Sim et al., 1988). However, the coagulant addition significantly increases the bulk of the sludge to be removed and dewatered (Middlebrooks et al., 1974).

2.4.7. Granular Media Filtration

Granular media filtration, the most overlooked unit operation for upgrading wastewater, is generally used for liquid-solids separation. The simple design and operation of this process make it applicable to wastewater streams containing up to 200 mg/L SS. Automation based on easily measured parameters results in minimum operation and maintenance costs.

In granular media, the wastewater passes through one or more layers of medium such as coal, sand, or garnet. As the wastewater flows through the granular material, the suspended material is removed by physical screening, sedimentation, and interparticle action. The liquid head loss increases until the filter reaches its removal capacity. At this point, the wastewater flow is stopped, and the filter must be cleaned (Middlebrooks et al., 1974).

Middlebrooks et al. (1974) reported that granular media filtration offers the advantages of automatic operation, sparkling clear effluent, and nutrient removal. They also reported the process as economically feasible in the removal of algal cells from lagoon effluents. However, direct filtration of algae-laden waters without coagulant addition has resulted in relatively poor removals of SS (less than 50%) and short run lengths (Borchardt and O'melia, 1961; Davis and Borchardt, 1966; Foess and Borchardt, 1969). Also, planktonic green algae are not removable by granular media filtration or microstraining because of their small size and negative charge. For a filter to be effective, the bulk of the algae must be removed before reaching the filter, and the remaining suspended matter must be well coagulated (Parker and Uhte, 1975).

2.4.8. Sand Filtration

Sand filtration is employed in water treatment for the removal of suspended solids present in surface waters (Naghavi and Malone, 1986). Many investigators such as Camp (1964) and O'Melia and Stumm (1967) discussed the various factors which may play important roles in suspended solids removal. The dominant mechanisms depend on the physical and chemical characteristics of the suspended solids, the filter media, the rate of filtration, and the physical and chemical properties of the water.

Esen et al. (1991) studied the efficiency of slow sand filtration in removing algae from high-rate pond effluents. It was found that the slow sand filtration system was found efficient in removing all types of algae from the integrated pond effluents, producing crystal-clear water with turbidity less than 3 NTU, which is lower than most drinking water.
Sand filtration of the algae performed by Borchardt and O'Melia (1961) and Davis and Borchardt (1966) indicate the importance of grain diameter in removing algae. Smaller sand diameters resulted in a better removal efficiency.

Naghavi and Malone (1986) investigated the feasibility of filtering algae from water without chemical coagulation using fine sand/silt as the filter media. They found that one of the major parameters influencing the operation of this filtration system was the amount of algae removed from the water. The major effect of influent algal concentration on the filtration technique shortened the run time. They concluded that run time was greatly affected by influent algal concentration. Higher influent algal concentrations resulted in shorter runs at a fixed headloss. Other investigators indicated that the effect of grain size diameter became greater at higher algae concentrations, and retention of algae occurred mostly at the upper part of the sand columns (Folkman and Wachs, 1970).

The practical problem associated with filtering algae-bearing waters is rapid clogging and fouling of the media with microbial growth.

2.4.9. Algae Removal with Clays

Han and Kim (2001) investigated the mechanism of clay and algae collision and algal removal thereafter by the clay spreading method. A set of trajectory analyses was performed to calculate the collision efficiency of clay particles and algal cells. The sensitivity of the removal efficiency to the physical characteristics of both clay and algae was investigated. It was found that both particle/cell size and zeta potential of clay and algae are the most important parameters that control removal efficiency. Selection of the appropriate particle size of clay in relation to the size of algal cells was found to be most important. They found that clay particle size should be similar to the algal cell size to obtain the best removal efficiency. In lakes with low ionic strength, the collision efficiency was determined to be very low and almost no removal is expected.

2.4.10. Rock Filters

An additional alternative for the removal of algae from lagoon effluents is the rock filter. A rock filter consists of a submerged bed rocks (5 -20 cm diam) through which lagoon effluent are passed horizontally; this allows the algae to settle out on the rock surface as the liquid flows through the void spaces. The accumulated algae are then biologically degraded (Middlebrooks, 1995; Swanson and Williamson, 1980).

The principal advantages of the rock filters are their relatively low construction costs and simple operation.

In their study, Mara and Johnson (2003) found that BOD and SS removal in the rock filters were inversely proportional to HLR. The rock filters treating maturation pond effluents produced high quality effluents with mean BOD and SS concentrations of <20 mg/L and <30 mg/L, respectively. Also, ammonia was not removed in any of the filters.

Rock filter performance and design have varied widely. In his study, Middlebrooks (1988) presented descriptions of rock filter experiments and several rock filters which were in operation at that time, and a summary of performance data.

Algae removal with rock filters were also studied at Eudora, Kans. Two experimental rock filters, with rock diameters of 1.3 cm and 2.5 cm, were studied, at a submerged rock depth of 1.5 m (O'Brien and McKinney, 1979). Filters were operated at loading rates for up to 1.2 m^3/m^3.d in the summer; and were decreased to 0.4 m^3/m^3.d in winter and spring. Tests on pond effluent with a BOD_5 of 10 to 35 mg/L and TSS level of 40 to 70 mg/L showed that filter reduced BOD_5 by only a relatively small amount and would usually reduce TSS to 20 to 40 mg/L. It was concluded that the rock filter could meet effluent requirements of 30 mg/L BOD_5, but could not consistently reduce the TSS to 30 mg/L. O'Brien and McKinney (1979) reported that rock filters provide limited reductions in effluent suspended solids at hydraulic loadings of 0.5 m^3/m^3.d–3.0 m^3/m^3.d. It was postulated that the filter would not become plugged for more than 20 years.

A 757 m^3/d full-scale rock filter was constructed in Veneta, Oreg. in 1975 (Swanson and Williamson, 1980). Hydraulic loadings on this rock filter varied from 0.05 m^3/m^3.d to 0.30 m^3/m^3.d, an order of magnitude lower than those researched by O'Brien and McKinney (1979). The Veneta rock filter was designed differently from the one at Eudora. Influent entered through a pipe that was on the bottom and passed through the center of the filter. Water rose through 2 m of rock (7.5 to 20 cm in diameter) and was collected in effluent weirs on the sides of the rock filter. As a conclusion, the Veneta rock filter consistently met daily maximum effluent limits of 20 mg/L TSS for hydraulic loading of 0.3 m^3/m^3.day. Swanson and Williamson (1980) found that BOD and SS removal in the rock filters were inversely proportional to HLR which agrees with the results reported by Mara and Johnson (2003). The settling tests on lagoon samples and the performance analysis and physical observation of the Veneta rock filter all confirmed that sedimentation was the primary mechanism by which algae, particularly green algae and flagellates, were removed within the rock filter. Also, most algae, including green algae, diatoms, and flagellates settled to the nearest rock surface. Blue-green algae, which can form gas vacuoles, rose and were trapped on the bottom of the rocks.

Another pilot-scale study was carried out by Oran (1993) to improve wastewater characteristics of a full-scale oxidation pond effluent. Treatibility studies were evaluated by using two types of rock media (Media$_1$: having a density 2.532 gr/cm^3, a porosity of 0.473, and a total volume of 0.032 m^3 and Media$_2$: having a density 2.703 gr/cm^3, a porosity of 0.447, and a total volume of 0.033 m^3), three surface overflow rates (2.053, 1.026 and 0.513 m^3/m^2.day) and six detention times (0.395, 0.790 and 1.580 day) were employed. It was observed that effluent values and concentrations of turbidity, Chl-a, SS and COD decreased considerably with decreasing surface overflow rate (SOR) for a fixed media. There were direct relationships between duration of filtration and removal efficiencies of turbidity, Chl-a, SS and COD for a fixed media and SOR. At all the loading rates and rock sizes, as filtration period increased, values and effluent concentrations of four quality parameters dropped, thus removal efficiency increased. The highest removal efficiencies, 64% for Chl-a, 72% for turbidity, 60% for SS, and 54% for COD were obtained when the media was Media$_2$, SOR was 0.513 m^3/m^2.day.

However, rock filters have lower removal efficiencies (Oran, 1993; Saidam et al., 1995) and odor problems can occur, and the design life for filters and the cleaning procedures have not been firmly established. High ammonia nitrogen concentrations in rock filter effluents could limit application of the process. Another drawback of rock filters is the production of hydrogen sulfide during summer and early fall when the filters becomes anaerobic. Effluent aeration would be required before discharge in many cases. The rate of sludge accumulation in the voids of the rock remains unknown (Middlebrooks, 1995).

2.5. Trickling Filters

Trickling filters are one of the oldest wastewater treatment processes used in the U.S. and around the world. These were first used in England in 1893. TF is an aerobic treatment system utilizing microorganisms attached on to a medium to remove organic matter from wastewater that passes over, around, through, or by the media. This type of system is typical of a number of technologies, such as rotating biological contactors and packed bed reactors. These systems are known as attached growth processes, in contrast to systems where microorganisms are sustained in a liquid, and are thus known as suspended growth processes (Solomon et al., 1998).

2.5.1. Process Description

A TF consists of a permeable media made of a bed of rock, slag, or more recently, plastic over which wastewater is distributed and trickles through (Solomon et al., 1998). The schematic representation of trickling filters is shown in Figure 2.2.

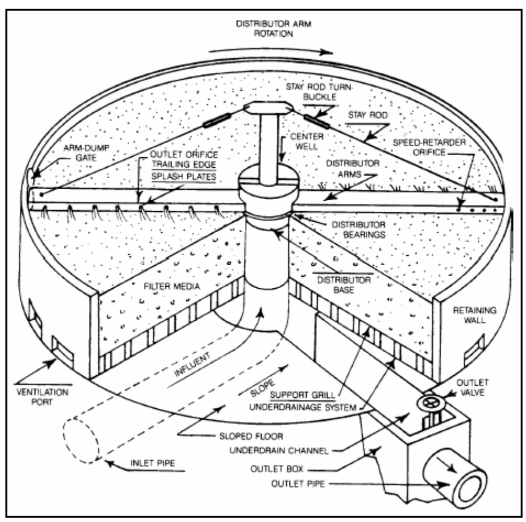


Figure 2.2. Schematic Representation of a Trickling Filter (WEF, 1996.)

The top surface of the media bed is exposed to sunlight, is in an aerobic state, contains microorganisms that are in a rapid growth phase, and is typically covered with algae. The lower portion of the bed is in an anaerobic state and contains microorganisms that are in a state of starvation (i.e., microorganism death exceeds the rate of reproduction). The biofilm covering the filter medium is aerobic to a depth of only 0.1 to 0.2 millimeters; the microbial film beneath the surface biofilm is anaerobic (Viessman and Hammer, 1993). As wastewater flows over the microbial film, organic matter is metabolized and absorbed by a population of microorganisms (aerobic, anaerobic, and facultative bacteria; fungi; algae; and protozoa) attached to the media as a biological film or slime layer (approximately 0.1 to 0.2 mm thick). This film is formed, as the wastewater flows over the media, from microorganisms already in the liquid that gradually attach themselves to the rock, slag, or plastic surface. Organic material is degraded by the aerobic microorganisms in the outer part of the slime layer (Solomon et al., 1998). Continuous air flow is necessary throughout the media bed to prevent complete anaerobic conditions

As the layer thickens (with microbial growth), oxygen cannot penetrate to the media face, and anaerobic organisms develop. As the biological film continues to grow, the microorganisms next to the surface lose their ability to cling to the media, and a portion of the slime layer falls off the filter. This is known as sloughing and is the main source of solids picked up by the underdrain system (Solomon et al., 1998).

Components of a trickling filter include a rotary distributor, underdrain system, and filter medium. Untreated wastewater enters the filter through a feedpipe and flows out onto the filter media via distributor nozzles, which are located throughout the distributor. The distributor spreads the wastewater at a uniform hydraulic load per unit area on the surface of the bed. The underdrain system, typically consisting of vitrified clay blocks, collects the filtrate as well as solids and also serves as a source of air for the microorganisms on the filter. The treated wastewater and solids are piped to a settling tank where the solids are separated. Usually part of the liquid from the settling chamber is recirculated to the TF to dilute the incoming wastewater, keep the filter moist, or in many cases, for process optimization. It is essential that sufficient air be available for successful operation of the TF. It has been found that natural draft and wind forces are usually sufficient if large enough ventilation ports are provided at the bottom of the filter and the media has enough void area (EPA, 2005)

The ideal medium used in a trickling filter should have the following properties: high specific surface area, high void space, light weight, biological inertness, chemical resistance, mechanical durability and low cost. Trickling filter media include redwood palettes, river rock, slag, ceramic, steel and polypropylene saddles and rings, and plastic cross-flow sheets. In rock filled trickling filters, the size of the rock typically varies from 25 to 100 mm in diameter. The depth of the rock varies with each particular design but usually ranges from 0.9 to 2.5 m and averages 1.8 m. Rock filter beds are usually circular, and the liquid wastewater is distributed over the top of the bed by a rotary distributor. Trickling filters that use plastic media have been built with depths varying from 3.6 to 11 m (Tchobanoglous and Burton, 1991).

2.5.2. Application and Performance

Like activated sludge, the primary objective of trickling filter is the removal of soluble organic matter through its conversion to microbial cells. Trickling filters are widely used for the treatment of municipal and industrial wastewaters.

The trickling filter medium has been crushed rock or stone; however, this type of media occupies most of the volume in a filter bed, reducing the void spaces for air passage and limiting surface area for biological growth. Many trickling filters now use a chemical-resistant plastic medium because it has a greater surface area and a large percentage of free space. These synthesized media forms offer several advantages over naturally available materials, particularly in terms of surface contact area, void space, packing density, and construction flexibility (Viessman and Hammer, 1993).

A single or two-stage trickling filter can remove N through biological nitrification. The nitrification process uses oxygen and microorganisms to convert ammonia to nitrite nitrogen, which is then converted to nitrate nitrogen by other microorganisms. Nitrate nitrogen is less toxic to fish and can be converted to nitrogen gas, which can be released to the atmosphere through denitrification, a separate anaerobic process following nitrification. Note that trickling filters are not capable of denitrifying.

A single-stage trickling filter removes BOD in the upper portion of the unit while nitrification occurs in the lower portion. A two-stage system removes BOD in the first stage while nitrification occurs in the second stage. Trickling filters do not typically remove P, but can be adapted to remove P from the wastewater effluent by chemical precipitation following BOD removal and nitrification (EPA, 2005).

It is critical to have a properly designed trickling filter system for efficient operation. An improperly designed system can significantly impact treatment performance and effluent quality. Media configuration, bed depth, hydraulic loading, and residence time all need to be carefully considered when designing a trickling filter system (Viessman and Hammer, 1993).

In a study using municipal wastewater, the average BOD removal was greater than 90 percent and TSS removal was greater than 87 percent using a trickling filter. The average effluent BOD concentration was 13 mg/L, while the average effluent TSS concentration was 17 mg/L. In another similar study that included municipal and a dairy waste, BOD and TSS concentrations never exceeded 100 mg/L in the effluent.

In another study using municipal wastewater and an anaerobic upflow filter prior to a trickling filter, the average effluent BOD and TSS concentrations both ranged from 5 to 10 mg/L, and the total N removal ranged from 80 to 95 percent.

Pathogen reduction in this particular system was expected to be good, due to the up flow filter component (EPA, 2005).

Information on the reduction of pathogens, antibiotics, and metals in trickling filters are not available, but it is expected to be minimal based on engineering judgment.

Finally, since they are simple to operate, trickling filters are used extensively by small communities which cannot afford the highly skilled operators required for activated sludge. Like activated sludge, however, they must be part of a complete treatment system because provision must be made for disposal of the biosolids produced.

2.5.3. Process Microbiology

In trickling filters, removal of organic matter is performed by the microbial film on the surface of the filter media and is made of organic materials (granula). Different species are attached to the media i.e. bacteria, fungi, protozoa, higher organisms, and exopolymeric substances (EPS) from microorganism. Structure of the biofilm varies according to wastewater composition and wastewater loads (Schubert and Günthert, 2001).

Organic material from the liquid is absorbed onto the biological film or slime layer. In the outer parts of the biological slime layer (0.1 to 0.2 mm), organic material is degraded by aerobic microorganisms. As the microorganisms grow, slime thickness increases and the diffused oxygen is consumed before it can penetrate to the full depth of the slime layer (Tchobanoglous and Burton, 1991). Upon microbiological growth biofilms get thicker and partially anaerobic zones develop in time. In these zones, due to lack of substrate and oxygen, parts of the population die and biofilm gets flushed off. This phenomenon is called "sloughing" and is primarily a function of the organic and hydraulic loading on the filter (Steinmann, 1989).

It has been known that the amount of film accumulating in the filter fluctuates seasonally and that the thickness of film increases in winter and decreases in summer. The causes of seasonal fluctuation in the film thickness have been considered as: change in microbial activity with temperature and change in the grazing activity of macro-fauna with temperature (Honda and Matsumoto, 1983). Shephard and Hawkes (1976) reported that in the presence of macro-fauna the film thickness is kept lower by the grazing activity than when controlled microbiologically. Factors other than temperature controlling growth of the film layer are flow rate and organic loading. The flow rate exerts influence on the hydraulic shear around the liquid layer of the film. As a result, film sloughing occurs. The organic loading also influences organic matter removed by the filter and the growth rate of the film. Some part of the organic matter is converted into the film, in turn increasing the film thickness (Honda and Matsumoto, 1983).

The biological community in the filter includes aerobic, anaerobic, and facultative bacteria, fungi, algae, protozoans. Higher animals, such as worms, insect larvae, and snails, are also present. Facultative bacteria are predominating microorganism in the trickling filter. Along with the aerobic and anaerobic bacteria, their role is to decompose the organic material in the wastewater. The higher animals, such as snails, worms, and insects, feed on the biological films in the filter and, as a result, help to keep the bacterial population in a state of high growth or rapid food utilization (Tchobanoglous and Burton, 1991).

Variations in the individual population of biological community occur throughout the filter depth with changes in organic loading, hydraulic loading, influent wastewater composition, pH, temperature, and air availability (Tchobanoglous and Burton, 1991).

In a study aiming to explore the effect of organic strength of wastewater on the population dynamics of TF, Shipin et al. (1999a) investigated a response to wastewater of low nonalgal organic content ($COD_{nonalgal}$ approx. 70 mg/L) but containing a high number of algae (COD_{algal} approx. 370 mg/L). In the case of high organic strength of wastewater ($COD_{nonalgal}$ up to 440 mg/l) but containing no microalgae (data not shown), they found that the high organic concentration in the feed predictably boosted bacterial, fungal and protozoal components. In contrast microalgae and rotifers were eliminated possibly due to competition with bacteria. Biofilm mass decreased 6 times. In the third case, column was fed wastewater containing high organic components of both algal (CODalgal approx. 350 mg/l) and nonalgal ($COD_{nonalgal}$ approx. 220 mg/l) nature i.e. ratio of nonalgal/algal COD was set at 2:3. Over a period of more than 2 months the consortium did not undergo dramatic changes as in case 1 and 2. Although a certain decrease in numbers of all the groups occurred (with a notable exception of microalgae). They found that the deficiency in available organic compounds led to a consistent decrease in bacterial numbers of four orders of magnitude, a second order of magnitude decrease in protozoa and approximately 6 times decrease in the biofilm mass. Their results confirmed field observation that supplementation is a crucial requirement for algae removal. Lack of dissolved organics in the algae rich in the TF inflow rapidly leads to the loss of a healthy biofilm consortium.

High performance of the system relies on the establishment of the biofilm on the filter medium. Shipin et al. (1998) proved that the development of the TF biofilm mass increases over time and is independent of loading rate and the efficiency of algal removal is directly dependent on the mass of biofilm present.

2.5.4. Advantages and Disadvantages

Solomon et al. (1998) listed some advantages and disadvantages of TFs. Some of the advantages are;

- Simple, reliable process that is suitable in areas where large tracts of land are not available for a treatment system
- May qualify for equivalent secondary discharge standards

- Effective in treating high concentrations of organics depending on the type of media used
- Appropriate for small- to medium-sized communities and onsite systems
- High degree of performance reliability
- Ability to handle and recover from shock loads
- Durability of process elements
- Relatively low power requirements
- Level of skill and technical expertise needed to manage and operate the system is moderate
- Cost-effective because it entails lower operating and maintenance costs than other biological processes, including less energy consumption and fewer skilled operators

There are also some disadvantages such as;

- Additional treatment may be needed to meet more stringent discharge standards
- Generates sludge that must be treated and disposed of
- Regular operator attention needed
- Relatively high incidence of clogging
- Relatively low loadings required depending on the media
- Limited flexibility and control in comparison with activated sludge processes
- Potential for vector and odor problems

2.5.5. Operation and Maintenance

TFs are typically preceded by primary clarification for solids separation and are followed by final clarification for collection of microbiological growths that slough from the media bed. They can also be preceded by other treatment units such as septic tanks or anaerobic filters. Trickling filters effectively degrade organic pollutants, but can also be designed to remove N and P from the wastewaters.

Trickling filters are relatively simple to operate, are lower in cost than other biological treatment processes, and typically operate at the temperature of the wastewater as modified by that of the air, generally within the 15-25°C range. A high wastewater temperature increases biological activity, but may result in odor problems. Cold wastewater (e.g., 5-10°C) can significantly reduce the BOD removal efficiency (Viessman and Hammer, 1993).

Although TFs are generally reliable processes, there is still a potential for operational problems. Common operating problems can be caused by increased growth of biofilm, changes in wastewater characteristics, improper design, or equipment failures. Solomon et al. (1998) have listed some common problems followed by possible causes and corrective actions:

1. Disagreeable Odors from Filter:

- Excessive organic load causing anaerobic decomposition in filter may cause disagreeable odors and reducing loading; increasing BOD removal in primary settling tanks; enhancing aerobic conditions in treatment units by adding chemical oxidants, preaerating, recycling plant effluent, or increasing air to aerated grit chambers; scrubbing off gases and using plastic media instead of rock overcome this problem.

- Inadequate ventilation is another cause of disagreeable odors and increasing hydraulic loading to wash out excess biological growth; removing debris from filter effluent channels, underdrains, and the top of filter media; unclogging vent pipes; reducing hydraulic loading if underdrains are flooded; installing fans to induce draft through filter; checking for filter plugging resulting from breakdown of media overcome this problem.

2. Ponding on Filter Media:

Excessive biological growth causes ponding on filter media in order to prevent this problem: reduce organic loading; increase hydraulic loading to increase sloughing; use high-pressure stream of water to flush filter surface; maintain 1 to 2 mg/L residual chlorine on the filter for several hours; flood filter for 24 hours; shut down filter to dry out media; replace media if necessary; remove debris

3. Filter Flies (Psychoda):

- Inadequate moisture on filter media may cause filter flies. Increasing hydraulic loading; unplugging spray orifices or nozzles; using orifice opening at end of rotating distributor arms to spray filter walls; flooding filter for several hours each week during fly season and maintaining 1 to 2 mg/L residual chlorine on the filter for several hours corrects this problem.

- Poor housekeeping may also cause filter flies and to overcome this problem; mow area surrounding filter and remove weeds and shrubs.

2.5.6. The PETRO Process for Removal of Algae

The patented PETRO (Pond Enhanced Treatment and Operation) process was developed in South Africa and features an anaerobic pond (with a fermentation pit) followed by an oxidation pond which is followed by a trickling filter and clarifier to remove algae and reduce BOD and TSS. The basic flow diagram is presented in Figure 2.3. A small proportion of the anaerobic pond effluent along with the oxidation pond effluent is fed to the trickling filter, where the growth of heterotrophic bacteria is able to flocculate and remove the algae. The trickling filter plays a unique role by converting microscopic algae in a biofilm, which can be sloughed off and easily settled without the addition of chemicals. Nearly, 85% nitrogen removal along with a high quality effluent is reported (Arceivela, 1998).

Shipin et al. (1998) studied three full-scale PETRO installations: the municipal sewage works in Kanyamazane (Mpumalanga), Letlhabile (North-West Province) and Elliot (Eastern Cape). An inefficient system of oxidation ponds in Elliot was upgraded according to the PETRO concept in July 1994.

A principal difference between the PETRO and conventional trickling filters is that the former receives an organic load a substantial portion of which is in a form of live algal biomass. The presence of algae in the inflowing wastewater appears to have important consequences for the TF operation (Meiring, 1992). Conventional TFs, among other options including in-line activated sludge reactors, when evaluated, have been found unable to remove algae from well-stabilized oxidation pond water (Shipin et al., 1998; Meiring, 1993; Meiring and Hoffmann, 1994).

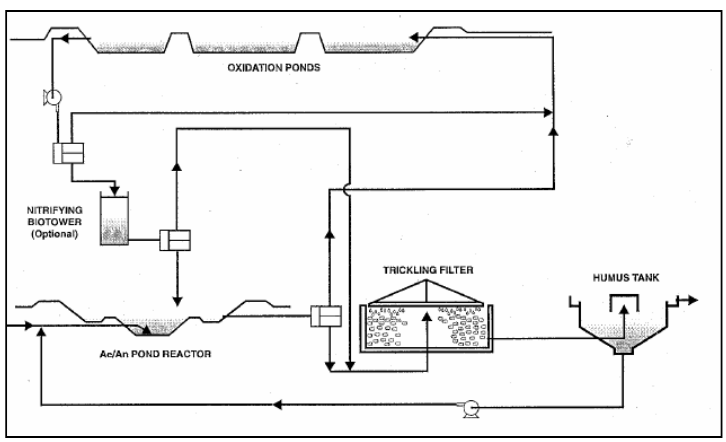


Figure 2.3. Basic flow diagram of PETRO system (TF variant) (Shipin et al., 1998)

The study reported by Shipin et al. (1998) confirmed previous observations concerning the functioning of the PETRO system (Meiring, 1993; Meiring and Oellermann, 1993). The system offers an efficient method of low-tech and low-cost treatment of municipal sewage which has been demonstrated in three full-scale plants for more than a decade (Shipin et al., 1998).

The system incorporates a stage of effective removal of micro-algae from the oxidation pond water. The key element of the algae removal is the PETRO TF. Unlike the conventional TF polishing systems, the algal biomass is now retained in the biofilm. The humus fraction produced has a high settling characteristic. It is recovered in the humus tank and a final effluent of a sparkling clarity is produced. The results obtained suggest a much greater importance of micro-algae in the PETRO TF compared to a conventional TF. Algal biomass appears to contribute to both biofilm production and an organic load reduction in the TF (Shipin et al., 1998).

The operation of the system relies on the establishment of a heterotrophic biofilm on the filter medium affected by supplementation of the TF oxidation pond feed with a nutrient- containing component of the partially treated effluent. The supply of primary pond effluent to the TF inflow is a prerequisite for development of an effective algae-removing biofilm consortium comprising algae, bacteria, fungi, protozoa and metazoa. Protozoa and rotifers grazing on algae in the TF also substantially contribute to the algal removal (Shipin et al., 1998; Shipin et al., 1999a). Results obtained by Shipin et al. (1999a) confirmed field observations that supplementation is a crucial requirement for algae removal.

Shipin et al. (1999b) studied the activated sludge process (ASP) integration into the ponding system in the framework of the PETRO concept (ASP-variant). The role of microalgae in the ASP was elucidated and biological mechanisms of algae removal were suggested. The mechanism of the algae removal in the ASP, the same in principle, appears to differ from that of the TF in several respects. Removal by rotifers and protozoa, though of significance in both cases, plays a greater role in the PETRO TF. Algae removal in the TF is thought to be characterized by both flocculation (due to algal and bacterial EPS production) and degradation (through bacterial activity) of algae. In the ASP this removal is achieved primarily through the stress-induced EPS production (both algal and bacterial) and subsequent flocculation (embedding of the algal biomass in the sludge flocs).

By short-circuiting less than 10 percent of the effluent from a fully loaded primary facultative oxidation pond to the trickling filter, the autotrophic nature or the film in the trickling filter was sufficiently shifted towards a heterotrophic state that had sufficient adsorptive capacity to retain the majority of the algae. It is concluded that the algae, although being absorbed, stay alive on the film and do not contribute significantly to the carbonaceous load on the trickling filter. Furthermore, the algae, although secluded from all sunlight, actually partake in the purification process, producing an effluent which, unlike a normal humus tank effluent, is surprisingly sparkling clear. This significant observation appears to be in line with laboratory findings by others who, when they artificially immobilized certain species of algae and passed water over them, concluded that the algae retained the potential to remove certain compounds from the water. Conglomerates of biologically flocculated dark-green algae are scoured off the biofilm (or sloughed off as part of the film) and, having been photosynthetically inactive for some days, tend not to float, but settle very rapidly (Meiring and Oellermann, 1995).

Trickling filters in general being reliable and simple in operation are classified as “appropriate” technology perfectly suitable to serve developing communities. Nevertheless designers regularly shy away because of their high initial cost. As far as capital cost is concerned the PETRO system has brought about a dramatic change. Affordability has once again become an attractive feature. A substantial reduction of the volumetric requirements of the TF and sludge drying beds, omission of the primary sedimentation tanks and digesters significantly reduces the construction cost. Low power consumption, simplicity of operation, low manpower requirements and minimum mechanical equipment requirements facilitating phase-wise construction, result in a reduced maintenance expenditure. Overall, the system is versatile and site-specific and can be employed in a number of flexible modes (Shipin et al., 1998).

CHAPTER 3

MATERIALS AND METHODS

3.1. Experimental Set-up

Experimental set-up for SFDT (Figure 3.1) which consisted of an algae tank, an influent tank, a pump and a trickling filter column was constructed as shown in Figure 3.2. Algae tank was simulating an oxidation pond effluents and contained only the algae and wastewater was not introduced to this tank. Different volume of algae taken from this tank was added to the influent tank in order to maintain an intended Chl-a concentration. Hence, the influent tank of the trickling filter contained synthetic wastewater having different COD and algae concentrations. Therefore, influent tank represents the mixing point of oxidation pond effluents with the portion of oxidation pond influent on the line of influent of TF. Synthetic wastewater was used to simulate oxidation pond influent which is directed to TF in SFDT in order to maintain a biofilm over the filter media since effective removal of microalgae was affected by the biofilm over the surfaces in the TF (Meiring and Hoffmann, 1994). For this purpose, in a real system, presumably a portion of the influent from the stabilization pond is directly sent to the TF to supply the dissolved organics to maintain a healthy biofilm in the filter; as shown in Figure 3.1.

3.1.1. Trickling Filter

A simple lab-scale trickling filter was constructed from a PVC pipe of 650 mm height and an inner diameter of 100 mm. Filter medium consisted of previously cleaned rocks (12.7-19.1 mm). The rock medium was supported by a perforated plexiglass plate at the bottom which rested on a steel tripod giving extra support. Filter was fed with an influent hose of 4.8 mm inner diameter. The influent was pumped to the top of the filter, and with the aid of a nozzle, sprayed uniformly over the filter. Schematic diagram of the experimental set-up for trickling filter is shown in Figure 3.3.

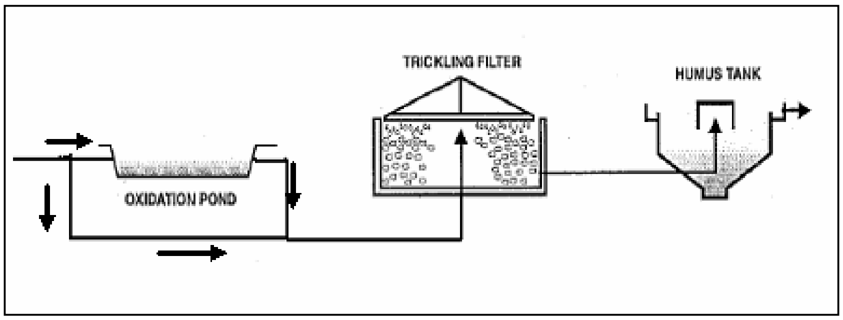


Figure 3.1. Schematic representation of Step Feed Dual Treatment (SFDT)

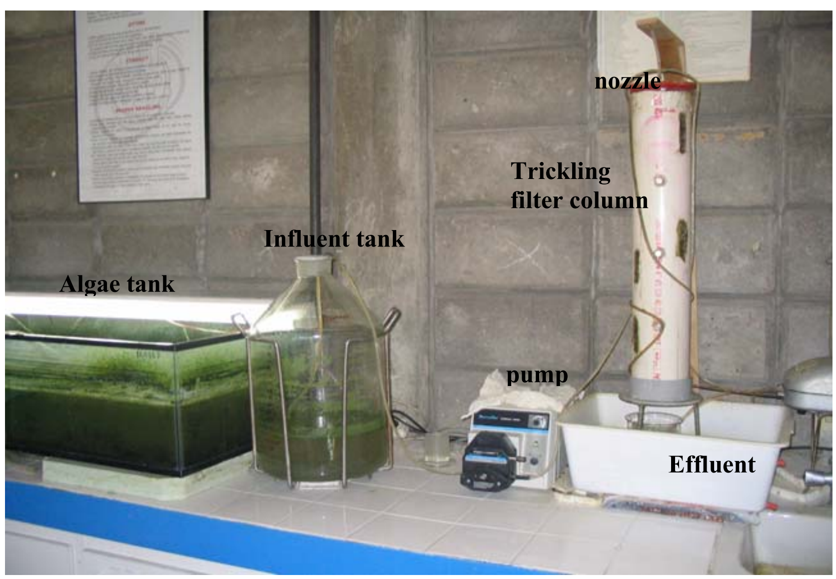


Figure 3.2. Experimental set-up

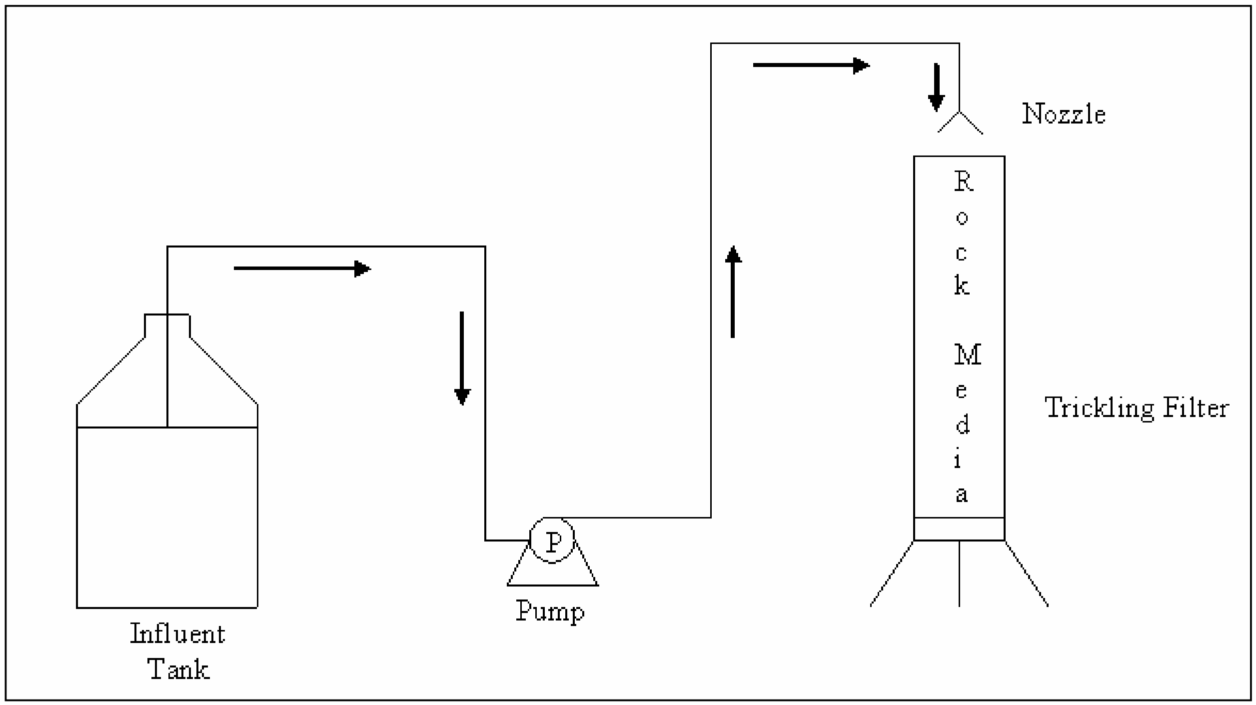


Figure 3.3. Schematic representation of the experimental set-up for trickling filter

3.1.2. Growth Medium

A synthetic wastewater was used throughout the experiments to feed microorganisms attached to the trickling filter medium. The chemical composition of the simulated wastewater is given in Table 3.1. A proteose-peptone was used as source of organic carbon and organic nitrogen and was added to the medium to maintain the intended COD. Phosphate salts were introduced to the medium to provide both buffer action and as phosphorous source for microorganisms.

Table 3.1. Composition of the synthetic wastewater (Dilek et al., 1998)

Constituents	**Concentration (mg/L)**
Proteose-peptone	517* or 141**
NaCl	407.4
Na_2SO_4	44.6
K_2HPO_4	44.6
$MgCI_2.6H_20$	3.7
$FeCI_2.2H_20$	3.7
$CaCI_2.2H_20$	3.7
$MnSO_4$	0.057
H_2MoO_4	0.031
NaOH	0.008
$ZnSO_4$	0.046
$CoSO_4$	0.049
$CuSO_4$	0.076

* For 550 mg/L COD, ** For 150 mg/L COD

3.2. Algal Growth System (Representing the stabilization pond effluents)

Algae were taken from Çubuk 1 dam, located in Ankara-Turkey, and were inoculated into an 80-l algae tank. The Modified Bold's Basal Medium (BBM) was used as inorganic nutrient medium for algal growth. The composition of the inorganic nutrient medium is given in Table 3.2. Furthermore, lighting was provided for 12 h a day with 18 W fluorescent lamps in addition to ambient light to simulate natural field conditions in the laboratory. Algae tank was aerated by an air pump to provide gentle mixing and CO_2, the sole carbon source, as well as O_2 to the algae. The temperature in the tank was maintained at about 25 °C by two heaters. To compensate for evaporation losses, distilled water was added to the growth system as needed. Algal growth system is shown in Figure 3.4. Algal growth was monitored in terms of Chl-a concentration since it is an indicator of viable algae concentration (Swanson and Williamson, 1980).

Table 3.2. Modified Bold's Basal Medium (Stein, 1977)

Stock	**Stock Solution**	**ml/L**
1. KH_2PO_4	8.75 g/500 mL	10 ml
2. $CaCl_2.2H_2O$	1.25 g/500 ml	10 ml
3. $MgSO_4.7H_2O$	3.75 g/500 ml	10 ml
4. $NaNO_3$*	37.5 g/500 ml	10 ml
5. K_2HPO_4	3.75 g/500 ml	10 ml
6. NaCl	1.25 g/500 ml	10 ml
7. Na_2EDTA	10 g/L	1 ml
KOH	6.2 g/L	
8. $FeSO_4.7H_2O$	4.98 g/L	1 ml
H_2SO_4 (conc.)	1 ml/L	
9. Trace Metal Solution	**	1 ml
10. H_3BO_3	5.75 g/500 ml	0.7 ml

****Trace Metal Solution**	
Substance	**g/L**
1. H_3BO_3	2.86 g
2. $MnCl_2.4H_2O$	1.81 g
3. $ZnSO_4.7H_2O$	0.222 g
4. Na $MoO_4.5H_2O$	0.390 g
5. $CuSO_4.5H_2O$	0.079 g
6. $Co(NO_3)_2.6H_2O$	0.0494 g

* Friedman et al., 1977

3.3. Experiments

Firstly, algae containing water taken from the algae tank was transferred to the influent tank which contained synthetic wastewater at the desired COD level (i.e. COD_{inf}). The amount of this water to be added to the influent tank was adjusted to give the intended influent algae level in terms of Chl-a (i.e. Chl-a_{inf}). Variation in Chl-a concentrations are in parallel with the viable algae count (Swanson and Williamson, 1980). Therefore, Chl-a concentrations were measured to represent the variations in algal content.

The influent pump was adjusted to give the desired flow rates to the TF from the influent tank. Wastewater was sprayed onto the top of the filter medium through a nozzle and effluents trickled down to the bottom of the filter. TF of SFDT operated continuously without any interruption for 13 months.

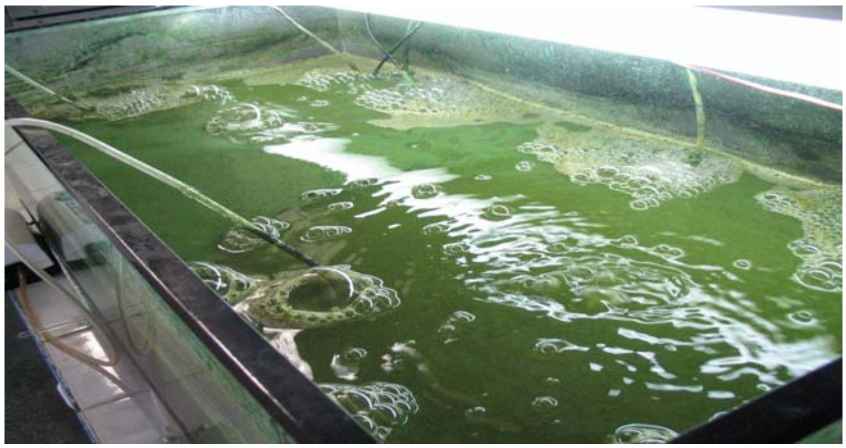


Figure 3.4. The top view of the algal growth system

Several filtration runs were made with different influent characteristics and hydraulic parameters. Table 4.1 presents the combinations applied during these runs. As seen from this table, 10 experimental sets were designed. The variable parameters were HLR (m^3/m^2.day), COD_{inf} (mg/L) and Chl-a_{inf} concentration (μg/L).

An important factor influencing the performance of the trickling filter is the hydraulic application rate per unit of cross-sectional area. In trickling filter design this is called hydraulic loading and is usually expressed as m^3/m^2.day (Schulze, 1960). The flow rate exerts influence on the hydraulic shear of liquid to the film, that is, suspended solids washed out from the filter (Honda and Matsumoto, 1983). As seen from Table 4.1, three different HLR values, namely, 0.5, 2 and 4 m^3/m^2.day were applied to the filter. In fact, at the beginning of the study, former two values determined on the basis of typical HLR values (0.5-3 m^3/m^2.day for low rate and 3-10 m^3/m^2.day for intermediate filters) were planned to study (Nazaroff and Alvarez-Cohen, 2001). However, the results obtained called the necessity for studying higher HLR values (i.e. 4 m^3/m^2.day) so as to determine the system HLR limits. However, within the period specified for this thesis study, it became only possible to run two experimental sets in which only Chl-a values were varied. Effect of COD loading on these additional runs remains to be studied.

Two different influent dissolved COD values were chosen (150 mg/L and 550 mg/L) in order to observe the effect of dissolved organics in the influent on the system performance and also to find the optimum level of organics needed to sustain an effective biofilm over the of the trickling filter medium. Shipin et al. (1998) have proven that the filter performance relies on the establishment of healthy biofilm on the medium. The organic loading influences organic matter removed by the filter and the growth rate of the film, as part of the organic matter is converted into the film and increases the film thickness (Honda and Matsumoto, 1983).

Similarly, Chl-a_{inf} values were varied between 250 and 600 μg/L in the influent, in order to observe the effect of algae concentration on the removal performance by the TF in SFDT.

Effluent quality was monitored at steady state conditions. Filter effluents were analyzed for turbidity and COD. Steady state conditions were usually reached within 2 weeks. Once reached, turbidity, COD, Chl-a and SS measurements were carried out in the settled effluents. All samples were analyzed on the same day. All the analyses conducted were carried out in duplicates. Reported values are the average duplicate measurements and standard deviations of them were also calculated and shown.

All the experiments were conducted at room temperature which varied between 18 and 25^{o}C (except one set in which temperature was about 13^{o}C because of cold weather in winter) depending on seasonal conditions prevailing throughout the experimental period. pH control was not employed during the experiments and pH was between 7 and 8.5 in the effluent and influent. Related records of temperatures and pH are presented in Table A.1 in Appendix A. As shown from table, pH values were not changing considerably with operational parameters. It was observed that there was not a good relationship between pH values and operational parameters.

3.4. Analytical Techniques

The SS and Chl-a measurements were carried out according to the Standard M ethods (APHA, 1996). Temperature, turbidity and pH were measured by thermometer, turbidimeter and pH meter, respectively. Turbidity was read as nephelometric turbidity units (NTU) (APHA, 1996). The COD of the samples were measured according to an EPA approved reactor digestion method (for a COD range of 0-1500 mg/l) using HACH DR2000.

3.5. Statistical Analysis

At the end of the experimental study, in order to examine the relationships between performance parameters such as Chl-a, SS, COD and turbidity (dependent variables) and HLR, COD_{inf} and Chl-a_{inf} concentration (independent variables), factorial design analyses were performed. In doing so, it was intended to determine the variable(s) affecting the removal efficiency significantly, and also interactive effect among the variables, if any, through statistical analysis. STATGRAPHICS Plus 3.1 program was used.

CHAPTER 4

RESULTS AND DISCUSSION

The main goal of this study was to test the ability of trickling filter to scavenge algae from oxidation pond effluents. The trickling filter performance in SFDT was followed in terms of turbidity, Chl-a, SS and COD removals, under various operational conditions.

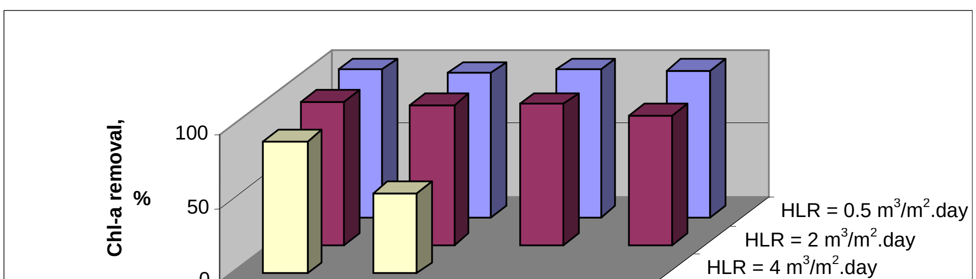
A factorial design summarized in Table 4.1 was set up. The results obtained are presented in Figure 4.1-4.4. Compilation of the related data is given in Table A.1 in Appendix A.

Table 4.1. Experimental sets

Set #	**HLR $m^3/m^2.day$**	**COD_{inf} mg/L**	**$Chl\text{-}a_{inf}$ µg/L**
1	2	550	250
2	2	150	250
3	0.5	150	250
4	0.5	550	250
5	2	550	600
6	2	150	600
7	0.5	150	600
8	0.5	550	600
9	4	150	600
10	4	150	250

When the results obtained are examined (Figure 4.1-4.4), it can be said that, in general, trickling filter performed very well in terms of Chl-a, COD, turbidity and SS removals; all being higher than 85 % for most of the cases with HLR of 0.5 and 2 m^3/m^2.day. Poor performance figures (between 55 and 68 %) were only observed when HLR was increased to its highest value, i.e. 4 m^3/m^2.day.

Highest Chl-a removal was obtained with a value of 99.9 %, when HLR, COD_{inf} and Chl-a_{inf} concentration were 0.5 m^3/m^2.day, 550 mg/L and 250 µg/L respectively (Figure 4.1 - case C). When HLR was increased from 0.5 to 2 m^3/m^2.day, the removal percentages of Chl-a decreased by 2.4-11.6 %, depending on the influent COD and Chl-a values. Most drastic decrease in the Chl-a removal was observed with the increase in HLR to 4 m^3/m^2.day when the COD_{inf} and Chl-a_{inf} concentrations were 150 mg/L and 600 µg/L respectively (Figure 4.1 - case B).



	A	B	C	D
HLR = 4 m^3/m^2.day	89.4	54.8		
HLR = 2 m^3/m^2.day	97.4	95.2	95.9	88
HLR = 0.5 m^3/m^2.day	99.8	97.6	99.9	99.6

A: COD=150 mg/L, Chl-a= 250 µg/L; B: COD=150 mg/L, Chl-a= 600 µg/L; C: COD=550 mg/L, Chl-a= 250 µg/L; D: COD=550 mg/L, Chl-a= 600 µg/L

Figure 4.1. Chl-a removal in TF of SFDT

It is obvious from Figure 4.1 that influent Chl-a concentration inversely affected removal efficiencies of Chl-a. This effect was much more pronounced at the highest HLR value studied. The interactive effects between the operational parameters can also be noticed from Table 4.2 which presents the Chl-a concentrations attained at different operational conditions. Depending on conditions, effluent Chl-a concentration can be as high as 250 µg/L or as low as 0.3 µg/L.

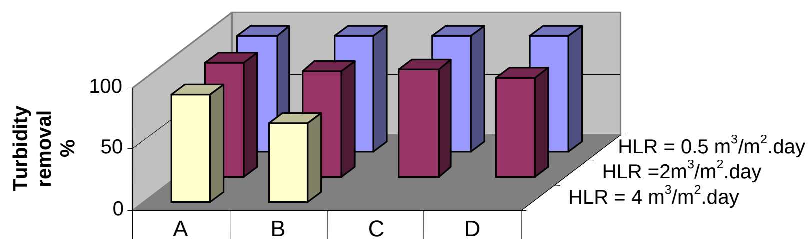
Table 4.2. Average effluent Chl-a concentrations observed

COD_{inf} ($Chla_{inf}$) mg/L (µg Chl-a/L)	**HLR = 0.5 m^3/m^2.day**	**HLR = 2 m^3/m^2.day**	**HLR = 4 m^3/m^2.day**
	Eff µg Chl-a/L	**Eff µg Chl-a/L**	**Eff µg Chl-a/L**
150 (250)	0.5±0.2	7.0±1.4	27.4±1.4
150 (600)	13.1±0.8	26.9±1.7	250.7±0.4
550 (250)	0.3±0.1	11.1±1.5	
550 (600)	2.4±0	68.1±7	

In a thesis study with the rock filter, Oran (1993) also found lower removal efficiency with higher HLR. However, in her study, up to 38 % Chl-a removal was obtained at the HLR of 2 m^3/m^2.day, and the removal did not exceed 64.8 % at the HLR of 0.5 m^3/m^2.day. Whereas, in our study, for all cases with HLR of 0.5 m^3/m^2.day, the removal was greater than 97 %, and for cases with HLR of 2 m^3/m^2.day, the removal was greater than 88 %. Even at HLR of 4 m^3/m^2.day, 89.4% removal became possible when the influent Chl-a was set at 250 µg/L. These findings indicate that TF performance is better than that of rock filter in terms of Chl-a removal. In the rock filter, algae containing water comes to the filter since there is no light algae die off and settle in filter and therefore, it is removed from the water. Whereas, in TF, algae was removed from the water by the aid of biofilm on the TF medium. So, high performance of TF over rock filter results from this.

The results of this study with studies other than Oran's study (1993) are not comparable since operational conditions are different.

Regarding the turbidity, removal efficiencies attained are illustrated in Figure 4.2. The effect of HLR on the removal efficiency of turbidity can be clearly seen from this figure. The highest removal, 95.7 %, was obtained when HLR was 0.5 m^3/m^2.day (COD_{inf} = 550 mg/L; Chl-a_{inf} = 600 µg/L). It can be said that when HLR was lower, effluent turbidity values were also lower (Table 4.3). The system produced clear effluent with turbidity values lower than 2 NTU for all cases of lowest HLR studied. WHO agricultural reuse standard calls for less than (or equal) 2 NTU in the effluents which are used for unrestricted irrigation, so, reuse can be applied (Angelakis et al., 1999).



	A	B	C	D
HLR = 4 m^3/m^2.day	88.6	64.5		
HLR =2m^3/m^2.day	93.9	87.3	88.3	81.7
HLR = 0.5 m^3/m^2.day	95.2	95.1	95.4	95.7

A: COD=150 mg/L, Chl-a= 250 µg/L; B: COD=150 mg/L, Chl-a= 600 µg/L; C: COD=550 mg/L, Chl-a= 250 µg/L; D: COD=550 mg/L, Chl-a= 600 µg/L

Figure 4.2. Turbidity removal in TF of SFDT

Relationship between influent COD concentration and average effluent turbidity values is depicted is Table 4.3. No relation was observed when HLR was 0.5 m^3/m^2.day whereas there was a decrease, though not so much, in the removal of turbidity with the increase in COD_{inf} when HLR was 2 m^3/m^2.day. This can also be seen in Figure 4.2. Turbidity removal efficiencies were about 95 % at HLR of 0.5 m^3/m^2.day, regardless of COD_{inf} and Chl-a_{inf} values. When HLR was 4 m^3/m^2.day, considerable decline in NTU removal was noticeable when influent Chl-a was increased to 600 µg/L.

Obtaining higher removal efficiencies at lower influent COD concentrations is considered as important observation as it would mean a short-circuiting of a lower fraction of stabilization pond influent to the trickling filter.

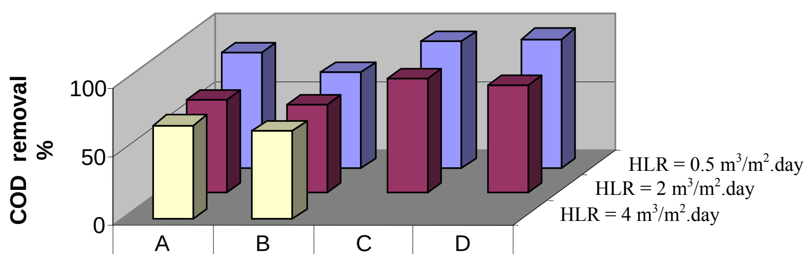
Table 4.3. Average effluent turbidity values observed

COD_{inf} ($Chla_{inf}$) mg/L (µg Chl-a/L)	**HLR = 0.5 m^3/m^2.day**	**HLR = 2 m^3/m^2.day**	**HLR = 4 m^3/m^2.day**
	Eff. NTU	**Eff. NTU**	**Eff. NTU**
150 (250)	1.0	1.3	2.5
150 (600)	1.4	4.4	11.6
550 (250)	1.0	2.4	
550 (600)	1.2	6.7	

When the effect of influent Chl-a concentration on the removal of turbidity is examined it was seen that Chl-a_{inf} concentration inversely affected removal efficiencies of turbidity at higher HLR values. It is very clear that the removal of turbidity decreased with increasing influent Chl-a concentration, being much more remarkable at HLR of 4 m^3/m^2.day with the decrease from about 88 % to 64 % (Figure 4.2).

To compare these findings with Oran's (1993) study; in Oran's study, for example, at HLR of 0.5 m^3/m^2.day, average effluent turbidity value was 6.2 NTU and the removal was about 73 % whereas at HLR of 2 m^3/m^2.day, average effluent turbidity value was 9.59 NTU and removal was 59 %. However, in this study (trickling filter), when HLR was 0.5 m^3/m^2.day, for all cases, average effluent turbidity values were below 1.5 NTU with removal efficiencies greater than 95 % while for HLR of 2 m^3/m^2.day, average effluent turbidity values were lower than 6.7 NTU with removal efficiencies greater than 81 % for all cases.

Within the studied range, when the COD removal efficiency is concerned, it was seen that lower HLR values caused higher COD removals. As can be seen from Figure 4.3, there was a subsequent decrease in the COD removal especially as flow was increased from 0.5 to 2 m^3/m^2.day. A deterioration in the COD removal efficiency was slight when HLR was increased from 2 to 4 m^3/m^2.day. Highest removal percentage was obtained as 93.8 % at the lowest HLR whereas lowest removal was obtained as 63.9 % at the highest HLR studied.



	A	B	C	D
HLR = 4 m^3/m^2.day	67.8	63.9		
HLR = 2 m^3/m^2.day	68.1	64.2	83	78.1
HLR = 0.5 m^3/m^2.day	84	69.5	91.9	93.8

A: COD=150 mg/L, Chl-a= 250 µg/L; B: COD=150 mg/L, Chl-a= 600 µg/L; C: COD=550 mg/L, Chl-a= 250 µg/L; D: COD=550 mg/L, Chl-a= 600 µg/L

Figure 4.3. COD removal in TF of SFDT

Regarding the effect of COD_{inf} on the COD removal performance of the system, higher removal efficiencies were observed when influent COD concentration was higher (i.e. cases C and D), as seen in Fig. 4.3. Corresponding effluent COD values can be seen from Table 4.4. Also, organic loadings applied to the system are tabulated in Table 4.5. Examining Figure 4.3 together with this table revealed that higher removal efficiencies were obtained with higher organic loadings.

For the sake of comparison with the study by Oran (1993), where average effluent COD concentrations were about 92 mg/L and 67 mg/L for HLR of 2 and 0.5 m^3/m^2.day, respectively, the COD removals were 39 % and 60 %, respectively. Whereas in this study, COD removals were about 83 % and 94 % for HLR of 2 and 0.5 m^3/m^2.day, respectively.

Table 4.4. Average effluent COD values observed

COD_{inf} ($Chla_{inf}$) mg/L (µg Chl-a/L)	HLR = 0.5 m^3/m^2.day	HLR = 2 m^3/m^2.day	HLR = 4 m^3/m^2.day
	Eff. mg COD/L	Eff. mg COD/L	Eff. mg COD/L
150 (250)	25.2±1.4	48.8±0.3	49.8±2.3
150 (600)	47.0±3.4	56.6±4	56.0±2.8
550 (250)	44.4±2.3	92.1±3.5	
550 (600)	33.8±2.5	121.0±0.3	

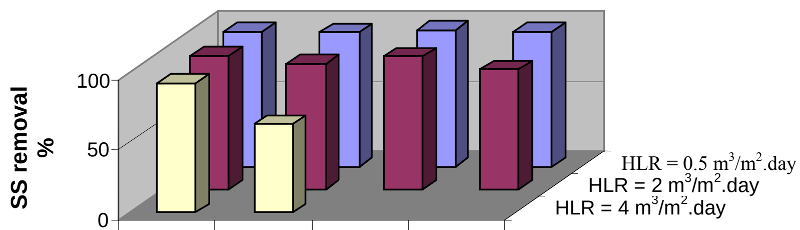
Table 4.5. Organic loading to the system

HLR m^3/m^2.day	COD_{inf} mg/L	Organic Load kg COD/m^3.day
0.5	150	0.12
	550	0.43
2	150	0.47
	550	1.72
4	150	0.94

From these results it is clear that SFDT process provides far better effluent treatment than the rock filter process.

Another quality parameter is suspended solids. Effluent SS concentrations observed in this study are given in Table 4.6. As it can be seen from this table and also from Figure 4.4, HLR had an important effect on SS removal efficiency especially when Chl-a_{inf} values were higher (i.e. cases B and D). HLR was found inversely related with SS removal.

The effect of COD_{inf} on the SS removal efficiency seems to be rather low, almost not detectable as seen in Fig. 4.4. It can be stated that the SS removal was affected primarily by HLR, rather than COD_{inf}. as can also be seen from Table 4.6.



	A	B	C	D
HLR = 4 m^3/m^2.day	92.9	63.8		
HLR = 2 m^3/m^2.day	96.5	90.9	95.9	86.5
HLR = 0.5 m^3/m^2.day	97.1	96.5	98.1	97.4

A: COD=150 mg/L, Chl-a= 250 µg/L; B: COD=150 mg/L, Chl-a= 600 µg/L; C: COD=550 mg/L, Chl-a= 250 µg/L; D: COD=550 mg/L, Chl-a= 600 µg/L

Figure 4.4. SS removal in TF of SFDT

When effect of Chl-a_{inf} on SS removal is examined it can be stated that higher Chl-a_{inf} resulted in higher effluent SS concentration (Table 4.6) and lower removal efficiency (Figure 4.4). One thing that should be mentioned here is that suspended solids mainly resulted from algal mass within the system. Hence, one should expect similar observations with the Chl-a removals. As a matter of fact, our findings for the removal of Chl-a and SS were very similar.

Table 4.6. Average effluent SS values observed

COD_{inf} ($Chla_{inf}$) mg/L (µg Chl-a/L)	**HLR = 0.5 m^3/m^2.day**	**HLR = 2 m^3/m^2.day**	**HLR = 4 m^3/m^2.day**
	SS Eff. mg/L	**SS Eff. mg/L**	**SS Eff. mg/L**
150 (250)	1.8±0.6	2.5±0.8	3.6±0.7
150 (600)	3.4±0.1	11.1±0.6	26.6±2.4
550 (250)	1.5±0.1	2.8±0.6	
550 (600)	2.5±1	15.2±0.7	

The system produced a clear effluent for most of the cases. Pinpoint flocs were usually present in the humus tank, the vessel in which filter effluents were retained for 2 hours, before discharge. The sludge produced was very low in amount, and was stable and settling rapidly as shown in Fig. 4.5.

A B

Figure 4.5. A: Settled effluent sample and B: Influent sample

Maximum SS removal efficiency obtained in Oran's study (1993) was 48.8 % at HLR of 0.5 m^3/m^2.day and 31.5 % at HLR of 2 m^3/m^2.day. In this study, however, for almost all the sets corresponding these two HLRs, the removal efficiencies were consistently higher than 90 %, with and average effluent SS concentrations below 15 mg/L and were usually running around 1.5-3.5 mg/L.

As a means for analyzing solo and combined effects of the three cardinal parameters effecting treatment performance, namely HLR, COD_{inf} and Chl-a_{inf}, a statistical approach was employed.

4.1. Statistical Analyses

4.1.1. Factorial Design

In order to see the effects of two or more independent variables, it is usually more efficient to manipulate these variables in one experiment than to run a separate experiment for each variable. Moreover, only in experiments with more than one independent variable it is possible to test for interactions among variables. For this purpose a factorial design was constructed in this work for the design of experimentation and analysis of results.

Determining which variables are important is sometimes referred to as screening. For this purpose, and also for establishing optimum conditions, it has been found that the family of designs known as the 2^n factorials is useful, as are some related designs (Oran, 1993).

To determine individual and combined effects of certain variables on the removal efficiency of Chl-a, COD, turbidity and SS, in this work the factorial design method was employed.

Firstly, experiments consistent with two-level factorial design with three independent variables, 2^3, namely HLR, COD_{inf} and Chl-a_{inf} concentration, were performed.

The linear first-order model that can be developed performing 2^3 factorial design is as follows;

$$Y = B_0 + B_1X_1 + B_2X_2 + B_3X_3 + B_{12}X_{12} + B_{13}X_{13} + B_{23}X_{23} + B_{123}X_{123} \qquad (4.1)$$

Where;

Y = Removal efficiency (dependent variable), (%)

B_0 = The percentage removal efficiency that corresponds to average operating conditions,

B_1, B_2 and B_3 = The coefficients for HLR, COD_{inf} and Chl-a_{inf} effects, respectively,

B_{12} = The coefficient for combined effects of HLR and COD_{inf}

B_{13} = The coefficient for combined effects of HLR and Chl-a_{inf},

B_{23} = The coefficient for combined effects of COD_{inf} and Chl-a_{inf},

B_{123} = The coefficient for combined effects of HLR, COD_{inf} and Chl-a_{inf},

X_1 = HLR level (-1 for the lowest level; +1 for the highest level and 0 for the average conditions)

X_2 = COD_{inf} level (-1 for the lowest level; +1 for the highest level and 0 for the average conditions)

X_3 = Chl-a_{inf} level (-1 for the lowest level; +1 for the highest level and 0 for the average conditions)

The terms in X_{12}, X_{13}, X_{23}, X_{123} are the interaction terms of independent variables where subscript numerals indicate interacting parameters.

The variables and the levels used in the factorial design are given in Table 4.7. This way each factor is normalized between +1 and -1 to eliminate the effect of magnitudes associated with each parameter. Factorial design matrix is given in Table 4.8.

Table 4.7. Independent variables and their levels for factorial design

Variables	**High level (+1)**	**Low level (-1)**
HLR (m^3/m^2.day)	2	0.5
COD_{inf} (mg/L)	550	150
Chl-a_{inf} (µg/L)	250	600

Table 4.8. Factorial design matrix

Run order	**Set #**	X_1	X_2	X_3	Y_1	Y_2	Y_3	Y_4
1	8	-1	1	1	93.8	99.6	95.7	97.4
2	1	1	1	-1	83	95.9	88.3	95.9
3	4	-1	1	-1	91.9	99.9	95.4	98.1
4	5	1	1	1	78.1	88	81.7	86.5
5	6	1	-1	1	64.2	95.2	87.3	90.9
6	2	1	-1	-1	68.1	97.4	93.9	96.5
7	7	-1	-1	1	69.5	97.6	95.1	96.5
8	3	-1	-1	-1	84	99.8	95.2	97.1

Y_1, Y_2, Y_3, Y_4 = Removal of COD, Chl-a, Turbidity, SS, respectively, %

4.1.1.1. Chl-a Removal

The factorial design shown in Table 4.8 was used to analyze the Chl-a removal.

Coefficients of the variables were obtained from program and substituted in Equation 4.1 in order to come up with the model equation (Eq.4.2);

$$Y=96.75 - 2.55*X_1 - 0.825*X_2 - 1.575*X_3 - 1.35*X_{12} - 0.95*X_{13} - 0.475*X_{23} - 0.95*X_{123} \quad (4.2)$$

From Eq. 4.2 it is obvious that the effect of X_1, HLR, is greater than the others and the next important effect was influent Chl-a concentration (X_3) with a coefficient value of 1.575. All variables seem to affect the Chl-a removal negatively, within the ranges tested. From the interaction terms it is seen that the negative effects are enhanced by their combinations. The negative effect of influent Chl-a concentration (X_3) is understandable, as lower the input, lower the output would be. The HLR effect is also understandable, as lower HLR would give more contact for removal. However negative effect of $COD_{inf.}$ on removal is difficult to explain since higher COD_{inf} should lead to a thicker biofilm on the media which in turn should yield better removals.

Figure 4.6 is a Pareto Chart which is a frequency histogram where the length of each bar on the chart is proportional to the absolute value of its associated estimated effect or the standardized effect. The bars are displayed in the order of the size of the effects, with the largest effects on top, which lets visually identify the most important effects. As displayed in Figure 4.6 the largest effects was belong to the HLR.

Figure 4.7 is a Normal Probability Plot of effects and displays a plot of the cumulative distribution of the effects. It is important for distinguishing effects from noise, especially when the design is unreplicated. The effects that correspond to noise behave as a random sample from a normal distribution; that is, they fall approximately along a straight line. Effects that correspond to real signals fall further from the line. It is understood from this figure that the effects of X_3, X_{12} and X_1 are real signals, while the effects of the others seem to be noise. The strong effect of X_1 (HLR) is also obviated in this graph.

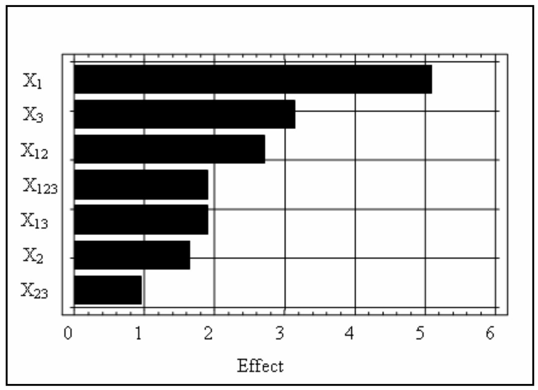


Figure 4.6. Pareto chart for removal of Chl-a

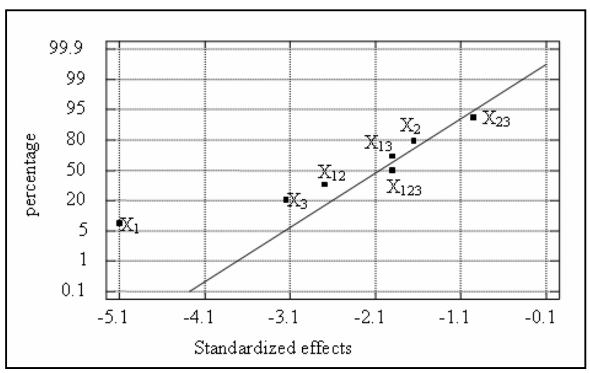


Figure 4.7. Normal Probability Plot for removal of Chl-a

Figure 4.8 displays observed versus predicted plot for the response variable, Chl-a removal. This plot helps to determine whether the model, as estimated, adequately fits the data. It is clear that the model estimated is adequately fits the data.

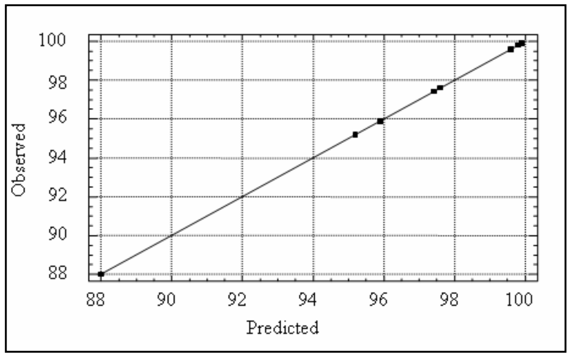


Figure 4.8. Removal of Chl-a values observed versus the predicted

Figures showing the estimated response surfaces for Chl-a removal are shown in Appendix B.

4.1.1.2. COD Removal

As it can be seen from Eq. 4.3, the coefficient of X_2 is 7.625. This value is the maximum among the others. This implies that removal efficiency of COD was largely affected by the influent COD concentration. That is, higher $COD_{inf.}$ concentration provides higher COD removals. This is understandable as higher COD would support a stronger and healthier biofilm on the media, in turn affecting higher COD removal. This is opposite of Chl-$a_{inf.}$ and Chl-a removal in the previous case, suggesting that Chl-a_{inf}, and the COD thereof, do not participate in biofilm growth. The coefficient of HLR is B_1: -5.725 and of Chl-a_{inf} is B_3: -2.675. The minus sign indicates that any decrease in HLR and Chl-a_{inf} results in increasing removal efficiency within the studied range. The coefficient of X_{123} is not small and this shows variables do not act independently.

$$Y=79.075 - 5.725*X_1 + 7.625*X_2 - 2.675*X_3 - 0.425* X_{12} + 0.475* X_{13} + 1.925* X_{23} - 2.175* X_{123} \quad (4.3)$$

Figure 4.9 also shows which variable has the maximum effect on the removal efficiency. From this figure, it is clear that X_{12} has a minimum effect on system performance.

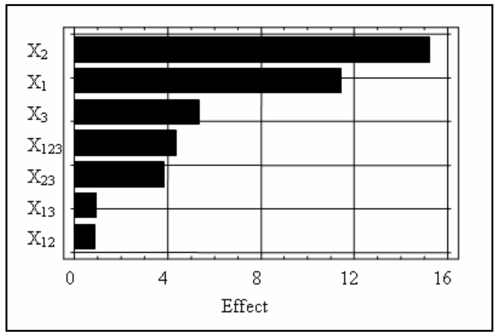


Figure 4.9. Pareto chart for removal of COD

Normal probability plot for removal of COD is given in Fig. 4.10. This figure shows that the effect of COD_{inf} is a real signal among the others. Figure 4.11 indicates that the model is accurate.

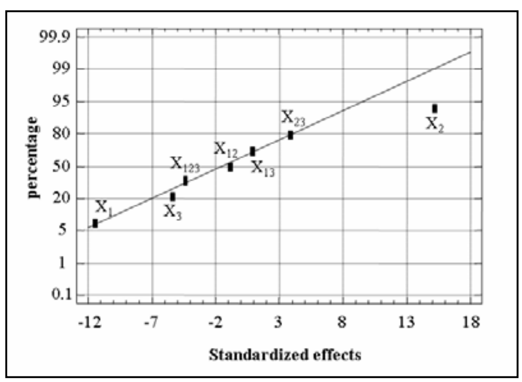


Figure 4.10. Normal Probability Plot for removal of COD

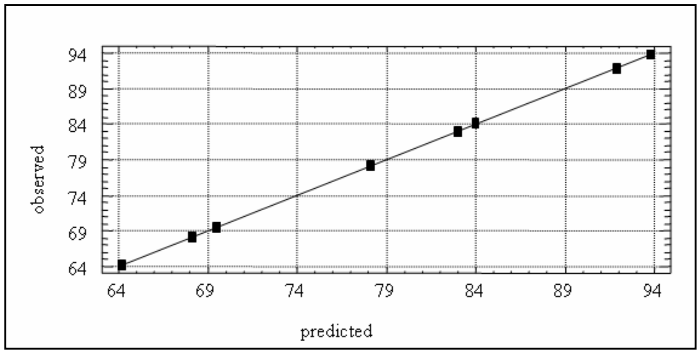


Figure 4.11. Removal of COD values observed versus the predicted

4.1.1.3. Turbidity Removal

From output of factorial design, Eq. 4.4 was obtained for turbidity removal. The coefficient of X_{23}, which is the interaction of X_3 and X_2, and X_{123}, the interaction of all the variables, were nearly zero, indicating these parameters do not influence each other. The coefficient of X_1 is greater than the others. This indicates that the effect of HLR on the removal efficiency of turbidity is higher than the effects of COD_{inf} and Chl-a_{inf}.

$$Y=91.575 - 3.775*X_1 - 1.3*X_2 - 1.625*X_3 - 1.5* X_{12} - 1.675*X_{13} + 0.05* X_{23} - 0.05* X_{123} \quad (4.4)$$

The effects of factors and their interactions are also clear from the pareto chart shown in Fig 4.12. The factor, X_1, with the highest effect was placed at the top and the factors, X_{23} and X_{123}, with the lowest effect at the bottom.

Figure 4.13 helps to distinguish the effects from noise. The effect of X_{12} and X_{23} seem to be noises because of falling approximately along the straight line. Observed versus predicted plot shows the model is reasonable (Figure 4.14).

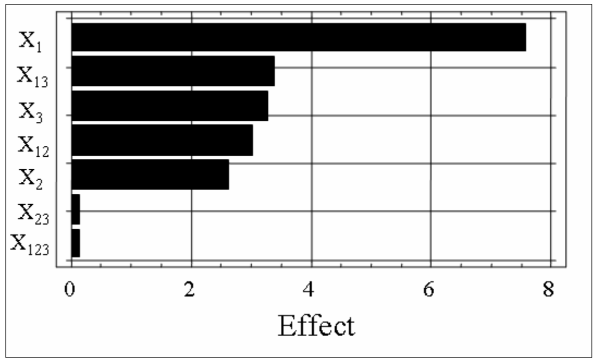


Figure 4.12. Pareto chart for removal of turbidity

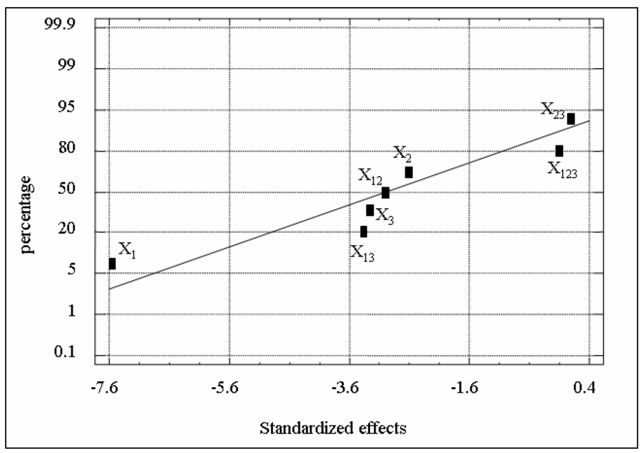


Figure 4.13. Normal Probability Plot for removal of turbidity

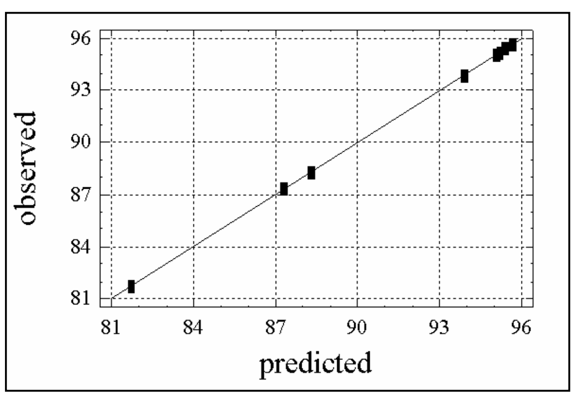


Figure 4.14. Removal of Turbidity values observed versus the predicted

4.1.1.4. SS Removal

As it is shown in Figure 4.15 and as it can be understood from the Eq. 4.5 X_1 has the greatest effect on the removal of SS and all the factors affect the removal negatively. The effect of COD_{inf} is very small like the effects of X_{123} and X_{23}. Any increase in the influent Chl-a concentration results in decreasing removal efficiency within the range studied. This is again understandable since SS and Chl-a parameters are largely correlated.

$$Y=94.8625 - 2.4125*X_1 - 0.3875*X_2 - 2.0375*X_3 - 0.8625*X_{12} - 1.7125*X_{13} - 0.4875*X_{23} - 0.4625*X_{123} \quad (4.5)$$

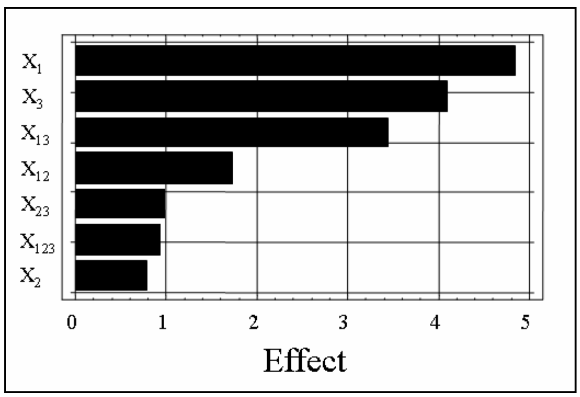


Figure 4.15. Pareto chart for removal of SS

From the plot of normal probability (Figure 4.16), the effects of X_1, X_3 and interaction of them, X_{13} seem to be real signals while the effects of the others seem rather to be noise.

As depicted from the Figure 4.17, the points are distributed uniformly about the diagonal line, indicating that the model, as estimated, fits the data.

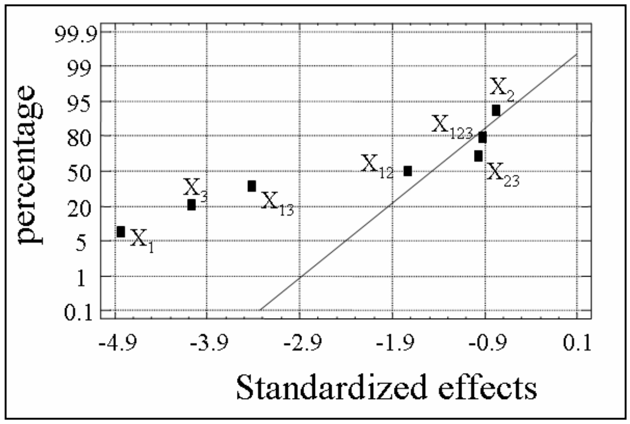


Figure 4.16. Normal Probability Plot for removal of SS

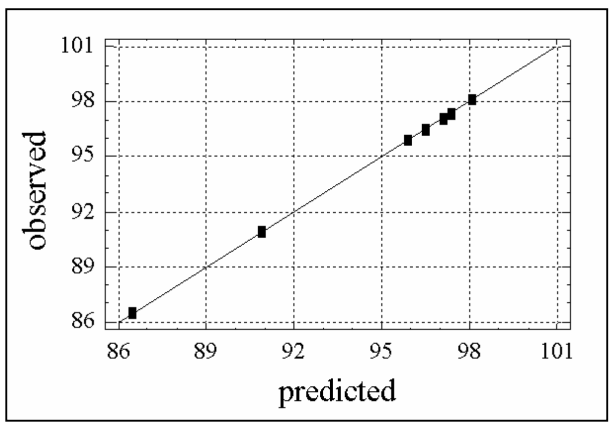


Figure 4.17. Removal of SS values observed versus the predicted

4.1.2. Half-Factorial Design

Full factorial design was performed within somewhat restricted domain and the system was never found close to its limits. In order to test the system at near its limits a half factorial design was employed. Where, HLR and Chl-a, the two prime parameters, were stretched close to their limits. The $COD_{inf.}$ parameter was eliminated from the design because of limited resources and owing to the lower response obtained from this parameter. Another reason for choosing a low constant influent COD, i.e. 150 mg/l, was because in SFDT environment influent is expected to be diluted around 1/4 - 1/5 when entering the TF unit, basing on the removals obtained at 150 mg/l COD_{inf} in the full factorial design. Assuming typical domestic wastewater strength of 500-600 mg/l COD, dilution should provide 100-150 mg/l COD_{inf}.

In this factorial design, influent COD concentration was at constant, at lower value, 150 mg/L. So, it was not chosen as a factor. Experiments consistent with two-level factorial design with two independent variables, 2^2, namely HLR and $Chl\text{-}a_{inf}$ concentration, were performed.
The linear first-order model that can be developed performing 2^2 factorial design experiments is as follows;

$$Y = B_0 + B_1X_1 + B_3X_3 + B_{13}X_{13} \qquad (4.6)$$

The variables and the levels used in the half factorial design were given in Table 4.9. Data obtained from the experiments of $2^{3-1}=2^2$ factorial design and levels of each factor were given in Table 4.10.

Table 4.9. Independent variables and their levels for half-factorial design

Variables	**High level (+1)**	**Low level (-1)**
HLR (m^3/m^2.day)	4	0.5
$Chl\text{-}a_{inf}$ (μg/L)	250	600

4.1.2.1. Chl-a Removal

Factorial design was made according to Table 4.10 in order to see the effects of variables namely HLR and $Chl\text{-}a_{inf}$, on Chl-a removal.

Table 4.10. Half-factorial design matrix

Run order	**Set #**	**X_1**	**X_3**	**Y_1**	**Y_2**	**Y_3**	**Y_4**
1	8	-1	-1	84	99.8	95.2	97.1
2	10	1	-1	67.8	89.4	88.6	92.9
3	9	1	1	63.9	54.8	64.5	63.8
4	7	-1	1	69.5	97.6	95.1	96.5

Y_1, Y_2, Y_3, Y_4 = % Removal of COD, Chl-a, Turbidity, SS, respectively,

Coefficients of the variables were obtained as shown in Eq. 4.7. The equation of the model was obtained as follows;

$$Y = 85.4 - 13.3*X_1 - 9.2*X_3 - 8.1*X_{13} \qquad (4.7)$$

The X_1, X_3 and Y represent HLR (m^3/m^2.day), Chl-a_{inf} (µg/L) and estimated chl-a removal (%), respectively. It is clear that all variables affect Chl-a removal negatively. This means any decrease in their value would result increase in the removal. The coefficient of X_1 is greater than the coefficient of X_3 indicating the effect of former is greater than the effect of latter. Also, the coefficient of interaction of factors is not small, so they do not act independently.

Figure 4.18 is a frequency histogram in which the length of each bar on the chart is proportional to the absolute value of its associated estimated effect or the standardized effect. The bars are displayed in the order of the size of the effects, with the largest effects on top, which is HLR.

From Figure 4.19 it is understood that the effect of HLR was a real signal because of its place far from the line. Also, Figure 4.20 indicates that the model estimated is accurate.

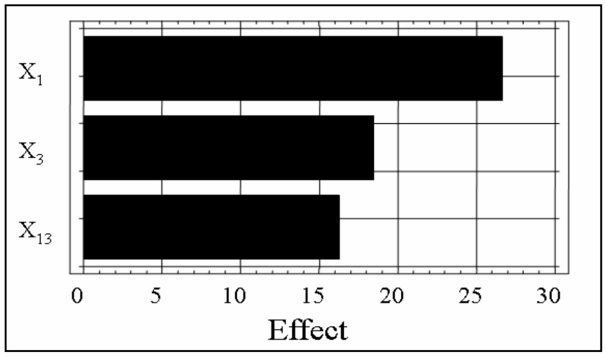


Figure 4.18. Pareto chart for removal of Chl-a (half-factorial)

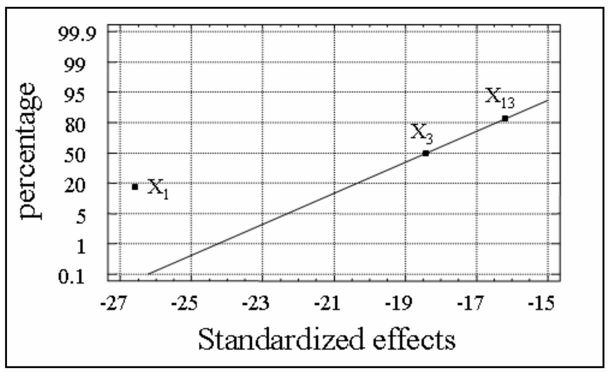


Figure 4.19. Normal Probability Plot for removal of Chl-a (half-factorial)

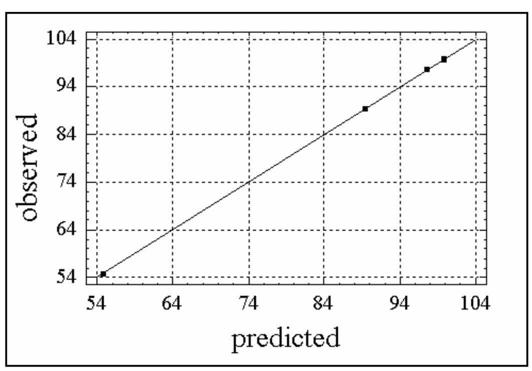


Figure 4.20. Removal of Chl-a values observed versus the predicted (half-factorial)

4.1.2.2. COD Removal

As it can be seen from Eq. 4.8, the coefficient of X_1 is -5.45. This value is the maximum among the others (Figure 4.21). This implies that removal efficiency of COD was largely affected from HLR and its effect is a real signal (Figure 4.22). The coefficient of Chl-a_{inf} is B_3: -4.6. The minus sign indicates that any decrease in HLR and Chl-a_{inf} results in increasing removal efficiency. The coefficient of X_{13} is not small and this shows variables do not act independently. Their combined effect influences COD removal positively. Figure 4.23 is indicative of the accurate model estimation.

$$Y = 71.3 - 5.45*X_1 - 4.6*X_3 + 2.65*X_{13} \tag{4.8}$$

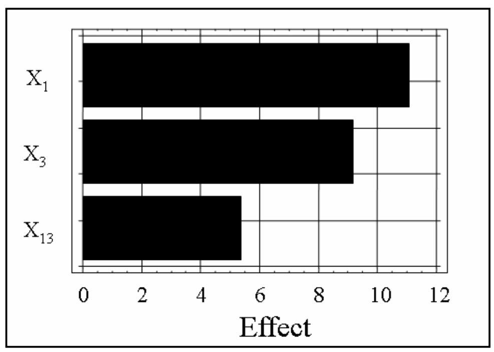


Figure 4.21. Pareto chart for removal of COD (half-factorial)

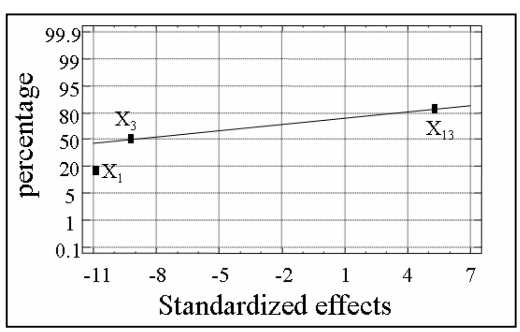


Figure 4.22. Normal Probability Plot for removal of COD (half-factorial)

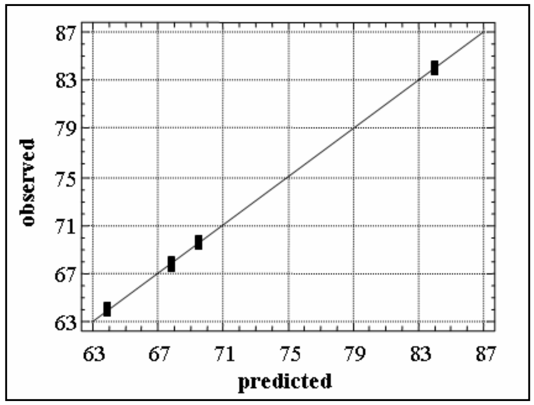


Figure 4.23. Removal of COD values observed versus the predicted (half-factorial)

4.1.2.3. Turbidity Removal

From output of factorial design, Eq. 4.9 was constructed for the removal of turbidity. The coefficient of X_1 is greater than the others. This indicates that the effect of HLR on the removal efficiency of turbidity is higher than the effects of Chl-a_{inf} and their combined effects (Figure 4.24). Also, its effect seems to be a real signal from Figure 4.25. Any increase in the values of factors results a decline in the removal of turbidity.

$$Y = 85.85 - 9.3*X_1 - 6.05*X_3 - 6.0*X_{13} \quad (4.9)$$

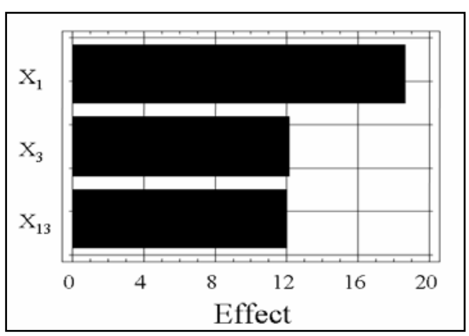


Figure 4.24. Pareto chart for removal of turbidity (half-factorial)

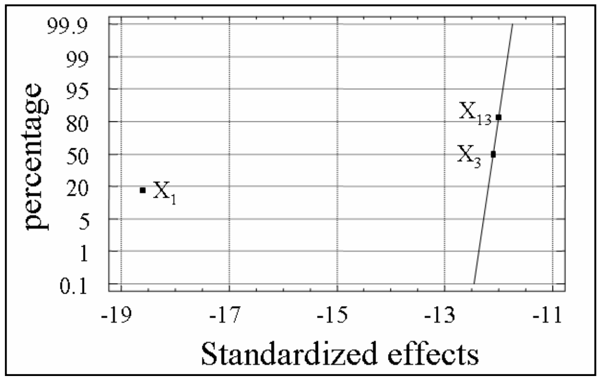


Figure 4.25. Normal Probability Plot for removal of turbidity (half-factorial)

Furthermore, as depicted from Figure 4.26, the points are distributed uniformly about the diagonal line, indicating that the model is reasonably accurate.

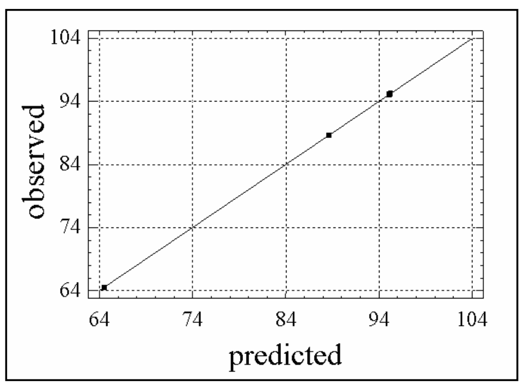


Figure 4.26. Removal of turbidity values observed versus the predicted (half-factorial)

4.1.2.4. SS Removal

As it is shown in Figure 4.27 and as it can be understood from the Eq. 4.10 that X_1 has the greatest effect on the removal of SS and all factors affect the removal negatively. Any increase in HLR and influent Chl-a concentration results in decreasing removal efficiency. Coefficient of combined effect of factors is very large showing that variables do not behave independently.

$$Y= 87.575 - 9.225*X_1 - 7.425*X_3 - 7.125*X_{13} \quad (4.10)$$

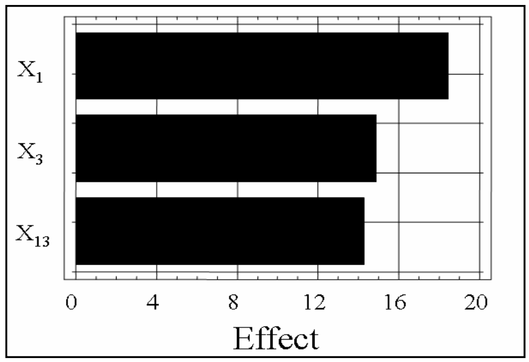


Figure 4.27. Pareto chart for removal of SS (half-factorial)

It is clear from Figure 4.28 that only the effect of HLR corresponds to a real signal while the others seem to be noise and it is understood from the Figure 4.29 that the model (Eq. 4.10) estimated is accurate.

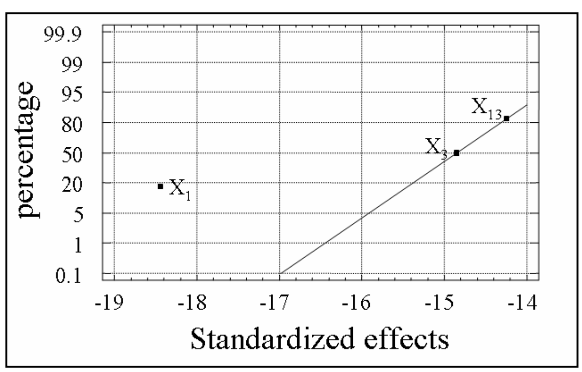


Figure 4.28. Normal Probability Plot for removal of SS (half-factorial)

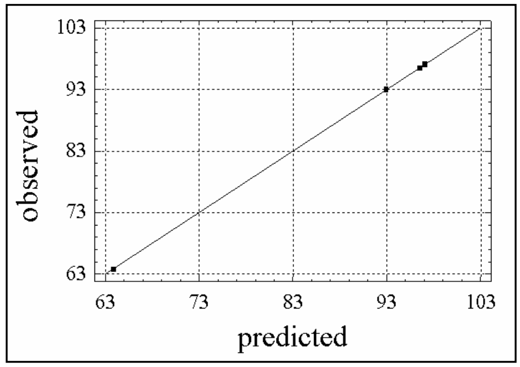


Figure 4.29. Removal of SS values observed versus the predicted (half-factorial)

From half factorial experimental design analysis it can be concluded that the most important parameter determining removals in SFDT process is the HLR. The HLR negatively effects removals in this domain too. Negative effects of Chl-a_{inf} on Chl-a, SS and COD removals are understandable as lower inputs of these parameters should yield lower outputs, and higher removals. Unlike $COD_{inf.,}$ these parameters have already proven ineffective on the biofilm growth hence should not indirectly affect removals.

4.3. Microbiological Observations

Figure 4.30 shows the top view of the model TF during operation. In this figure, yellowish parts indicate biofilm whereas greenish parts refer to the algal mass adsorbed on the biofilm. It can also be seen from Fig. 4.31 that the slime layer on the surface of the filter bed is green. This color is clearly caused by the algae adsorbed on the slime by microbial activity.

As it is obvious from Figure 4.32 that filamentous bacteria were present in the biofilm at the 3rd set in which HLR was 0.5 m^3/m^2.day, COD_{inf}: 150 mg/L and $Chla_{inf}$: 250 μg/L. This was not surprising since low substrate in the effluent favors filamentous bacteria (Tchobanoglous and Burton, 1991).

Higher animals, such as worms, insect larvae, nematodes and filter flies were also present throughout the filter depth through visual observations from the windows on the trickling filter column (Figure 3.2). Figure 4.33 shows a part of biofilm containing nematodes (within red oval) taken from surface of the filter media and the black part (with green oval) shows anaerobic decomposition (at 4th set in which HLR: 0.5 m^3/m^2.day, COD_{inf}: 550 mg/L and Chl-a_{inf}: 250 μg/L). Anaerobic composition at the surface of the filter media was expected since higher COD_{inf} concentration in this set caused rapid development of the thick biofilm. Thicker biofilm prevents oxygen penetration to the full depth of the biofilm and anaerobic zone develops.

Figure 4.34 shows a section of the filter, 35 cm below the top of the filter in which there is an aerobic to anaerobic transition observable (black region indicate anaerobic region) (10th set in which HLR: 4 m^3/m^2.day, COD_{inf}: 150 mg/L, Chl-a_{inf}: 250 μg/L). This was taken as the indicative of the presence of anaerobic and facultative bacteria throughout the filter depth.

The algae found in the influent of TF were mainly consisting of Chlorella species (Figure 4.35). One function of algae in TF may be to enhance dissolved organics conversion into colloids form of extracellular polymeric substances (EPS). Although not indicated by the factor experiments, the EPS imparts viscosity to the biofilm, enhancing immobilization of microbial consortium and preventing its wash-off. Excellent flocculating properties of the algal EPS were demonstrated by Shipin et al., 1999a. These biopolymers aggregate the suspended solids and residual algal biomass and causing sloughing off from the TF rock medium (Shipin et al., 1999a). The solids are removed from the system in the form of readily gravitating flocs.

In order to understand the mechanism of removal of algae, a separate experiment was conducted. In this set HLR, COD_{inf}, and Chl-a_{inf} were 4 m^3/m^2.day, about 550 mg/L and about 600 μg/L, respectively. In this experiment, a simple material balance analysis was followed in terms of Chl-a measurements. To this purpose, effluent sample collected was separated into two portions. First portion was allowed to settle and then Chl-a concentration was measured in the supernatant. Second portion which was containing both effluent and sludge was directly subjected to Chl-a analysis. Results of this analysis are presented in Table 4.11. It was found that about 40 percent of Chl-a was removed from effluent by adsorption on a sloughed biofilm while overall removal from effluent was 70 %. About 30 % of the Chl-a was adsorbed or degraded by the biofilm within the filter. In fact, considering that the filter was being operated for a long time of period (about 13 months) and therefore can be considered as steady in terms of the biofilm as well, one can speculate that this 30 % is standing for degradation rather than adsorption, and the entire adsorbed amount left the system within the sloughed biomass. However, it still remained as speculation as the proof for this could not be provided. During these assessments, findings of Shipin et al (1999b) were taken into consideration such that the removal of algae in TF is characterized by both flocculation (due to algal and bacterial EPS production) and degradation (through bacterial activity) of algae. They reported that conglomerates of biologically flocculated algae were scoured off the film (or as part of the film) and tend to not float, but settle very rapidly. The algae sloughed off the media can be easily thickened and available for ultimate recovery from the water phase without addition of chemicals.

Table 4.11. Removal percentages of Chl-a for removal mechanism

Influent Chl-a, μg/L	**Supernatant Chl-a, μg/L**	**Mixed Effluent Chl-a, μg/L**	**A μg/L**	**B %**	**C %**	**Overall Chl-a removal, %**
547.7	134.3	369.1	234.7	42.7	32.6	75.3

A: Chl-a absorbed on the sludge, μg/L, B: % of Chl-a absorbed on the biofilm sloughed off

C: % of Chl-a degraded by bacterial activity or adsorbed on the biofilm

Figure 4.30. Top view from the model trickling filter (6^{th} set in which HLR: 2 m^3/m^2.day, COD_{inf}: 150 mg/L, Chl-a_{inf}: 600 μg/L)

Figure 4.31. Picture of biofilm from top of the filter (x100) (as greenish parts within red oval shows algae adsorbed on the film and yellowish parts shows biofilm, (at 3^{rd} set in which HLR: 0.5 m^3/m^2.day, COD_{inf}: 150 mg/L, Chl-a_{inf}: 250 μg/L)

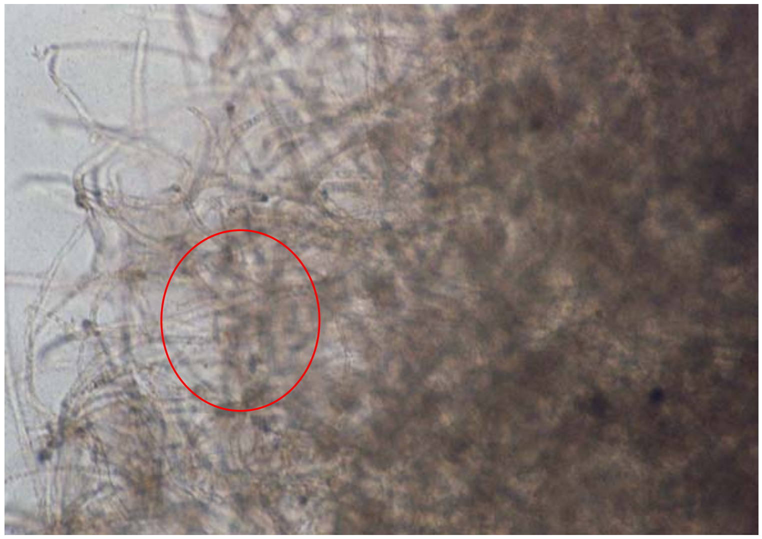


Figure 4.32. Filamentous bacteria in the biofilm (x100)
(at 3rd set in which HLR: 0.5 m^3/m^2.day, COD_{inf}: 150 mg/L, Chl-a_{inf}: 250 μg/L)

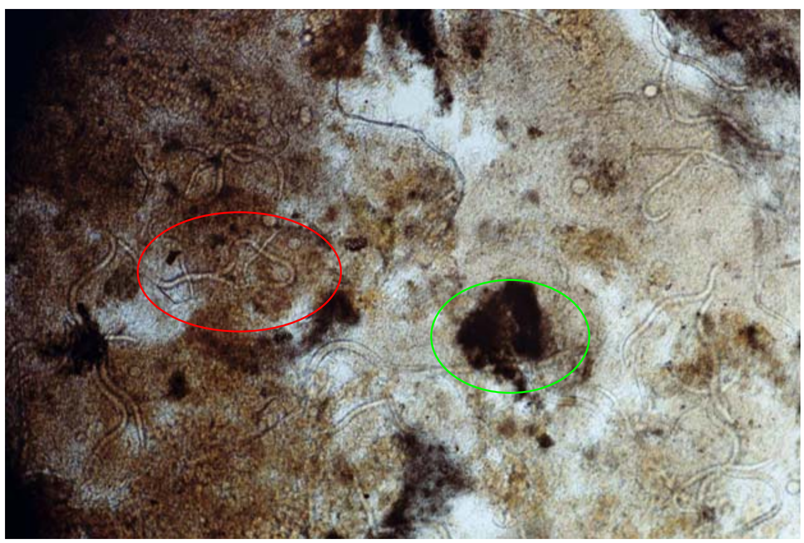


Figure 4.33. A part of biofilm containing nematodes (within red oval) taken from surface of the filter media and black part (with green oval) shows anaerobic decomposition (x100)
(at 4th set in which HLR: 0.5 m^3/m^2.day, COD_{inf}: 550 mg/L, Chl-a_{inf}: 250 μg/L)

Figure 4.34. Section 35 cm from the top of the filter showing aerobic anaerobic transition (black region indicate anaerobic region)
(10th set in which HLR: 4 m^3/m^2.day, COD_{inf}: 150 mg/L, Chl-a_{inf}: 250 µg/L)

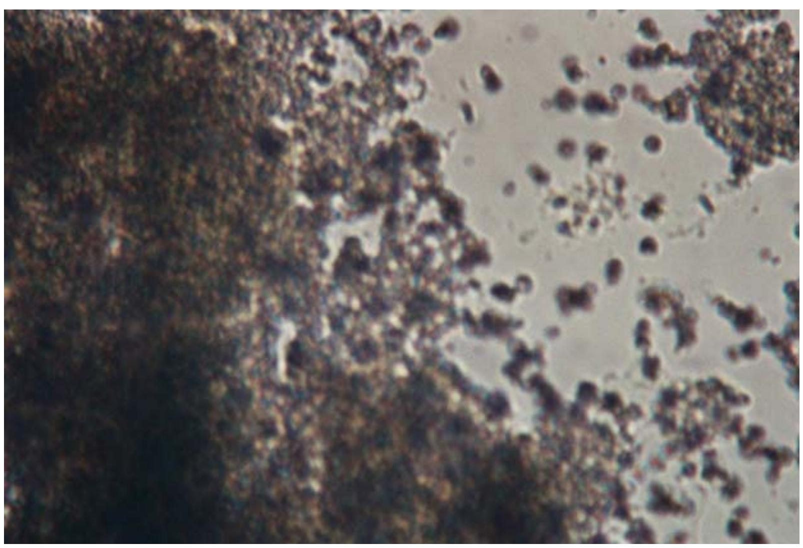


Figure 4.35. Observed algae species (chlorella species) in the influent of the filter (x400)

CHAPTER 5

SUMMARY AND CONCLUSIONS

In this study, the removal of Chl-a, SS and turbidity originating from algae present in oxidation ponds effluents, were investigated using a lab-scale SFDT process. SFDT process developed in this study is the unique, inexpensive and new system to scavenge algae from oxidation pond effluents.

HLR (0.5-2-4 m^3/m^2.day), influent COD (150-550 mg/l) and influent Chl-a concentrations (250-600 μg/l) were selected as operational variables. To determine their magnitude of effects and interactions an experimental design was applied. The following are the conclusions drawn from this study:

SFDT process produced clear effluents, with < 2 NTU, for most of the experimental sets. Since WHO agricultural reuse standard calls for less than (or equal) 2 NTU in the effluents which are used for unrestricted irrigation, treated wastewater can be reused for irrigation purposes.

All the performance parameters studied were affected considerably by hydraulic loading rate variations. Highest removals were obtained at lowest HLR studied.

Data obtained from the experiments showed that when HLR (m^3/m^2.day) was increased from 0.5 to 2, a slight decrease was observed in Chl-a, NTU and COD removals, however, more than 90 % removal was attained in every case, except for COD removals. The lowest removal efficiencies were obtained for all the quality parameters when hydraulic loading was increased to 4 m^3/m^2.day. It was observed that in general removal percentages for turbidity, chl-a, suspended solids and COD increased considerably with the decreasing HLR. Highest removals were obtained at lowest HLR.

The removal of algae in TF was presumably due to both flocculation (due to algal and bacterial EPS production) and degradation (through bacterial activity) of algae.

The relationships between HLR, COD_{inf} and Chl-a_{inf} and removal efficiencies of Chl-a, COD, turbidity and SS were analyzed by the help of a Factorial Design. Table 5.1 gives summary of the relationships for these four quality parameters. Since coefficients of X_1 were highest for all parameters, HLR was the primary operational parameter which affects removal efficiencies of all parameters significantly. Except for removal of COD, the influence of influent Chl-a concentration on the removal efficiencies of all quality parameters was secondary like HLR its effect was also negative. The combined effects were also found to be important.

Table 5.1. Summary of the Model Coefficients

HLR	Parameter	B_0	B_1 for X_1	B_2 for X_2	B_3 for X_3	B_{12} for X_{12}	B_{13} for X_{13}	B_{23} for X_{23}	B_{123} for X_{123}
0.5 – 2 m^3/m^2.day	Chl-a Removal	96.75	**-2.55**	-0.825	**-1.575**	**-1.35**	-0.95	-0.475	-0.95
	COD Removal	79.075	-5.725	**+7.625**	-2.675	-0.425	+0.475	+1.925	-2.175
	Turbidity Removal	91.575	**-3.775**	**-1.3**	**-1.625**	-1.5	**-1.675**	+0.05	**-0.05**
	SS Removal	94.8625	**-2.41**	-0.3875	**-2.0375**	-0.8625	**-1.7125**	-0.4875	-0.4625
0.5 – 4 m^3/m^2.day	Chl-a Removal	85.4	**-13.3**		-9.2		-8.1		
	COD Removal	71.3	**-5.45**		-4.6		+2.65		
	Turbidity Removal	85.85	**-9.3**		-6.05		-6.0		
	SS Removal	87.575	**-9.225**		-7.425		-7.125		

Coefficients with bold font correspond to a real signal.

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APPENDIX A. COMPILATION OF ALL DATA

Table A.1.All Influent and Effluent Data and Removal Efficiencies

		INFFLUENT							EFFLUENT						REMOVAL			
Set #	**HLR $m^3/m^2.d$**	**Day**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**COD %**	**Chl-a %**	**NTU %**	**SS %**
1	2	**1**	8.2	21.2	21.0	69.0 ±4.2	543.0 ±4.2	272.4 ±16.5	8.4	21.9	2.4	2.7 ±1.0	92.0 ±1.4	11.1 ±1.3	83.1	95.9	88.6	96.1
		2	8.3	20.6	25.0	85.6 ±7.9	553.0 ±15.6	249.4 ±10.2	8.6	22.0	2.5	3.3 ±0.4	92.0 ±2.8	12.3 ±1.7	83.4	95.1	90.0	96.2
		3	8.0	22.4	18.0	60.3 ±6.6	530.5 ±23.3	272.3 ±20	8.5	22.1	2.0	1.5 ±0.6	96.0 ±9.9	11.5 ±1.4	81.9	95.8	88.9	97.6
		4	8.2	19.8	20.0	67.5 ±3.5	534.5 ±14.8	289.4 ±19.8	8.3	20.7	2.0	1.5 ±0.6	94.5 ±0.7	10.1 ±1.1	82.3	96.5	90.0	97.9
		5	8.1	21.5	19.0	63.9 ±5.5	556.5 ±19.1	278.5 ±14	8.2	22.6	3.0	5.1 ±0.1	86.0 ±4.2	10.6 ±2.1	84.5	96.2	84.2	92.0
1	**2**	**AVG**	**8.2**	**21.1**	**20.6**	**69.3 ±2.9**	**543.5 ±7.5**	**272.4 ±2.6**	**8.4**	**21.9**	**2.4**	**2.8 ±0.6**	**92.1 ±3.5**	**11.1 ±1.5**	**83.0**	**95.9**	**88.3**	**95.9**
2	2	**1**	8.1	20.7	21.0	70.0 ±7.1	153.0 ±7.1	271.6 ±16.7	8.5	19.7	1.3	2.5 ±0.7	49.0 ±2.8	7.1 ±2.1	68.0	97.4	93.8	96.4
		2	8.4	20.0	19.0	63.9 ±1.6	149.0 ±19.8	272.6 ±14.8	8.8	19.7	1.1	1.0 ±0	39.0 ±0	8.7 ±1.6	73.8	96.8	94.2	98.4
		3	8.0	21.1	22.0	74.8 ±5.3	149.0 ±14.1	280.5 ±15	8.5	19.8	1.2	3.0 ±0	55.0 ±1.4	5.1 ±1.1	63.1	98.2	94.5	96.0
		4	8.1	21.3	22.0	74.8 ±9.6	150.0 ±2.8	265.1 ±11.6	8.6	19.6	1.4	3.0 ±1.4	53.0 ±1.4	6.3 ±1.1	64.7	97.6	93.6	96.0
		5	8.1	20.5	20.2	68.2 ±3.2	165.0 ±12.7	268.7 ±4.3	8.3	19.6	1.4	3.0 ±2.8	48.0 ±4.2	8.0 ±0.8	70.9	97.0	93.1	95.6
2	**2**	**AVG**	**8.1**	**20.7**	**20.8**	**70.3 ±4.7**	**153.2 ±11.3**	**271.7 ±6.6**	**8.5**	**19.7**	**1.3**	**2.5 ±1.0**	**48.8 ±0.3**	**7.0 ±1.4**	**68.1**	**97.4**	**93.9**	**96.5**

Table A.1.All Influent and Effluent Data and Removal Efficiencies (Continued)

		INFFLUENT							EFFLUENT						REMOVAL			
Set #	**HLR $m^3/m^2.d$**	**Day**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**COD %**	**Chl-a %**	**NTU %**	**SS %**
3	0.5	**1**	8.1	22.1	23.0	78.4 ±7.6	167.0 ±8.5	279.5 ±25.6	8.4	20.1	1.0	1.0 ±0	25.0 ±2.8	0.0 ±0	85.0	100.0	95.7	98.7
		2	8.0	21.8	23.0	78.4 ±2.0	159.0 ±12.7	273.2 ±14.6	8.5	19.5	1.1	2.0 ±1.4	29.0 ±1.4	1.6 ±1	81.8	99.4	95.2	97.4
		3	8.0	22.0	22.0	74.8 ±1.7	158.0 ±2.8	281.3 ±4.4	8.5	19.4	0.9	1.0 ±0	25.0 ±1.4	0.0 ±0	84.2	100.0	95.9	98.7
		4	8.0	20.3	20.0	67.5 ±0.7	163.0 ±4.2	265.7 ±11.5	8.5	19.3	1.0	3.0 ±0.7	23.0 ±4.2	1.1 ±0.1	85.9	99.6	95.0	95.6
		5	8.0	21.1	19.0	63.9 ±2.7	151.0 ±1.4	278.1 ±5.4	8.5	19.1	1.0	2.0 ±0.7	24.0 ±0	0.0 ±0	84.1	100.0	94.7	96.9
3	**0.5**	**AVG**	**8.0**	**21.5**	**21.4**	**72.6 ±2.0**	**159.6 ±5.9**	**275.6 ±4.3**	**8.5**	**19.5**	**1.0**	**1.8 ±0.6**	**25.2 ±1.4**	**0.5 ±0.2**	**84.0**	**99.8**	**95.2**	**97.1**
4	0.5	1	7.9	20.5	22.0	74.8 ±5.3	543.0 ±5.7	289.7 ±11.3	8.3	17.3	1.0	1.0 ±0.3	53.0 ±7.1	0.0 ±0	90.2	100.0	95.5	98.7
		2	7.9	20.7	20.0	67.5 ±2.2	551.0 ±5.7	234.9 ±7.9	8.3	17.1	1.0	1.5 ±0.3	43.0 ±2.8	0.0 ±0	92.2	100.0	95.0	97.8
		3	8.0	20.7	25.0	85.6 ±4.7	546.0 ±2.8	256.3 ±8.8	8.0	17.2	1.1	3.0 ±0.4	44.0 ±1.4	0.6 ±0.3	91.9	99.8	95.6	96.5
		4	8.0	20.8	21.0	71.1 ±5.4	548.0 ±1.4	298.5 ±12.9	8.2	18.1	1.0	1.0 ±0.1	40.0 ±1.4	0.0 ±0	92.7	100.0	95.2	98.6
		5	8.0	21.6	23.0	78.4 ±3.7	546.0 ±1.4	281.9 ±7.4	8.4	18.7	1.0	1.0 ±0.3	42.0 ±4.2	1.1 ±0.4	92.3	99.6	95.7	98.7
4	**0.5**	**AVG**	**8.0**	**20.9**	**22.2**	**75.5 ±1.3**	**546.8 ±3.4**	**272.3 ±9.6**	**8.2**	**17.7**	**1.0**	**1.5 ±0.1**	**44.4 ±2.3**	**0.3 ±0.1**	**91.9**	**99.9**	**95.4**	**98.1**

Table A.1.All Influent and Effluent Data and Removal Efficiencies (Continued)

		INFFLUENT							EFFLUENT						REMOVAL			
Set #	**HLR $m^3/m^2.d$**	**Day**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**COD %**	**Chl-a %**	**NTU %**	**SS %**
5	2	1	7.6	25.7	34.0	78.0 ±7.1	538.5 ±0.7	581.2 ±8.2	7.6	24.1	7.0	19.0 ±1.4	111.0 ±4.2	88.4 ±12.9	79.4	84.8	79.4	75.6
		2	7.9	24.9	35.0	122.0 ±2.8	557.0 ±21.2	564.2 ±2.1	7.8	26.5	5.9	11.5 ±2.1	110.0 ±1.4	33.1 ±6.2	80.3	94.1	83.1	90.6
		3	7.9	24.7	37.0	142.0 ±11.3	565.0 ±8.5	572.4 ±25.9	7.8	26.5	6.1	16.0 ±0.7	111.0 ±2.8	57.5 ±6.8	80.4	89.9	83.5	88.7
		4	7.9	24.4	38.0	118.0 ±9.9	546.0 ±5.7	548.4 ±11.9	7.8	27.3	7.0	14.5 ±4.9	136.0 ±0	76.1 ±1	75.1	86.1	81.6	87.7
		5	7.9	24.6	39.0	146.0 ±4.2	551.0 ±2.8	578.9 ±44.8	7.7	27.1	7.5	15.0 ±1.4	137.0 ±7.1	85.4 ±8	75.1	85.2	80.8	89.7
5	**2**	**AVG**	**7.8**	**24.9**	**36.6**	**121.2 ±0.3**	**551.5 ±5.5**	**569.0 ±13.8**	**7.7**	**26.3**	**6.7**	**15.2 ±0.7**	**121.0 ±0.3**	**68.1 ±7**	**78.1**	**88.0**	**81.7**	**86.5**
6	2	1	8.1	19.5	35.0	129.0 ±1.4	152.0 ±2.8	523.7 ±6.2	8.1	11.9	4.5	11.9 ±2.0	51.0 ±4.2	23.4 ±2.8	66.4	95.5	87.1	90.8
		2	8.1	20.7	34.0	118.3 ±8.9	159.0 ±4.2	614.6 ±33	8.1	13.2	4.0	9.5 ±2.2	61.0 ±1.4	31.1 ±2.1	61.6	94.9	88.2	91.9
		3	8.1	17.8	32.0	111.0 ±5.7	161.0 ±5.7	497.7 ±33.4	8.1	12.9	4.0	9.5 ±0.8	57.0 ±4.2	19.7 ±1.3	64.6	96.0	87.5	91.4
		4	8.1	17.4	37.0	129.1 ±2.6	161.0 ±1.4	570.5 ±4	8.1	14.2	5.0	13.2 ±2.6	58.0 ±5.7	30.6 ±3.8	64.0	94.6	86.5	89.8
		5	8.1	19.0	35.0	121.9 ±4.4	158.0 ±0	582.3 ±57.8	8.0	14.0	4.5	11.4 ±0.5	56.0 ±4.2	29.9 ±1.6	64.6	94.9	87.1	90.7
6	**2**	**AVG**	**8.1**	**18.9**	**34.6**	**121.9 ±1.8**	**158.2 ±0.6**	**557.8 ±11**	**8.1**	**13.2**	**4.4**	**11.1 ±0.6**	**56.6 ±4**	**26.9 ±1.7**	**64.2**	**95.2**	**87.3**	**90.9**

Table A.1.All Influent and Effluent Data and Removal Efficiencies (Continued)

		INFFLUENT							EFFLUENT						REMOVAL			
Set #	**HLR $m^3/m^2.d$**	**Day**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**COD %**	**Chl-a %**	**NTU %**	**SS %**
7	0.5	1	7.9	21.5	29.5	101.0 ±2.8	158.0 ±1.4	560.9 ±7.6	7.7	20.7	1.3	4.5 ±0.7	54.0 ±1.4	12.1 ±2.5	65.8	97.8	95.6	95.5
		2	7.6	23.1	27.0	92.9 ±7.2	149.0 ±2.8	525.8 ±11.8	7.5	22.3	1.4	3.5 ±0.7	46.0 ±0	13.3 ±1.7	69.1	97.5	94.8	96.2
		3	7.5	21.9	28.0	96.5 ±0.7	152.0 ±2.8	547.6 ±19.2	7.0	21.5	1.4	2.0 ±0.3	43.0 ±5.7	10.9 ±0.6	71.7	98.0	95.0	97.9
		4	7.7	22.4	31.0	107.0 ±2.8	154.0 ±1.4	529.7 ±5.5	6.8	21.7	1.6	4.0 ±0.8	45.0 ±1.4	12.8 ±2.3	70.8	97.6	94.8	96.3
		5	7.5	23.2	27.0	92.9 ±1.6	158.0 ±0	568.3 ±26.3	7.2	21.9	1.3	3.0 ±0.6	47.0 ±8.5	16.3 ±2.4	70.3	97.1	95.2	96.8
7	**0.5**	**AVG**	**7.6**	**22.4**	**28.5**	**98.1 ±0.5**	**154.2 ±0.6**	**546.5 ±0.5**	**7.3**	**21.6**	**1.4**	**3.4 ±0.1**	**47.0 ±3.4**	**13.1 ±0.8**	**69.5**	**97.6**	**95.1**	**96.5**
8	0.5	1	7.7	24.3	24.0	83.0 ±2.8	567.0 ±7.1	535.4 ±10.9	7.1	22.9	1.2	3.5 ±0.7	33.0 ±2.8	5.6 ±2.1	94.2	99.0	95.0	95.8
		2	7.4	22.4	29.0	100.1 ±5.9	538.0 ±1.4	580.1 ±15.3	7.1	21.8	1.4	4.0 ±1.6	36.0 ±4.2	0.0 ±0	93.3	100.0	95.2	96.0
		3	7.3	20.5	27.0	92.9 ±4.1	561.0 ±1.4	552.0 ±9.1	7.4	21.5	1.0	1.0 ±0.3	35.5 ±0.7	0.9 ±0.3	93.7	99.8	96.3	98.9
		4	7.0	25.2	26.0	89.3 ±6.0	544.0 ±4.2	543.2 ±2.4	7.4	22.0	1.0	1.0 ±0.6	31.5 ±2.1	3.7 ±1.4	94.2	99.3	96.2	98.9
		5	7.4	22.8	33.0	114.6 ±9.4	512.0 ±9.9	587.7 ±17.5	7.1	19.7	1.3	3.0 ±0.8	33.0 ±2.8	1.7 ±0.3	93.6	99.7	96.1	97.4
8	**0.5**	**AVG**	**7.4**	**23.0**	**27.8**	**96.0 ±5.6**	**544.4 ±0.3**	**559.7 ±6.4**	**7.2**	**21.6**	**1.2**	**2.5 ±0.8**	**33.8 ±2.5**	**2.4 ±0**	**93.8**	**99.6**	**95.7**	**97.4**

Table A.1.All Influent and Effluent Data and Removal Efficiencies (Continued)

		INFFLUENT							**EFFLUENT**						**REMOVAL**			
Set #	**HLR $m^3/m^2.d$**	**Day**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**pH**	**T °C**	**NTU**	**SS mg/L**	**COD mg/L**	**Chl-a µg/L**	**COD %**	**Chl-a %**	**NTU %**	**SS %**
9	4	1	8.1	25.1	31.0	74.0 ±2.8	157.0 ±4.2	575.8 ±16.1	7.6	26.6	12.0	32.0 ±1.4	57.0 ±4.2	240.8 ±13	63.7	58.2	61.3	56.8
		2	8.2	24.7	32.0	72.0 ±1.4	153.0 ±4.2	521.2 ±35.1	7.7	26.2	13.0	30.0 ±0.7	55.0 ±8.5	240.0 ±14.3	64.1	54.0	59.4	58.3
		3	8.2	23.1	34.0	74.0 ±7.1	154.0 ±2.8	579.6 ±24.5	7.7	26.7	11.0	25.0 ±2.8	56.0 ±0	273.4 ±6.6	63.6	52.8	67.6	66.2
		4	8.2	25.2	31.0	75.0 ±1.4	155.0 ±1.4	532.1 ±8.2	7.6	25.9	10.0	22.0 ±2.8	55.0 ±0	248.0 ±14.8	64.5	53.4	67.7	70.7
		5	8.0	24.6	36.0	73.0 ±2.8	156.0 ±4.2	566.2 ±31.5	7.6	24.8	12.0	24.0 ±4.2	57.0 ±1.4	251.5 ±18.1	63.5	55.6	66.7	67.1
9	**4**	**AVG**	**8.1**	**24.5**	**32.8**	**73.6 ±2.5**	**155.0 ±2.8**	**555.0 ±16.6**	**7.6**	**26.0**	**11.6**	**26.6 ±2.4**	**56.0 ±2.8**	**250.7 ±0.4**	**63.9**	**54.8**	**64.5**	**63.8**
10	4	1	7.7	22.1	19.0	47.0 ±2.8	155.0 ±1.4	241.4 ±0.1	7.8	26.4	3.1	6.0 ±1.8	50.0 ±4.2	36.8 ±1.6	67.7	84.8	83.7	87.2
		2	7.7	21.6	25.0	58.0 ±1.4	157.0 ±1.4	281.2 ±21.9	7.7	25.1	2.5	4.5 ±0.7	51.0 ±1.4	32.5 ±4.8	67.5	88.4	90.0	92.2
		3	7.7	21.8	22.0	46.0 ±4.2	151.0 ±4.2	269.1 ±28.6	7.8	25.9	2.4	4.0 ±0.6	49.0 ±1.4	39.4 ±2.7	67.5	85.4	89.1	91.3
		4	7.7	20.1	21.0	41.0 ±4.2	154.0 ±12.7	232.8 ±10	7.8	25.3	2.2	1.0 ±0.1	49.0 ±0	13.8 ±1.3	68.2	94.1	89.5	97.6
		5	7.7	19.9	22.0	66.0 ±5.7	156.0 ±7.1	266.2 ±18.5	7.7	24.8	2.1	2.5 ±0.3	50.0 ±7.1	14.5 ±2	67.9	94.6	90.5	96.2
10	**4**	**AVG**	**7.7**	**21.1**	**21.8**	**51.6 ±3.7**	**154.6 ±0.8**	**258.1 ±4.4**	**7.7**	**25.5**	**2.5**	**3.6 ±0.7**	**49.8 ±2.3**	**27.4 ±1.4**	**67.8**	**89.4**	**88.6**	**92.9**

Appendix B

Estimated Response Surfaces

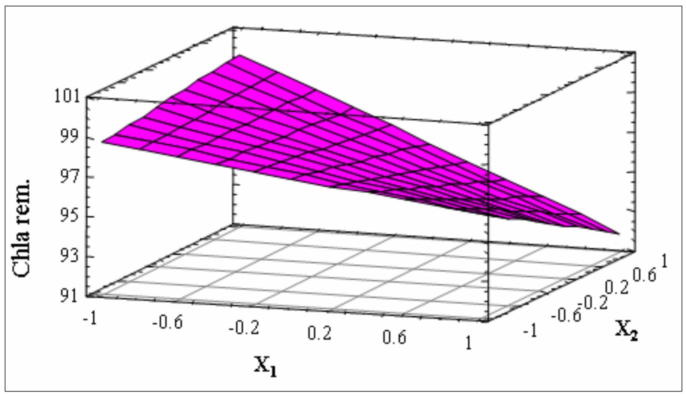

Figure B.1. Estimated response surface for Chl-a removal, X_1 versus X_2

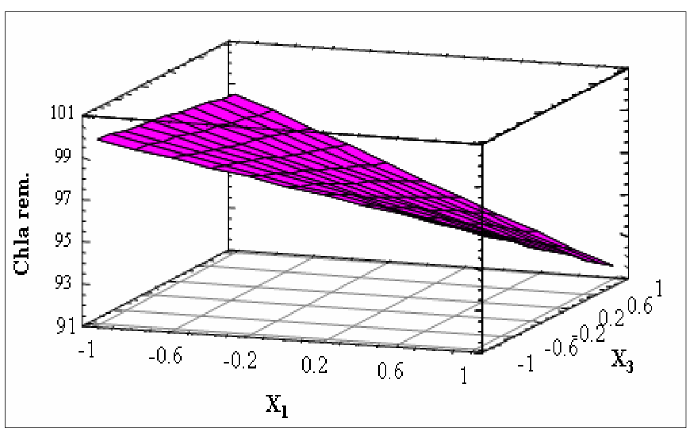

Figure B.2. Estimated response surface for Chl-a removal, X_1 versus X_3

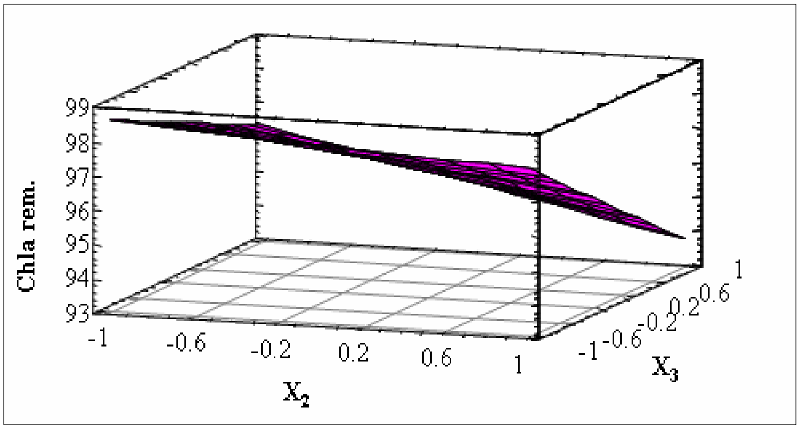

Figure B.3. Estimated response surface for Chl-a removal, X_2 versus X_3

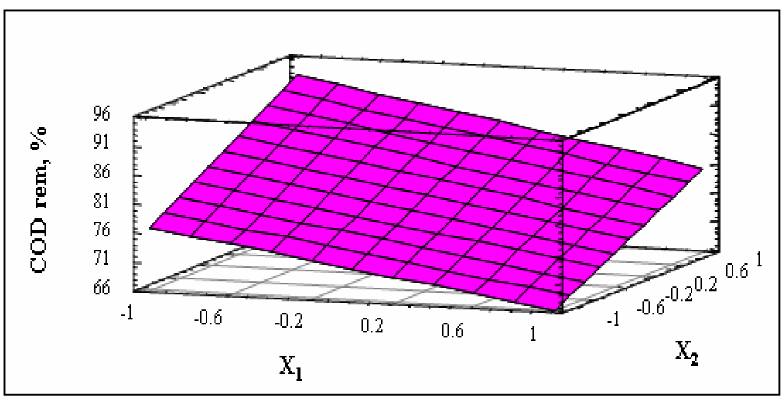

Figure B.4. Estimated response surface for COD removal, X_1 versus X_2

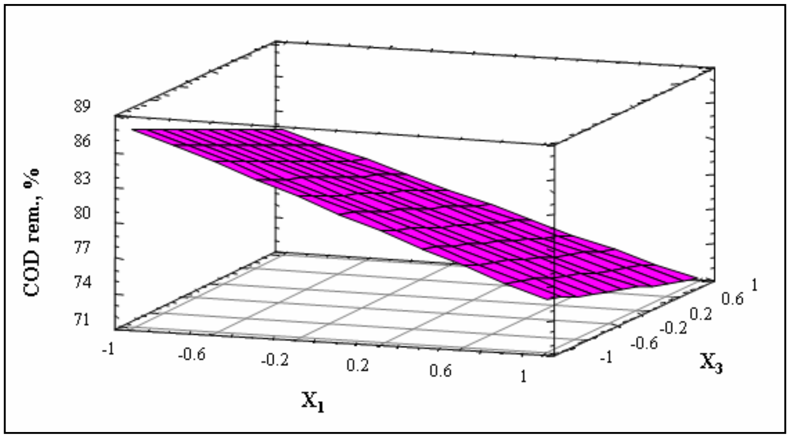

Figure B.5. Estimated response surface for COD removal, X_1 versus X_3

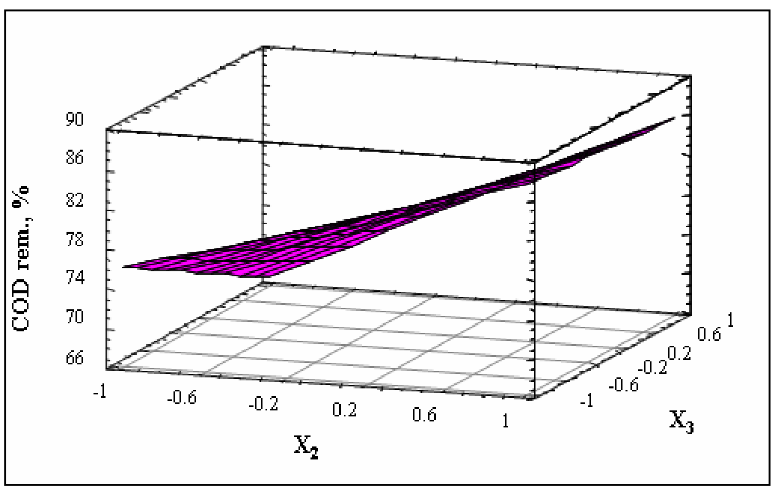

Figure B.6. Estimated response surface for COD removal, X_2 versus X_3

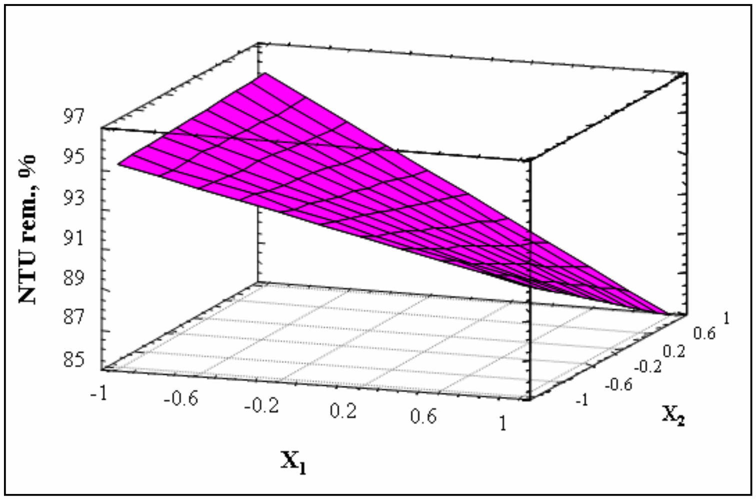

Figure B.7. Estimated response surface for Turbidity removal, X_1 versus X_2

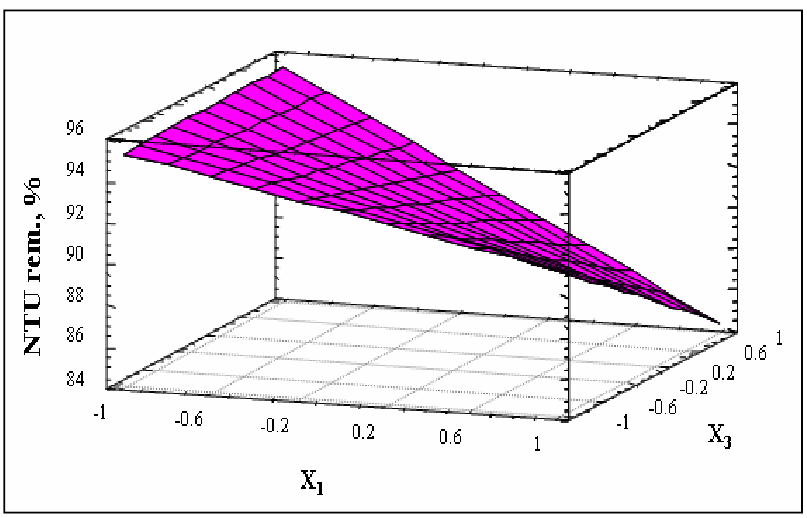

Figure B.8. Estimated response surface for Turbidity removal, X1 versus X3

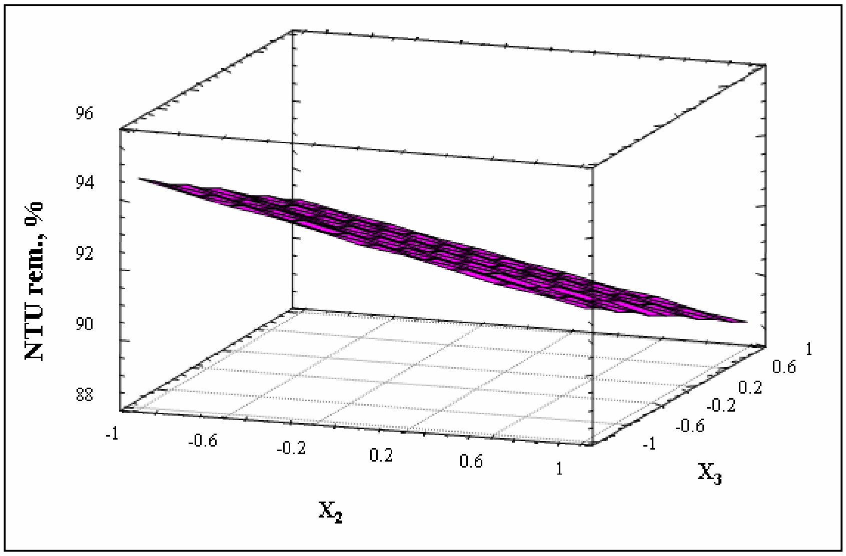

Figure B.9. Estimated response surface for Turbidity removal, X_2 versus X_3

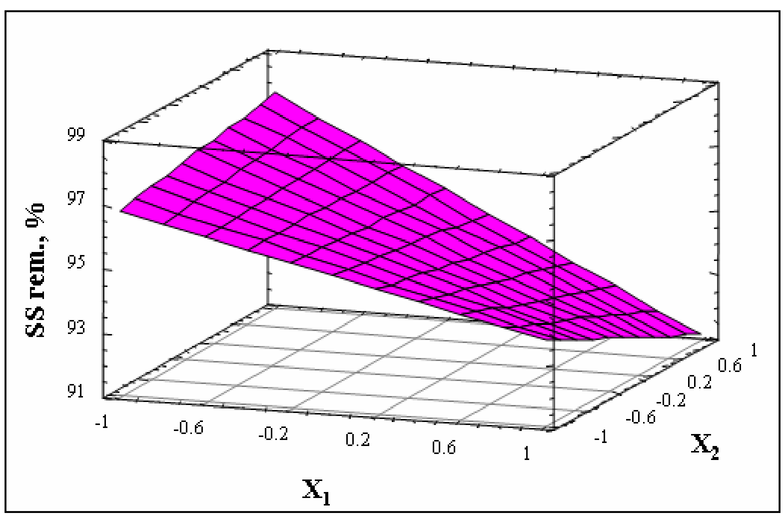

Figure B.10. Estimated response surface for SS removal, X_1 versus X_2

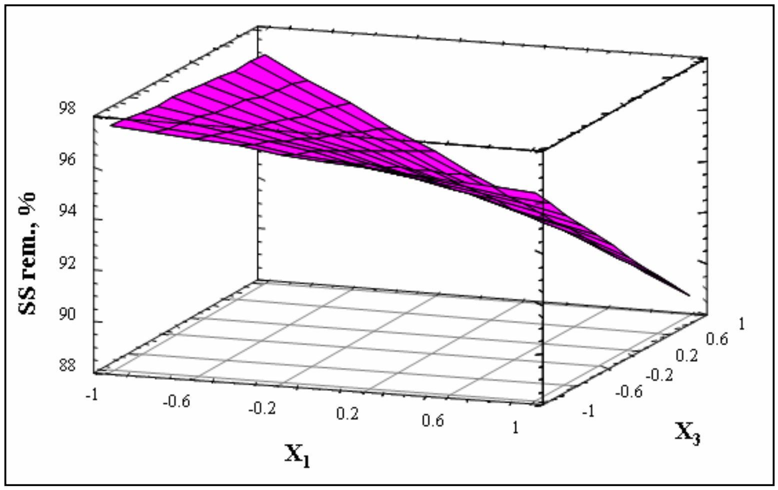

Figure B.11. Estimated response surface for SS removal, X_1 versus X_3

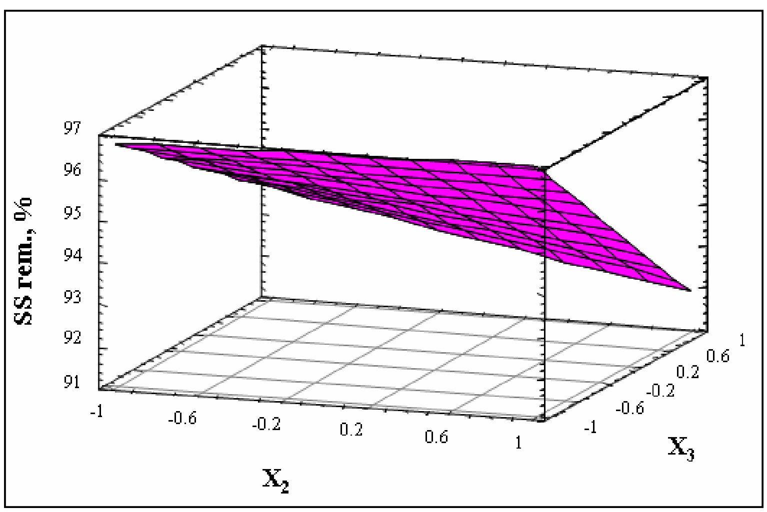

Figure B.12. Estimated response surface for SS removal, X_2 versus X_3

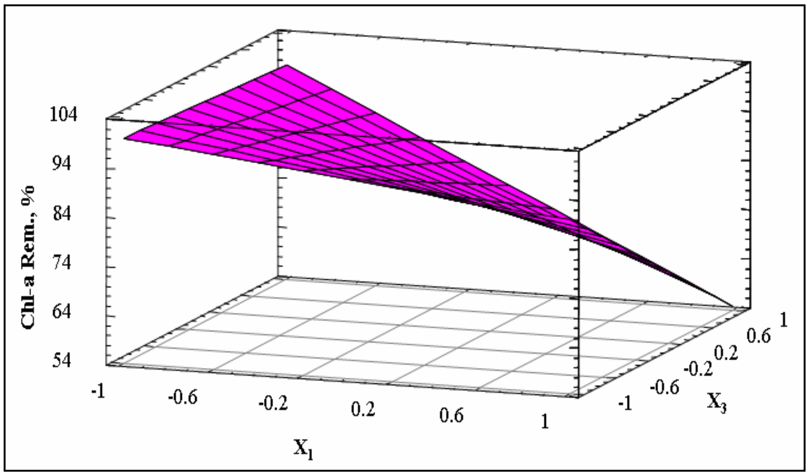

Figure B.13. Estimated response surface for Chl-a removal, X_1 versus X_3 (half-factorial)

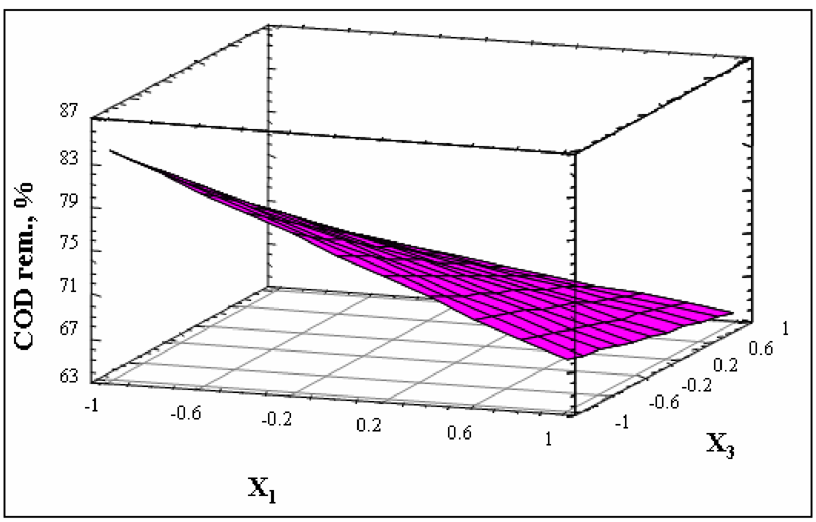

Figure B.14. Estimated response surface for COD removal, X_1 versus X_3 (half-factorial)

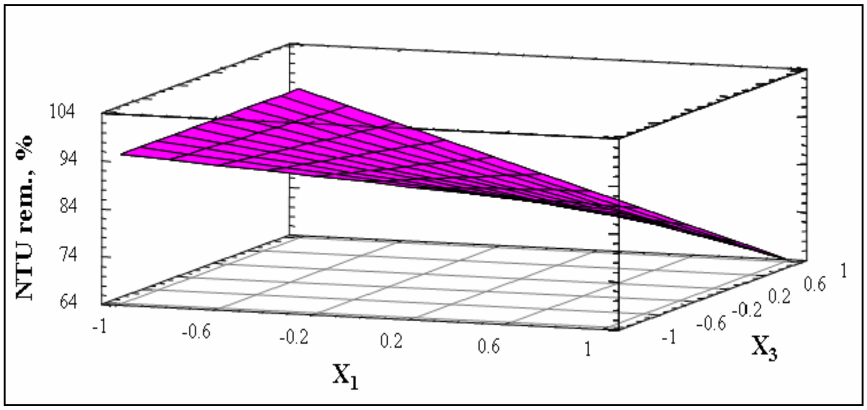

Figure B.15. Estimated response surface for turbidity removal, X_1 versus X_3 (half-factorial)

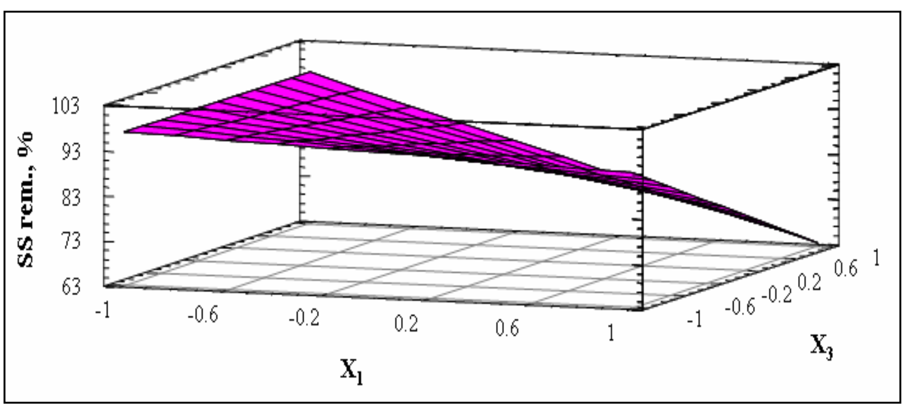

Figure B.16. Estimated response surface for SS removal, X_1 versus X_3 (half-factorial)