İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

SUSTAINABLE UPGRADING TECHNOLOGIES FOR RURAL WWT SYSTEMS - A CASE STUDY

M.Sc. Thesis by

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Department : **Environmental Engineering**

Programme: **Environmental Science and Engineering**

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

KIRSAL KESİM ATIKSU ARITMA SİSTEMLERİ İÇİN GELİŞTİRİLEBİLİR YENİLEME TEKNOLOJİLERİ – DURUM ÇALIŞMASI

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PREFACE

Firstly I would like to thank my supervisor, dear Prof. Lütfi AKÇA for his great support and help during my study. He showed an incredible understanding and a real concern to me all the time. Under his supervision and thanks to his vast expertise, it became possible for me to find the answers to the problems I have faced very quickly and continue on to the next step.

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December 2007 Gülsan Saraçoğlu

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SUMMARY

In these days, natural treatment systems are being developed and become common all over the world. Principally for small communities of rural areas which meet the field requirements of natural systems and have a population ranged between 2000 - 5000, natural treatment systems are more suitable than conventional treatment systems and recommended to use. Improving these systems and optimization play a significant role to meet today's needs both technically and economically.

This study is a part of the European Union's LIFE organization project named "Sakhnin Center as a Model for Environment Education and International Cooperation on Advanced Wastewater Treatment (A-WWT) in Rural Areas". In order to monitor performance of improvements, different systems with various conditions were tested. Anaerobic lagoons, facultative lagoon, seasonal reservoir and wetlands were designated as four different tasks which to be tested. Parameters such as pH, conductivity, COD, BOD, TSS, NH_4^+, PO_4^{-3} and TKN were measured according to standardized methods in each of these different tasks.

Covered anaerobic tanks were tested instead of conventional systems in anaerobic lagoons. By these plastic covers, pollutant reduction, elimination of odor, and improved biogas production were intended to be obtained as well as refined heated conditions. For Task 1, Covered tanks overall gave better results compared to uncovered tanks with exceptions for some individual parameters.

In Task 2 biofilters were considered as a replacement for facultative lagoons. Stabilized biomass from the biofilters for agricultural use, saving surface area in the WWT plant for other needs, and utilization of various vegetative wastes were aimed to be reached by biofilters. Different heights of tanks and different feedstock sizes were tested. Improved results were obtained from biofilter experiments compared to facultative lagoon.

To investigate some physical improvement, different shaped small-scale concrete reactors were constructed and filled with waste water form seasonal reservoir. Extented surface area and aeration were two major goal of Task 3. Improved biological activties were obtained on enlarged surface areas by plastic curtains. Tunnel shaped reactors with aeration gave the best result compared to hexagonal shaped reactors.

Different types of plants and different sized rocks were tested in tanks in Task 4 to obtain; removal of suspended solids, removal of BOD and COD, removal of nitrogen and phosphorus as well as permitting use of the water for drip irrigation without clogging interruptions or discharge of excess water to the environment without risking water resources. Small sized rocks yielded improved results in extended

attachment surface respect. Eventhough there is no clear separation obtained between different type of plants, reed and cane combination gave comparetively better results.

All tests were analyzed seperately within various retention times and results are given in text in detail.

KIRSAL KESİM ATIKSU ARITMA SİSTEMLERİ İÇİN GELİŞTİRİLEBİLİR YENİLEME TEKNOLOJİLERİ – DURUM ÇALIŞMASI

ÖZET

Son zamanlarda, gittikçe gelişmekte olan doğal arıtma sistemleri tüm dünyada yaygın bir hale gelmektedir. Özellikle doğal arıtma sistemlerinin arazi gereksinimlerini karşılayabilen ve nüfusu 2000 ila 5000 arasında olan küçük yerleşim birimleri için doğal arıtma sistemleri, standart arıtma sistemlerine göre daha uygundur. Günümüzün teknik ve ekonomik ihtiyaçlarını karşılamada bu sistemlerin geliştirilmesi ve optimizasyonu çok önemli bir rol oynamaktadır.

Bir Avrupa Birliği organizasyonu olan LIFE'ın bir projesi olan bu çalışma "Sakhnin Center as a Model for Environment Education and International Cooperation on Advanced Wastewater Treatment (A-WWT) in Rural Areas" olarak isimlendirilmektedir. Bu tez adı geçen projenin bir parçası olarak çalışılmıştır. Mevcut system üstüne yapılan geliştirmelerin performansının izlenmesi için değişik sistemler farklı koşullarda test edilmiştir. Anaerobik havuzlar, fakültatif havuz, mevsimsel rezervuar ve sulakalanlar dört ayrı çalışma için seçilmiştir. Tüm çalışmalarda pH, iletkenlik, KOİ, BOİ, AKM, NH4+, PO4-3 ve TKN gibi parametreler standart metotlara göre ölçülmüştür.

Çalışma dört aşamadan oluşmaktadır. Birinci aşamada, varolan anaerobik havuzlar yerine üzeri plastik malzeme ile kapatılan anaerobik tanklar kullanılmıştır. Bu plastik örtüler aracılığıyla, daha iyi ısıl koşulların yanısıra biogaz üretiminde artış, kötü kokuların giderilmesi ve kirletici miktarında azalma amaçlanmıştır. Bu birinci çalışmada üstü kapatılan tanklar, referans olarak üstü açık bırakılan tanklara oranla bazı parametreler haricinde daha iyi sonuçlar vermiştir.

İkinci çalışmada biyofiltreler fakültatif havuzlara alternatif olarak düşünülmüştür. Biyofiltre kullanılarak, stabilize biyokütle eldesi, atıksu arıtma tesislerinde ek arıtma sistemleri için yer tasarrufu ve bitkisel atıkların verimli kullanımı sağlanmaya çalışılmıştır. Farklı yüksekliklerde ve farklı boyutlarda ağaç parçaları ile doldurulmuş tanklar test edilmiş olup, biyofiltrelerin fakültatif havuzlara göre çok daha iyi sonuçlar verdiği gözlemlenmiştir.

Üçüncü çalışmada, fiziksel iyileştirmeleri gözlemleyebilmek için farklı şekillerde küçük ölçekli beton reaktörler inşa edilerek mevsimsel rezervuardan gelen atık suyla doldurulmuştur. Genişletilmiş yüzey alanı ve iyi bir havalandırma bu üçüncü çalışmadaki amaç olarak belirlenmiştir. Plastik perdeler aracılığıyla elde edilen genişletilmiş yüzey alanında iyileştirilmiş biyolojik aktiviteler gözlemlenmiştir. Havalandırmalı dikdörtgen şeklindeki reaktörlerin hekzagonal şeklinde olanlara nazaran daha iyi sonuçlar verdiği görülmüştür.

Dördüncü çalışmada farklı bitki türleri ve farklı boyutlardaki taşlar tanklarda denenerek askıda katı madde giderimi, KOİ ve BOİ giderimi, nitrogen ve fosfor giderimi ve verimli damlatmalı sulama sistemlerini uygulama ve tıkanmayı önleme amaçlanmıştır. Küçük ölçekli taşlar geniş tutunma yüzeyi sağlaması açısından daha iyi sonuçlar vermektedir. Sonuçlarda net bir farklılık yada avantaj görünmemesine karşılık, Kamış-Sazlık kombinasyonu şeklinde ekilen bitkilerin nispeten daha iyi sonuç veridiği görülmüştür

Bütün testlerde ayrı ayrı farklı bekleme süreleri uygulanmış ve sonuçlar detaylı olarak çalışmanın içeriğinde verilmiştir.

1. INTRODUCTION

Water is the most important thing for every living organism (mankind, animals, plants etc.). But day by day our water resources getting reduced and dirty, so it is easy to see if we do not care about this most important life resource, our life resource is going to become our natural killer.

Today, about 1 billion people do not have access to safe drinking water, and 2.6 billion have no adequate sanitation facilities (WHO 2004). In 1998, water-related diseases cause an estimated 3.4 million deaths, mostly children. The main killers are diarrhea (2.21 million) and malaria (1.11 million), trypanosomiasis, intestinal worm infections, dengue, and schistosomiasis.

While the amount of water on earth stays the same, demand for it is growing, putting stresses on arid countries and on the infrastructure in the world's rapidly growing cities. In the year 2000, 450 million people in 29 countries will suffer chronic water shortages, particularly in Africa and the Middle East. By 2050, some two-thirds of the world's population will be affected if current rates of consumption, population growth, and development continue.

In this purpose there are two main problems to solve:

- Using less amount of water for needs and reuse it if possible
- Keeping water resources clean

Nowadays, mankind's needs getting increased with developing technology day by day. But it means also using and polluting more water. Demand for water also shifts. As the countries of the world industrialize and urbanize, water use patterns change and competition grows. Industrial uses for water generate higher income and export earnings. As these uses take a larger share of the water, agriculture suffers. So to keep this life resource alive as long as possible, it should be taken care of.

In big and developed cities more advanced, complicated and expensive treatment plants are used to clean and reuse wastewater. It is a quite expensive process but it is a must. Maybe it is not a big problem to have, to operate and to maintenance for big and rich cities.

But even in really big and reach countries there are small rural communities which does not have enough fund to have, operate or maintenance this kind of big technological treatment plants also they need well trained staff. However it does not mean there is nothing to do for small rural areas at all.

1.1 The Significance of the Subject

Water scarcity and contamination of surface and groundwater are major regional Middle East problems. Water resources are insufficient to meet rising demands due to dramatic increases in population and water consumption. Lack of natural resource, planning, the inadequate maintenance of existing systems, and the absence of appropriate sewage treatment facilities have resulted in serious contamination of groundwater and soil. Wastewater treatment is desperately needed for the protection of freshwater sources. Reuse of treated wastewater is important for irrigation.

Most urban wastewater treatment systems are energy intensive. Such systems are costly and require complex mechanical equipment and highly skilled personnel. Attempted transfers of urban intensive technology to rural areas in the Middle East have failed.

Most of the Middle East countries are suffering from water problems. The most important factor causing this is the climatic and geographic conditions.

Outside of big cities there are plenty of little towns where these problems are raising day by day with additional economical and educational problems. So the major problem today is how these problems can be solved, with a view to saving both time and money.

As mentioned above since climatic and geographic conditions are big disadvantages for Middle East countries, there is no balance between water usage and natural annual recharge of rain. This problems comes with, extremely serious results. But the most important one is salinization by salt water intrusion which causes eliminating the fresh water sources. Increased salinity also causes extreme damage to the soil, reducing crop yields and possibly even leading to an increase in blood pressure in children (Zaslavsky, 2000). Leakage of sewage water also adds salts and heavy metals to fresh water resources.

Rural areas of the Middle East are in need, of wastewater treatment and reuse technologies that are appropriate to their climate, economy, and population. In contrast to intensive systems, extensive wastewater treatment technologies depend primarily on natural-components. Extensive systems are suitable for rural areas since they need low maintenance and simple to operate. They also require low investment costs, and their large land requirements are easily satisfied. Furthermore, these technologies are more efficient in pathogen removal, and therefore carry a greater ability to protect against the spread of disease. Yet, there are currently only a few extensive wastewater treatment systems existing in small, isolated settlements in rural areas in the region.

Sakhnin is one of the Arabic cities of Israel, placed in north of the country which has a population of 21000. Located in the Beit Natufa Basin, Sakhnin consists of 2,400 hectares of rich agricultural land, on which a majority of olive trees and seed crops are grown. Although most of Sakhnin's population is employed outside of the agricultural sector, approximately 3 percent of the population receives its main income from farming, and many others receive partial income from the sector. Many young people in Sakhnin have expressed their desire to farm full-time, but the current lack of available water for irrigation purposes prevents them from doing so.

1.2 Aim and Scope of the Study

The aim of the study is to upgrade the overall efficiency of the WWT system typical for rural areas, but without using expensive technologies that will increase significantly the level of maintenance needed routinely. This requires that the upgrading will limited to the addition of devices and systems which are selectively simple and do not require highly trained personal for maintaining and will make effective use of the existing infrastructures of the regional WWTP's.

LIFE is the EU’s financial instrument supporting environmental and nature conservation projects throughout the EU, as well as in some candidate, acceding and neighboring countries.

While many other EU funding programs have environmental components, LIFE has been the only program devoted entirely to supporting the development and implementation of environmental policy in the Member States of the European Union, in candidate countries who are associated to LIFE and in certain third countries bordering on the Mediterranean and the Baltic Seas.

This study is about LIFE’s “Sakhnin Center as a Model for Environment Education and International Cooperation on Advanced Wastewater Treatment (A-WWT) in Rural Areas” project. As name clears the aim of the project, this study is concerned with advanced wastewater treatment part.

The Sakhnin Regional Demonstration Centre (SRDC) was the first one of its kind in the Arab community of Israel and has several ongoing activities, including environmental planning, education and WWT. The SRDC’s activities are based around the operation of the local WWT plant, which treats effluent from about 70% of local households. Although basic infrastructure exists, there is a dire need to upgrade existing wastewater treatment facilities to produce improved quality effluent for local agricultural irrigation.

The scientific study was therefore divided into 4 main technical tasks. Each "task" is devoted to the upgrading of one of the four "traditional" steps of WWT of typical rural WWTP, which is actually a series of wastewater stabilization ponds. The existing WWTP of the city of Sakhnin is used as a model and a source for the effluents used for the experiments which are performed simultaneously in all of the four technical tasks, supplying the research subjects for the specific works of the students.

The scientific activity is concentrating deeply in each of the specific technologies used for WWT in the Sakhnin WWTP, as follows:

- Enhanced rate anaerobic digestion in controlled plastic covered ponds instead of the conventional settling-anaerobic ponds.
- Bio-filters, with fixed biomass activity replacing the conventional facultative ponds.
- Enhanced, plug-flow type treatment, with different combinations w/o curtain, bio-filters fixed media and/or aeration instead of conventional treatment in the seasonal reservoir.

- Additional "polishing" treatment of the seasonal reservoir effluents by constructed wetland (CWL) technology, utilizing plants with economicalvalue.

2. APPROPRIATE WWT SYSTEMS FOR RURAL COMMUNITIES (LITERATURE REVIEW)

2.1 Centralized Treatment Systems (Treatment of wastewater from small communities which gathered by sewer system CWT)

Centralized wastewater management has been the norm in municipal engineering circles for more than 100 years and centralized management is the structure of choice in most cities and counties.

A centralized wastewater management system consists of collection sewers and a centralized treatment facility. Hence CWT are used to collect and treat wastewater from entire communities.

Centralized treatment systems are also applicable for small rural areas by gathering the all wastewater in a centralized treatment plant by sewers. In Figure 2.1, the difference between centralized and decentralized systems can be seen clearly.

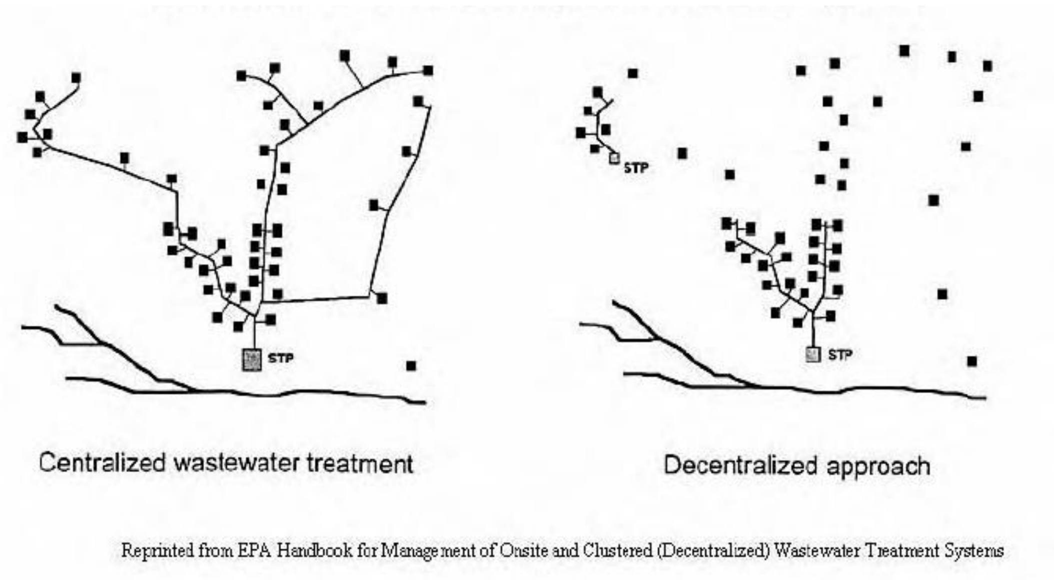

Figure 2.1: Centralized Wastewater Treatment vs. Decentralized Approach (USEPA,2006).

CWT facilities benefits from a wide variety of technologies to treat wastes and wastewater generated on site.

Current processes for the CWT may be divided into three main categories of primary treatment, secondary treatment, and tertiary treatment, Primary treatment of wastewater consists of the removal of insoluble matter such as grit, grease, scum and settable solids from water. The first step in primary treatment normally is screening. Screening maybe used in conjunction with grinding and removes or reduces the size of large objects that get into the sewage system. These solids are collected on screens and scraped off for subsequent disposal. Most screens are cleaned with power rakes (Manahan, 2001).

In general The CWT technologies currently in use can be grouped into the following three main categories. One or several of the technologies below are used together in a sewage treatment system:

- Physical /Chemical/Thermal Treatment
    - Neutralization
    - Flocculation/Coagulation
    - Emulsion Breaking
    - Gravity Assisted Separation
        - Gravity Oil/Water Separation
        - Clarification
        - Dissolved Air Flotation
    - Chromium Reduction
    - Cyanide Destruction
    - Chemical Precipitation
    - Filtration
    - Carbon Adsorption
    - Ion Exchange
    - Stripping
- Biological Treatment
    - Sequencing Batch Reactors
    - Attached Growth Biological Treatment Systems
        - Trickling Filters
        - Biotowers

- Activated Sludge
- Sludge Treatment and Disposal
  - Plate and Frame Pressure Filtration
  - Belt Pressure Filtration
  - Vacuum Filtration
  - Filter Cake Disposal

2.1.1 Physical/Chemical/Thermal Treatment

Both strength and volume of the wastes may vary depending on the site, where the wastes are received. Therefore CTW facilities generally need to equalize wastes by holding them in an equalization tank for a certain period of time, consolidate small waste volumes and to minimize the variability of incoming wastes before treatment, in order to obtain a stable waste stream which is easier to treat. A waste stream with more uniform pollutant content, results in more predictable and uniform treatment results. Equalization is not a treatment process but a technique that improves the effectiveness of secondary and advanced wastewater treatment processes (Carl E, 1999).

Equalization tanks are commonly equipped with agitators or aerators to mix the wastewater and to prevent suspended solids from settling to the bottom of the unit in the desired area. The mixing of acid and alkaline wastes is an example of effective equalization. Figure 2.2 illustrates an equalization system (USEPA, 1997).

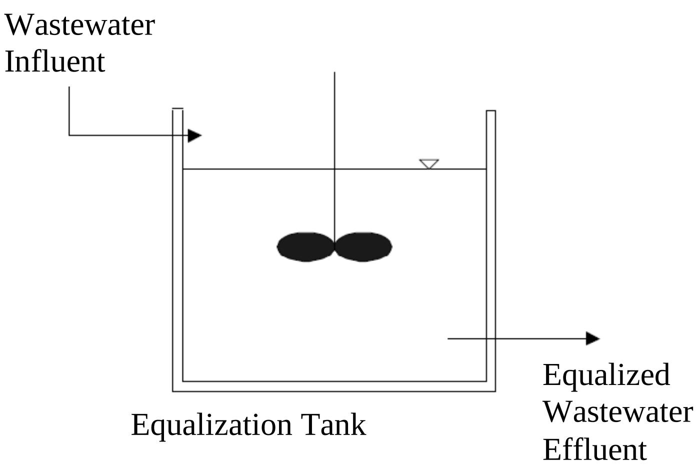

Figure 2.2: Equalization system

2.1.1.1 Neutralization

pH values of wastewaters treated at CWT facilities vary depending on the types of wastes accepted. Untreated wastewater may require neutralization to eliminate either high or low pH values prior to certain treatment systems, such as biological treatment. Neutralization is the restoration of the hydrogen or hydroxyl ion balance in solution so that the ionic concentrations of each are equal (Carl E, 1999).

Facilities often use neutralization systems also in conjunction with certain chemical treatment processes, such as chemical precipitation, to adjust the pH of the wastewater to optimize treatment efficiencies.

Neutralization may be performed in a holding tank, rapid mix tank, or an equalization tank. Typically, facilities use neutralization systems at the end of a treatment system to control the pH of the discharge to between 6 and 9 in order to meet pretreatment limitations (USEPA, 1998). This is not a common approach for a sewage treatment system.

2.1.1.2 Flocculation / Coagulation

"Coagulation" is the reduction of the net electrical repulsive forces at particle surfaces by addition of coagulating chemicals, whereas "Flocculation" is the agglomeration of the destabilized particles by chemical joining and bridging. This process begins in the aeration tank and is the basic mechanism for removal of suspended matter in the final clarifier (Spellman, 2003).

Flocculation process increases the performance of a sedimentation or filtration treatment system by increasing particle size resulting in increased settling rates and filter capture rates (USEPA, 1997).

The waste stream is initially mixed while a coagulant and/or a coagulant aid is added. After mixing, the coagulated wastewater flows into a flocculation basin where slow mixing of the waste occurs. The slow mixing allows the particles to agglomerate into heavier, more settleable/filterable solids. Either mechanical paddle mixers or diffused air provides mixing.

The figure below presents a diagram of a clarification system incorporating coagulation and flocculation (USEPA, 1998).

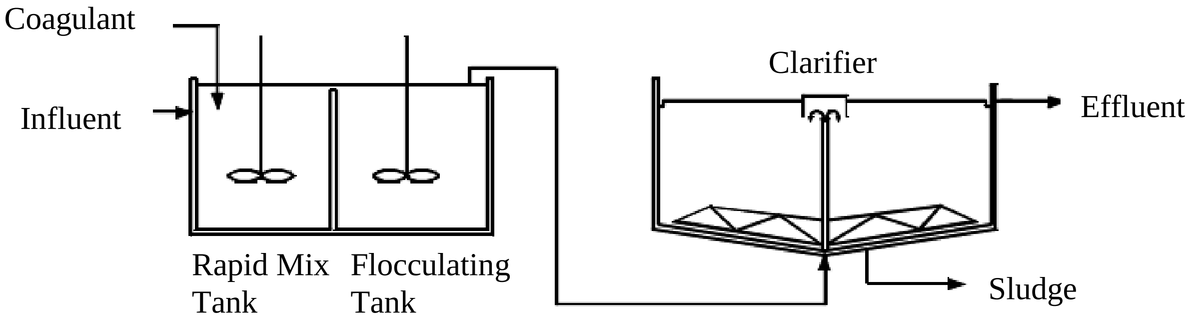

Figure 2.3: Clarification System Incorporating Coagulation And Flocculation

2.1.1.3 Emulsion Breaking

Emulsion breaking is a process used to treat emulsified oil/water. There are two types of emulsion, one of them is stable emulsion where small droplets of oil are dispersed within the water and are prevented from coalescing by repulsive electrical surface charges that are often a result of the presence of emulsifying agents and/or surfactants or unstable. The other type is unstable emulsion where dispersion and settling is very rapid hence there is no need to break the emulsion.

Several physical methods can separate oils and SS from wastewater, including gravity separation, dissolved air flotation, centrifugation, filtration, and electrical dehydration. Selecting the method depends on the nature of the wastewater and the degree of treatment required.

Emulsion breaking is achieved through the addition of chemicals and/or heat to the emulsified oil/water mixture. Chemical methods of breaking water–oil emulsions are based on the addition of chemicals that destroy the protective action of hydrophobic or hydrophilic emulsifying agents and allow the water globules and oil to coalesce. The most commonly-used method is acid-cracking where sulfuric or hydrochloric acid is added to the oil/water mixture until the pH value reaches 1 or 2 (Carl E, 1999). This is not a common approach for a sewage treatment system.

2.1.1.4 Gravity Assisted Separation

A. Gravity Oil/Water Separation

Unlike emulsion breaking, gravity separation is only effective for the bulk removal of free oil and grease. It is not effective in the removal of emulsified or soluble oils. Typically CWT facilities use gravity separation in conjunction with emulsion breaking. Because gravity separation is such a widely used technology, there is abundance of equipment configurations. A very common unit is the API (American Petroleum Institute) separator, shown in Figure 2.4 (USEPA, 1997).

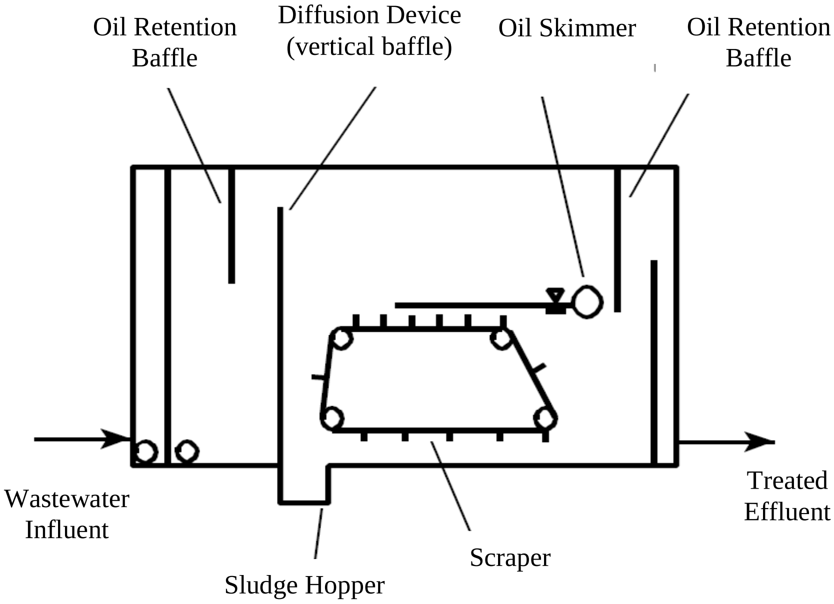

Figure 2.4: Gravity Separation Unit

B. Clarification

The purpose of primary treatment (primary sedimentation or primary clarification) is to remove settleable organic and floatable solids. Normally, each primary clarification unit can be expected to remove 90 to 95% settleable solids, 40 to 60% TSS, and 25 to 35% BOD (Spellman, 2003). In a clarifier, wastewater is allowed to flow slowly and uniformly, permitting the solids more dense than water to settle to the bottom. The clarified wastewater is discharged by flowing from the top of the clarifier over a weir. Solids accumulate at the bottom of the clarifier hence sludge must be periodically removed, dewatered and disposed. The next figure (Figure 2.5) presents a circular clarification system (USEPA, 1997). For Clarification System Incorporating Coagulation and Flocculation see Figure 2.3.

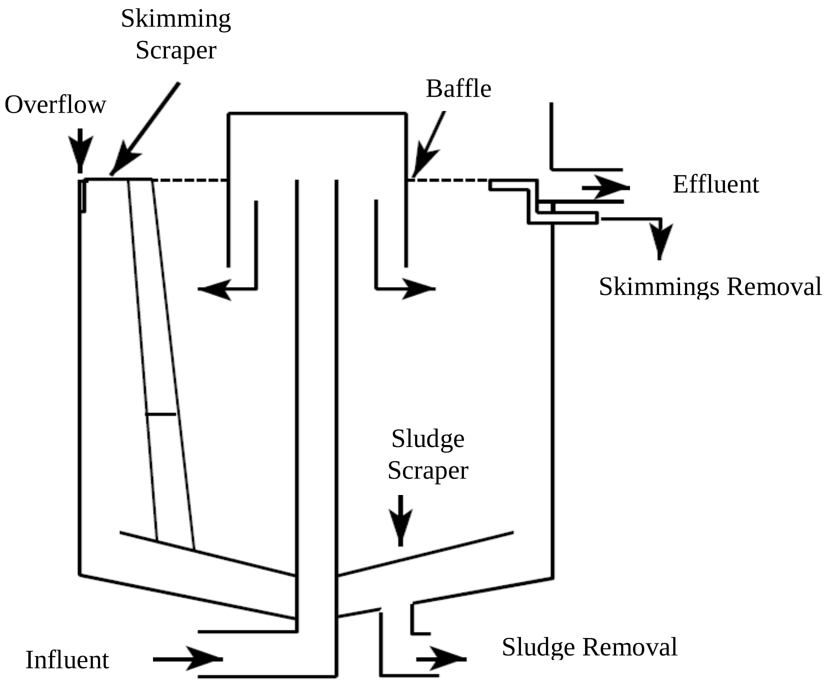

Figure 2.5: Circular Clarification System

C. Dissolved Air Flotation

Flotation is the process of using fine bubbles to induce suspended particles to rise to the surface of a tank where they can be collected and removed. Gas bubbles are introduced into the wastewater and attach themselves to the particles, thereby reducing their specific gravity and causing them to float.

Fine bubbles may be generated by dispersing air mechanically, by drawing them from the water using a vacuum, or by forcing air into solution under elevated pressure followed by pressure release (USEPA, 2005).

This process is commonly used to remove suspended solids and dispersed oil and grease from oily wastewater. It may effectively reduce the sedimentation times of suspended particles that have a specific gravity close to that of water. Use of a gas other than air is referred to as “dissolved gas flotation” or “DGF” (Pankratz, 2001).

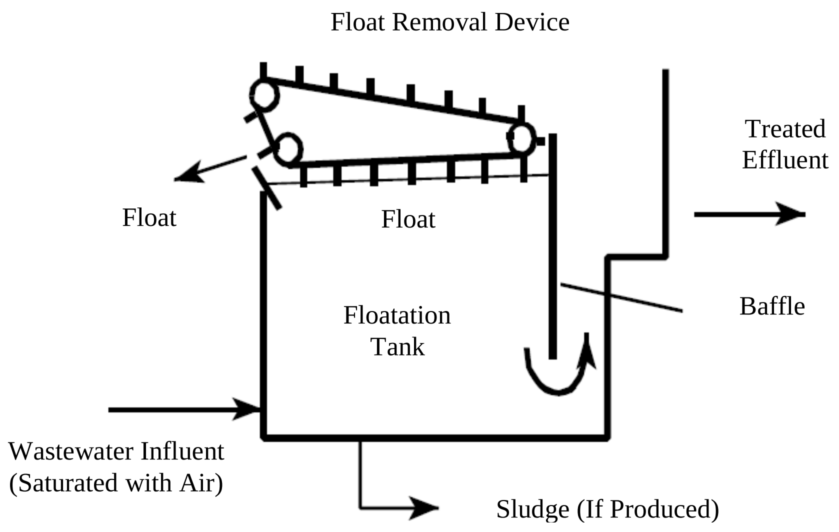

Figure 2.6: Flotation Process

2.1.1.5 Chromium Reduction

Reduction is a chemical reaction in which electrons are transferred from one chemical to another. The main reduction application at CWT facilities is the reduction of hexavalent chromium to trivalent chromium, which is subsequently precipitated from the wastewater in conjunction with other metallic salts (USEPA, 2005).

Once the chromium has been reduced to the trivalent state, it can be further treated in a chemical precipitation process, where it is removed as a metal hydroxide or sulfide (USEPA, 1997). This is not a common approach for a sewage treatment system. A typical chromium reduction process is shown in the Figure 2.7.

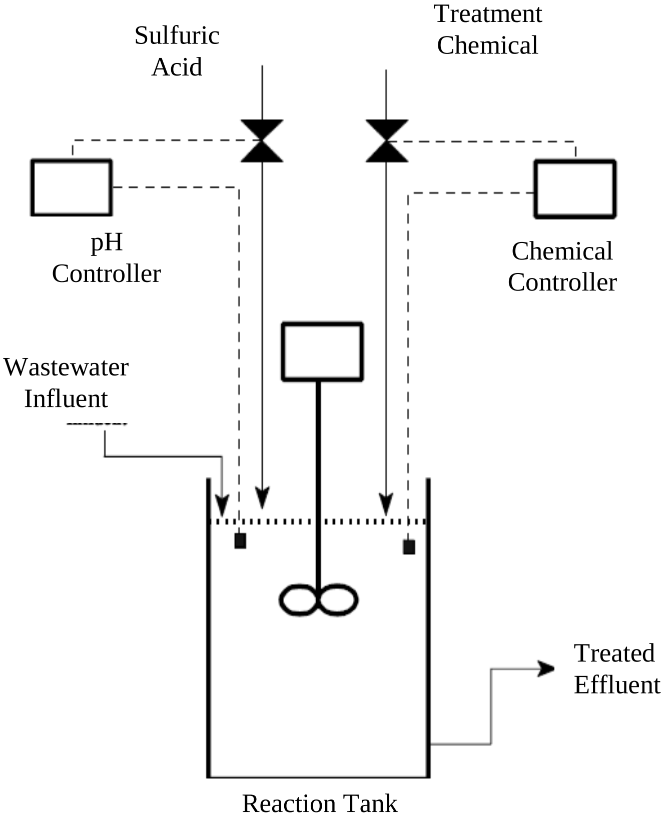

Figure 2.7: Chromium Reduction Process

2.1.1.6 Cyanide Destruction

Electroplating and metal finishing operations produce the major portion of cyanide-bearing wastes accepted at CWT facilities.

The destruction of the cyanide takes place in two stages. The primary reaction is the partial oxidation of the cyanide to cyanate at a pH above 9. In the second stage, the pH is lowered to a range of 8 to 8.5 for the oxidation of the cyanate to nitrogen and carbon dioxide (as sodium bicarbonate) (USEPA, 2005). This is not a common approach for a sewage treatment system.

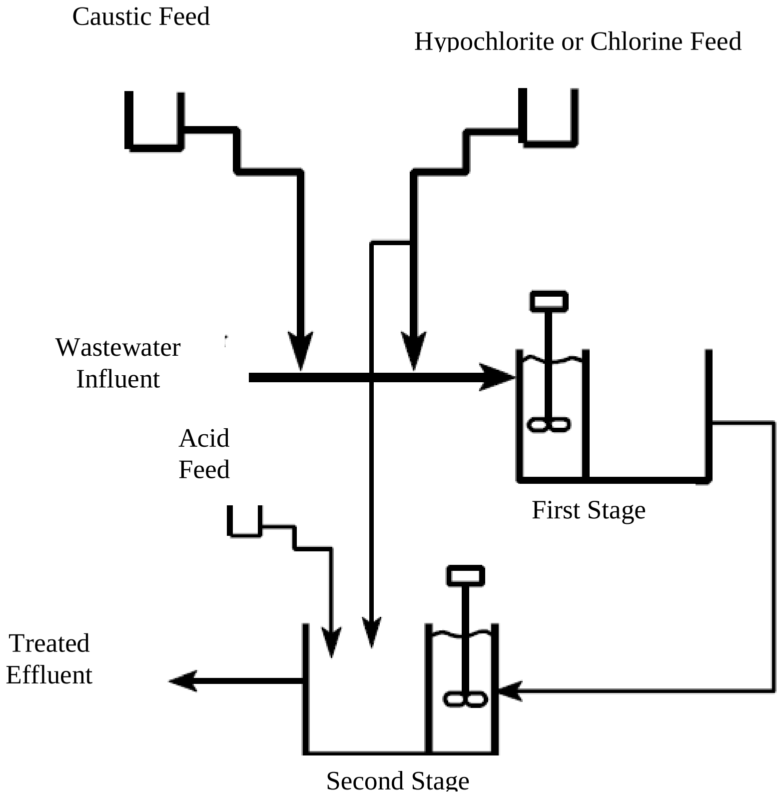

Figure 2.8: Cyanide Destruction

2.1.1.7 Chemical Precipitation

Many CWT facilities use chemical precipitation to remove metal compounds from wastewater. Chemical precipitation converts soluble metallic ions and certain anions to insoluble forms, which precipitate from solution. Chemical precipitation is usually performed in conjunction with coagulation/flocculation processes (USEPA, 1997).

This process step can reduce the SS up to 85%. The accumulated chemical sludge is removed by gravity flow or pumping to conditioning or disposal or both. The chemicals and sewage are flash-mixed in a mixing tank that has only a few minutes detention time followed by 30 to 90 min detention in a flocculation tank that is slowly agitated to aid floc growth (Carl E, 1999).

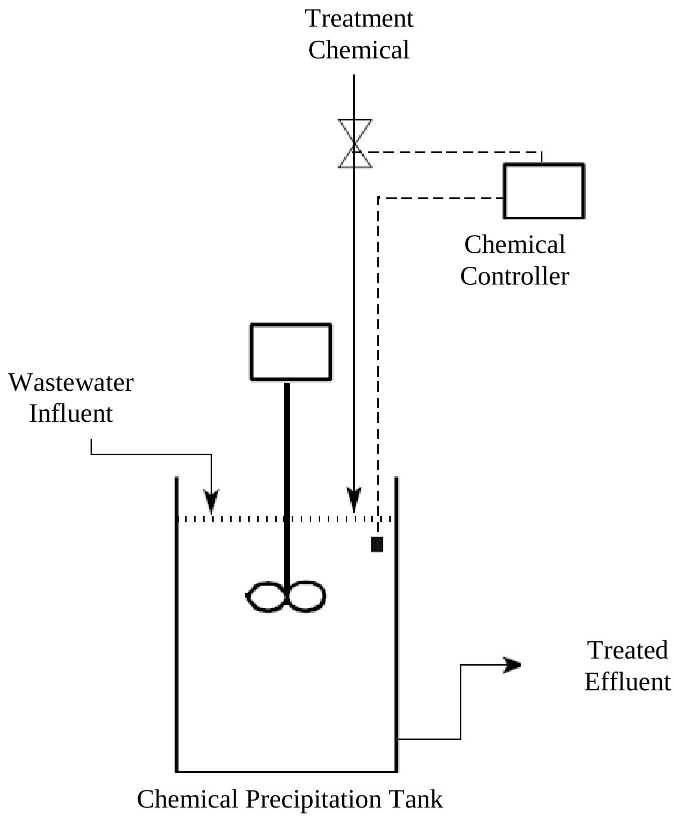

Figure 2.9: Chemical Precipitation System Diagram

2.1.1.8 Filtration

Filtration is a method for separating solid particles from a fluid through the use of a porous medium.

As a general description of the process, wastewater flows to a filter (gravity or pressurized). The filter contains single, dual, or multimedia. Wastewater flows through the media, which removes solids. The solids remain in the filter. Backwashing the filter as needed removes trapped solids. Backwash solids are returned to the plant for treatment. Processes typically remove 95 to 99% of the suspended matter (Spellman, 2003).

There are various types of filtration in use at CWT facilities.

A. Sand Filtration

Sand filtration processes consist of either a fixed or moving bed of media that traps and removes suspended solids from water passing through the media. There are two types of fixed sand bed filters; pressure and gravity. Pressure filters contain media in an enclosed, watertight pressure vessel and require a feed pump to force the water through the media. A gravity filter operates on the basis of differential pressure of a static head of water above the media, which causes flow through the filter. Filter loading rates for sand filters are typically between 2 to 6 gpm/sq-ft (USEPA, 2005).

B. Multimedia Filtration

In granular bed filtration, the wastewater stream is sent through a bed containing two or more layers of different granular materials. The solids are retained in the voids between the media particles while the wastewater passes through the bed. Typical media used in granular bed filters include anthracite coal, sand, and garnet (USEPA, 1997.

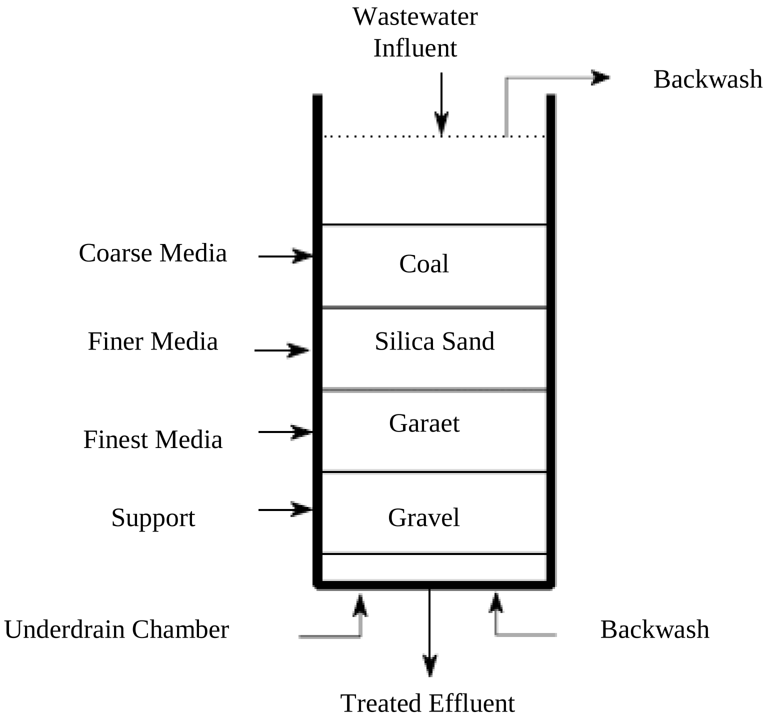

Figure 2.10: Multimedia Filtration

C. Plate and Frame Pressure Filtration

Another filtration system for the removal of solids from waste streams is a plate and frame pressure filtration systems. Although plate and frame filter presses are more commonly used for dewatering sludges, they are also used to remove solids directly from wastewater streams (USEPA, 2005).

Sludges are then dewatered to 30 to 50 percent solids by weight using a plate and frame filter. Sludges from treatment systems can be thickened by gravity or stabilized prior to dewatering, or may be processed directly with the plate and frame pressure filtration unit (USEPA, 1997).

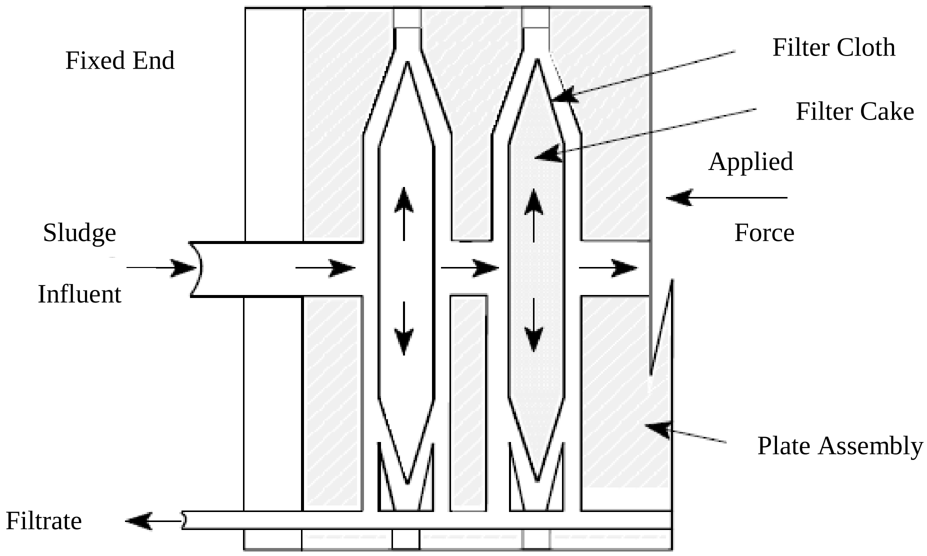

Figure 2.11: Plate And Frame Pressure Filtration

D. Membrane Filtration

Surface filtration at high pressures (50 to 1000 psig) and low flow rates through the films or dynamically formed membranes is termed membrane filtration. This process employs semi-permeable membranes and a pressure differential to remove solids in wastestreams. Reverse osmosis and ultrafiltration are two commonly-used membrane filtration processes (Carl E, 1999).

Ultrafiltration

CWT facilities commonly use ultrafiltration (UF) for the treatment of metal-finishing wastewater and oily wastes. It can remove substances with molecular weights greater than 500, including suspended solids, oil and grease, large organic molecules, and complexed heavy metals. UF can be used when the solute molecules are greater than ten times the size of the solvent molecules, and are less than one-half micron (USEPA, 2005).

Tighter or less porous ultramembranes, with flux rates (hydraulic loadings) initially ranging from 50 to 300 gpd per sq ft at 50 psig, which are capable of rejecting high-molecular-weight (soluble, organic substances, but not salt) be used in UF (Carl E, 1999).

Reverse Osmosis

Reverse osmosis (RO) is a process for separating dissolved solids from water. CWT facilities commonly use RO in treating oily or metal-bearing wastewater. RO is applicable when the solute molecules are approximately the same size as the solvent molecules. A semi–permeable, microporous membrane and pressure are used to perform the separation. RO systems are typically used as polishing processes, prior to final discharge of the treated wastewater (USEPA, 2005).

Specially prepared membranes or hollow fibers with flux rates at 5 to 50 gpd per sq ft at 400 to 800 psig affect salt, soluble organic matter, colloidal or soluble silica, and phosphate removal at 80 to 95% efficiency.

All membrane processes are considered to be final polishing filters, with common particulate removals in excess of 99%. In so doing, they foul easily, and their flux flow rate declines logarithmically with running time. Therefore, wastewater treatment facilities must protect membrane filters from fouling by pretreating the feeds using coagulation and rough filtration (Carl E, 1999).

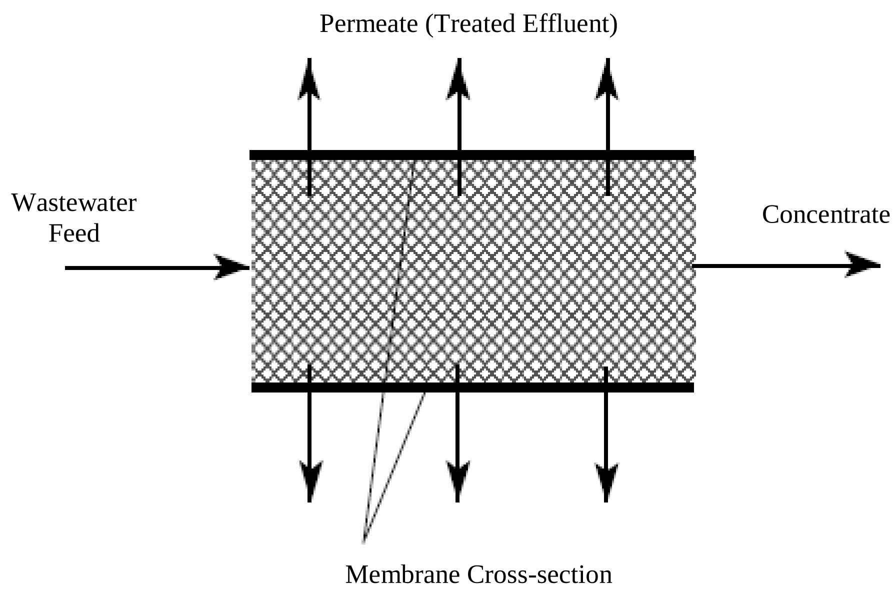

Figure 2.12: Ultrafiltration System Diagram

2.1.1.9 Carbon Adsorption

Activated carbon adsorption is a demonstrated wastewater treatment technology that uses activated carbon to remove dissolved organic pollutants from wastewater. Granular activated carbon adsorption (GAC) is a physical separation process in which organic and inorganic materials are removed from wastewater by adsorption, attraction, and/or accumulation of the compounds on the surface of the carbon granules. While the primary removal mechanism is adsorption, the activated carbon also acts as a filter for additional pollutant removal. Adsorption capacities of 0.5 to 10 percent by weight are typical (USEPA, 1997).

The main purpose of carbon adsorption used in advanced treatment processes is the removal of refractory organic compounds (non-BOD) and soluble organic material that are difficult to eliminate by biological or physical or chemical treatment.

In the carbon adsorption process, wastewater passes through a container filled either with carbon powder or carbon slurry. Organics adsorb onto the carbon (i.e., organic molecules are attracted to the activated carbon surface and are held there) with sufficient contact time.

A carbon system usually has several columns or basins used as contactors. Most contact chambers are either open concrete gravity-type systems or steel pressure containers applicable to either upflow or downflow operation (Spellman, 2003).

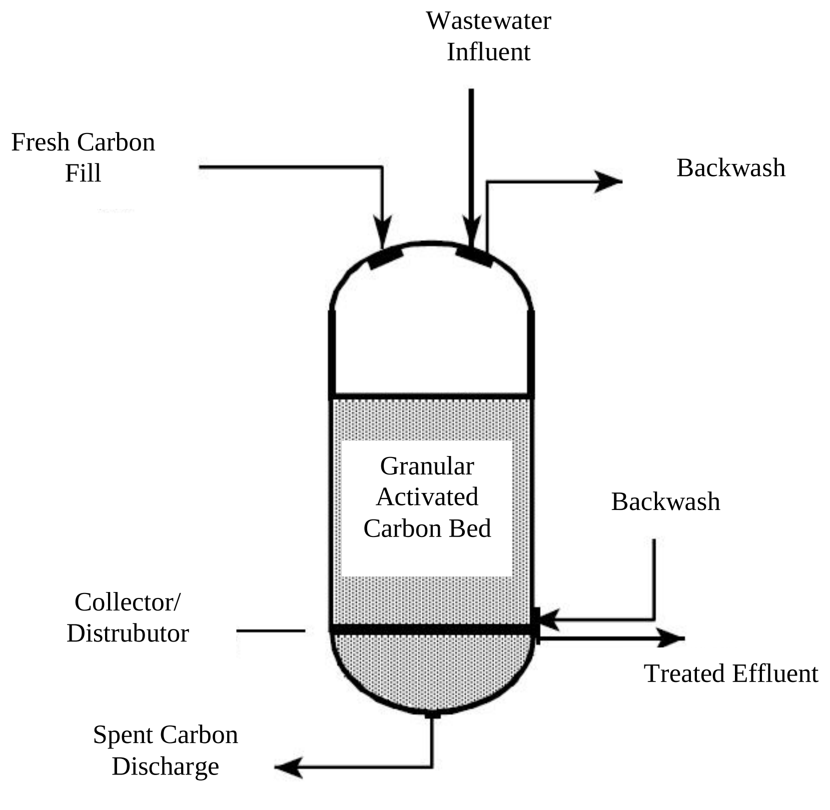

Figure 2.13: Carbon Adsorption System Diagram

2.1.1.10 Ion Exchange

The ion exchange process used for removal of inorganics consists of passing the water successively over a solid cation exchanger and a solid anion exchanger, which replace cations and anions by hydrogen ion and hydroxide ion, respectively, so that each equivalent of salt is replaced by a mole of water (Stanley E, 2001).

A key advantage of the ion exchange process is that the metal contaminants can be recovered and reused. Another advantage is that ion exchange may be designed to remove certain metals only, providing effective removal of these metals from highly contaminated wastewater. A disadvantage is that the resins may be fouled by some organic substances (USEPA, 2005).

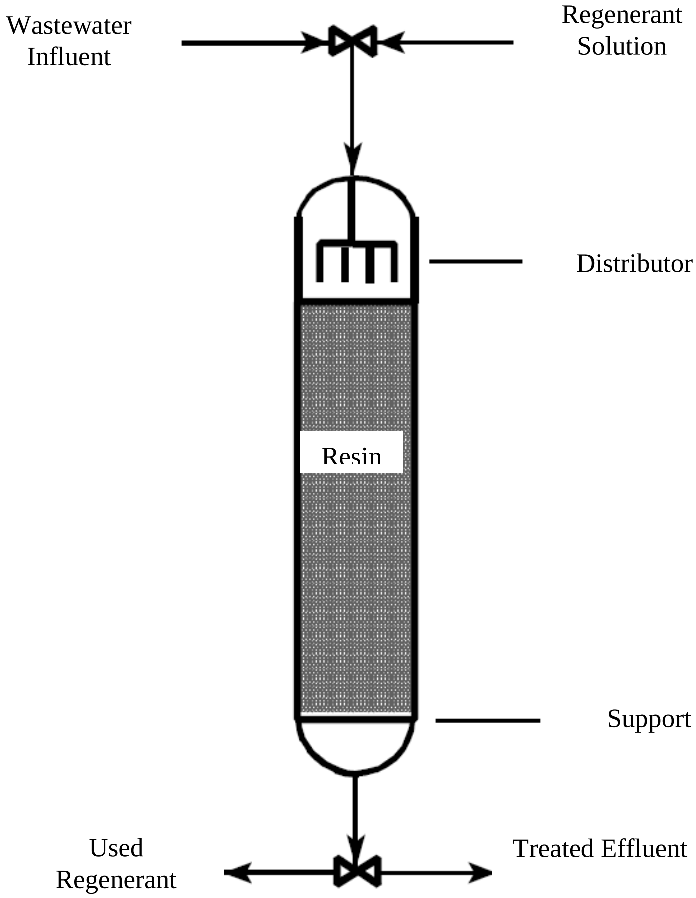

Figure 2.14: Ion Exchange System Diagram

2.1.1.11 Stripping

Stripping is a method for removing dissolved volatile organic compounds from wastewater. The removal is accomplished by passing air or steam through the agitated waste stream.

The primary difference between air stripping and steam stripping is that steam stripping is operated at higher temperatures and the resultant off-gas stream is usually condensed and recovered or incinerated. The off-gas from air stripping contains non-condenseable air which must be either passed through an adsorption unit or incinerated in order to prevent transfer of the volatile pollutants to the environment.

2.1.2 Biological Treatment

Biological treatment systems use microbes which consume, and thereby destroy, organic compounds as a food source. The microbes use the organic compounds as both a source of carbon and as a source of energy. These microbes may also need supplemental nutrients for growth, such as nitrogen and phosphorus, if the waste stream is deficient in these nutrients. Aerobic microbes require oxygen to grow, whereas anaerobic microbes will grow only in the absence of oxygen. Facultative microbes are an adaptive type of microbe that can grow with or without oxygen.

The success of biological treatment is dependent on many factors, such as the pH and temperature of the wastewater, the nature of the pollutants, the nutrient requirements of the microbes, the presence of inhibiting pollutants, and variations in the feed stream loading. Certain compounds, such as heavy metals, may be toxic to the microorganisms and must be removed from the waste stream prior to biological treatment.

There are several adaptations of biological treatment. These adaptations differ in three basic ways. First, a system may be aerobic, anaerobic, or facultative. Second, the microorganisms may either be attached to a surface (as in a trickling filter), or be unattached in a liquid suspension (as in an activated sludge system). Third, the operation may be either batch or continuous.

2.1.2.1 Sequencing Batch Reactors

The sequencing batch reactor (SBR) is a single, fill-and-draw, completely-mixed reactor that operates under batch conditions. Recently, SBRs have emerged as an innovative wastewater treatment technology (Irvine and Ketchum, 1989). SBRs are unique in that a single tank acts as an equalization tank, an aeration tank, and a clarifier.

An SBR is operated on a batch basis where the wastewater is mixed and aerated with the biological floc for a specific period of time. The contents of the basin are allowed to settle and the supernatant is decanted. The batch operation of an SBR makes it a useful biological treatment option for the CWT industry, where the wastewater volumes and characteristics are often highly variable. Each batch can be treated differently depending on waste characteristics (USEPA, 2005).

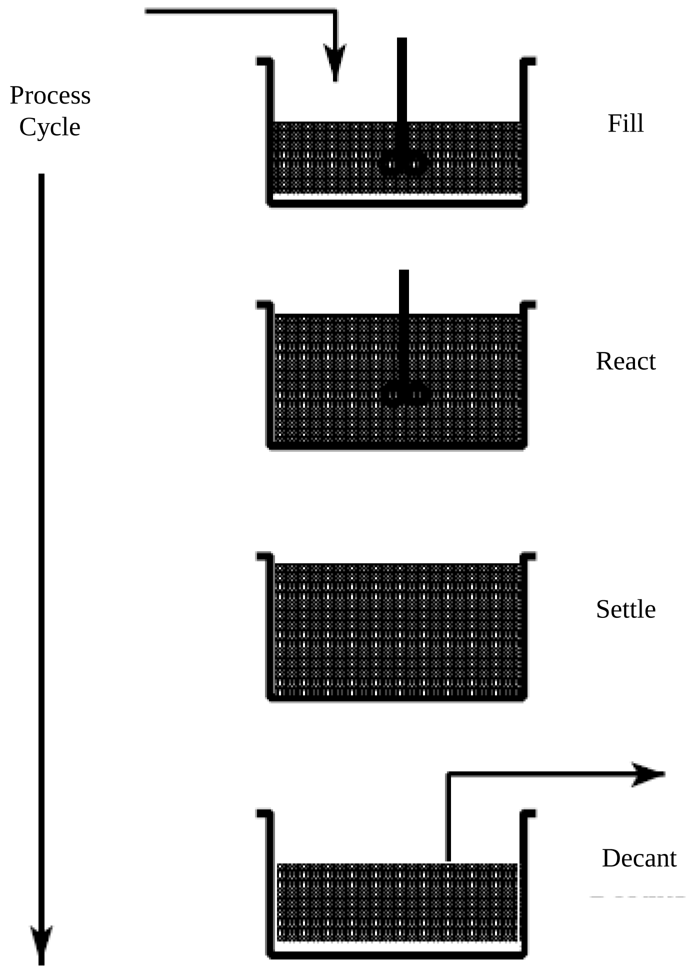

Figure 2.15: Sequencing Batch Reactor System Diagram

2.1.2.2 Attached Growth Biological Treatment Systems

Another system used to biodegrade the organic components of a wastewater is the attached growth biological treatment system. In these systems, the biomass adheres to the surfaces of rigid supporting media. As wastewater contacts the supporting medium, a thin-film biological slime develops and coats the surfaces.

As this film (consisting primarily of bacteria, protozoa, and fungi) grows, the slime periodically breaks off the medium and is replaced by new growth. This phenomenon of losing the slime layer is called sloughing and is primarily a function of organic and hydraulic loadings on the system. The effluent from the system is usually discharged to a clarifier to settle and remove the agglomerated solids (USEPA, 2005).

The two major types of attached growth systems used at CWT facilities are trickling filters and biotowers.

A. Trickling Filters

Trickling filters have been used for wastewater treatment for nearly 100 years. It was found that if settled wastewater was passed over rock surfaces, slime grew on the rocks and the water became cleaner.

A trickling filter (see Figure 2.16) is an attached-growth, biological process that uses an inert medium to attract microorganisms, which form a film on the medium surface.

A rotatory or stationary distribution mechanism distributes wastewater from the top of the filter percolating it through the interstices of the film-covered medium. As the wastewater moves through the filter, the organic matter is adsorbed onto the film and degraded by a mixed population of aerobic microorganisms. The oxygen required for organic degradation is supplied by air circulating through the filter induced by natural draft or ventilation (Carl E, 1999).

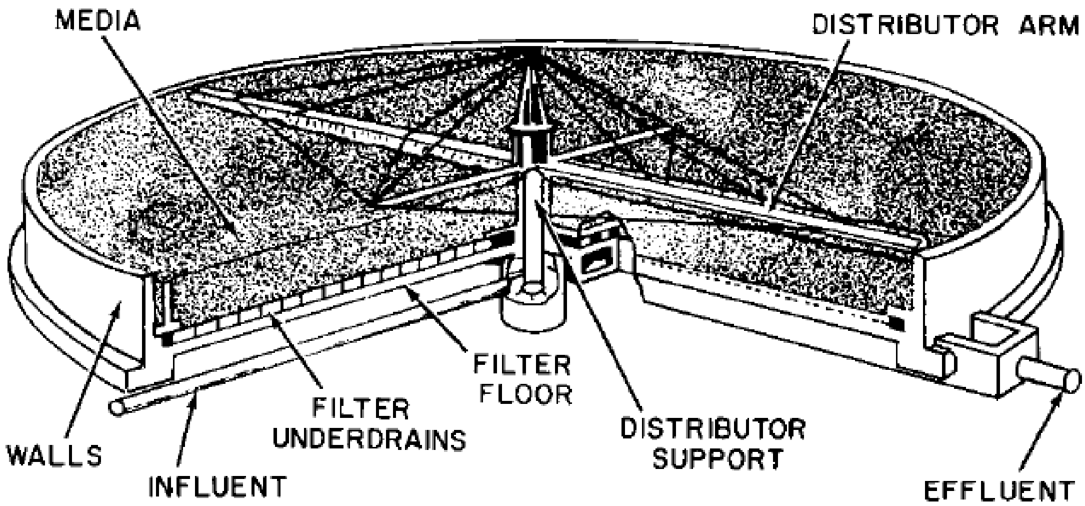

Figure 2.16: Cross section of a stone media trickling filter.

B. Biotowers

A variation of a trickling filtration process is the aerobic biotower. Biotowers may be operated in a continuous or semi-continuous manner and may be operated in an upflow or downflow manner. In the downflow mode, influent is pumped to the top of a tower, where it flows by gravity through the tower. The tower is packed with plastic or redwood media containing the attached microbial growth. Biological degradation occurs as the wastewater passes over the media. Treated wastewater collects in the bottom of the tower.

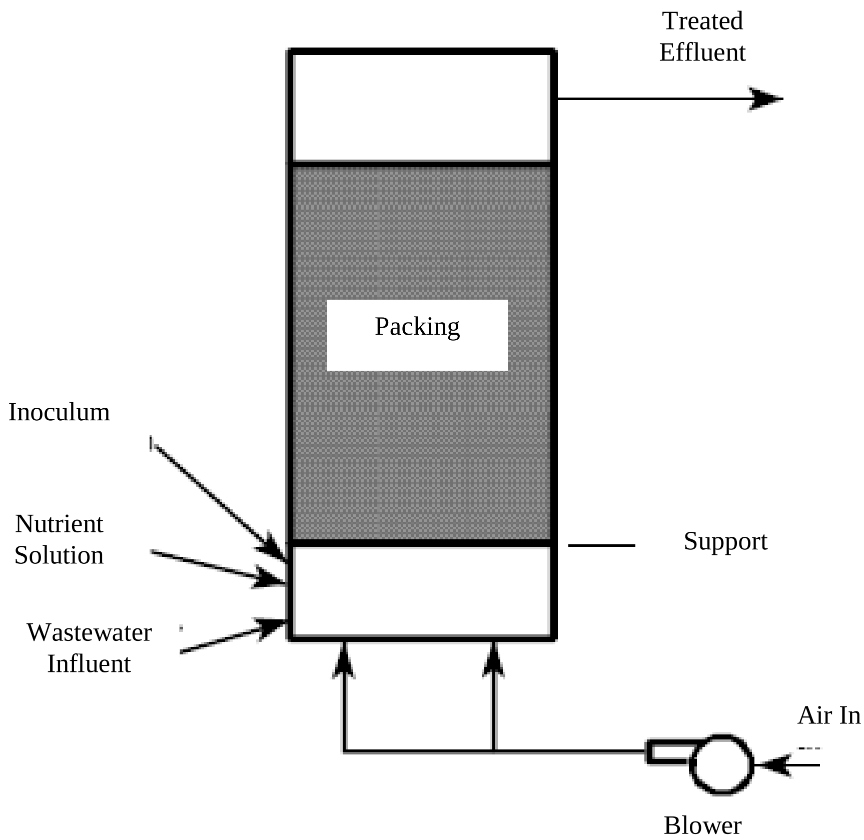

Figure 2.17: Biotower System Diagram

2.1.2.3 Activated Sludge

The activated sludge process is a continuous-flow, aerobic biological treatment process in which suspended-growth aerobic microorganism biodegrades organic contaminants.

In this process, a suspension of aerobic microorganisms is maintained by mechanical mixing or turbulence induced by diffused aerators in an aeration basin. This suspension of microorganisms is called the mixed liquor (USEPA, 2005).

Microorganisms in the aeration tank convert organic material in wastewater to microbial biomass and CO_2. Organic nitrogen is converted to ammonium ion or nitrate. Organic phosphorus is converted to orthophosphate.

The activated sludge is subsequently separated from the treated mixed liquor by sedimentation and is returned to the process as needed. The treated wastewater overflows the weir of the settling tank in which separation from the sludge takes place (Spellman, 2003).

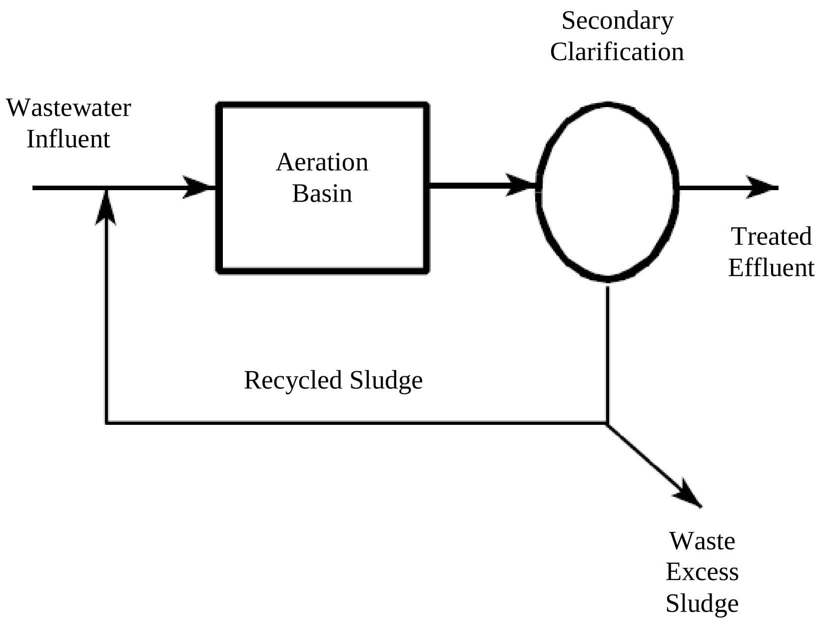

Figure 2.18: Activated Sludge System Diagram

2.1.2.4 Sludge Treatment and Disposal

Several of the waste treatment processes used in the CWT industry generate sludge. These processes include chemical precipitation of metals, clarification, filtration, and biological treatment.

There are several widely-used, treatment methods for sludge dewatering. Plate and frame pressure filtration, belt pressure filtration, and vacuum filtration are the primary methods used for sludge dewatering at CWT facilities.

2.1.2.5 Plate and Frame Pressure Filtration

Plate and frame pressure filtration systems are a widely used method for the removal of solids from waste streams. In the CWT industry, plate and frame pressure filtration system are used for filtering solids out of treated wastewater streams and sludges.

A pressure filter consists of a series of screens upon which the sludge is applied under pressure. A precoat material may be applied to the screens to aid in solids removal. The applied pressure forces the liquid through the screen, leaving the solids to accumulate behind the screen. Filtrate which passes through the screen media is typically recirculated back to the head of the on-site wastewater treatment plant (USEPA, 2005).

2.1.2.6 Belt Pressure Filtration

A belt pressure filtration system uses gravity followed by mechanical compression and shear force to produce a sludge filter cake. Belt filter presses are continuous systems which are commonly used to dewater biological treatment sludge. Most belt filter installations are preceded by a flocculation step, where polymer is added to create a sludge which has the strength to withstand being compressed between the belts without being squeezed out.

The advantages of a belt filtration system are its lower labor requirements and lower power consumption. The disadvantages are that the belt filter presses produce a poorer quality filtrate, and require a relatively large volume of belt wash water (USEPA, 2005).

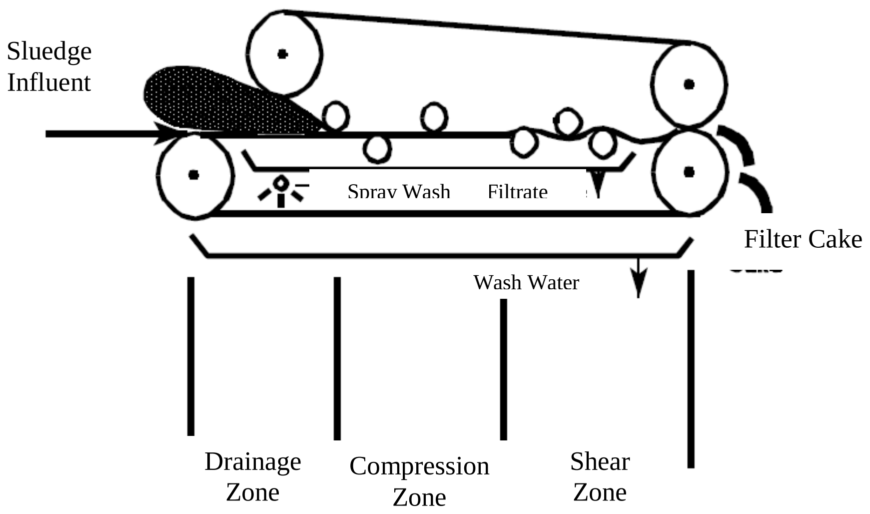

Figure 2.19: Belt Pressure Filtration System Diagram

2.1.2.7 Vacuum Filtration

A commonly-used process for dewatering sludge is rotary vacuum filtration. These filters come in drum, coil, and belt configurations. The filter medium may be made of cloth, coil springs, or wire-mesh fabric. A typical application is a rotary vacuum belt filter; a diagram of this equipment is shown in Figure 2.20.

In operation, chemically treated solids are pumped to a vat or tank in which a rotating drum is submerged. As the drum rotates, a vacuum is applied to the drum. Solids collect on the media and are held there by the vacuum as the drum rotates out of the tank. The vacuum removes additional water from the captured solids. When solids reach the discharge zone, the vacuum is released and the dewatered solids are discharged onto a conveyor belt for disposal. The media are then washed prior to returning to the start of the cycle (Spellman, 2003).

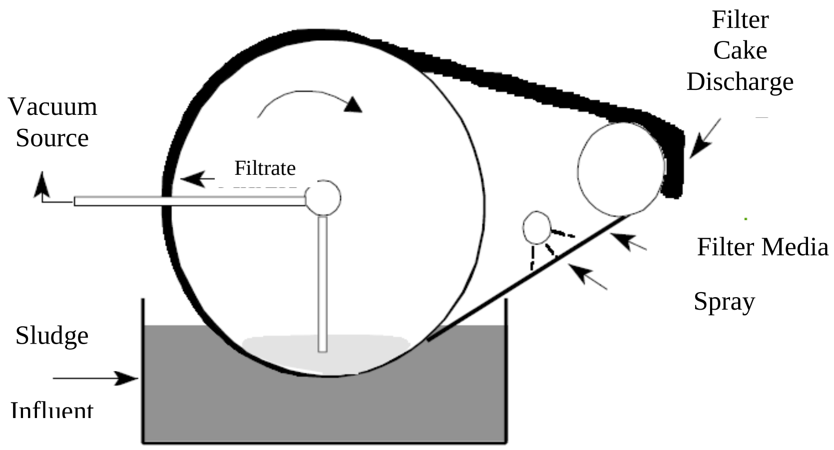

Figure 2.20: Vacuum Filtration System Diagram

2.1.2.8 Filter Cake Disposal

After sludge is dewatered, the resultant filter cake must be disposed. The most common method of filter cake management used in the CWT industry is transport to an off–site landfill for disposal. Other disposal options are incineration or land application. Land application is usually restricted to biological treatment residuals.

2.2 On-Site Treatment Systems

Decentralized Wastewater Treatment (DWWT) System is an onsite or cluster wastewater system that is used to treat and dispose of relatively small volumes of wastewater, generally from individual or groups of dwellings and businesses (Figure 2.1).

CWT has high technology hence they are very expensive (both in investment and operation). High amount of fund is required to install the sewerage systems required for CWT, and the maintenance of these systems is also expensive. Geographical location and size is another big problem along economical problems for small communities in rural areas.

However these small communities are also wanted to achieve the same standard degree of treatment as the large communities which use CWT.

A number of new technologies have been introduced for small treatment systems that have made it possible to produce an effluent of the same quality, or even better, as compared to large treatment plants.

The aim of centralized wastewater treatment system is to have a system which makes treatment for a whole area while the aim of decentralized wastewater treatment (DWWT) system is to have treatment systems as close as possible to the wastewater source.

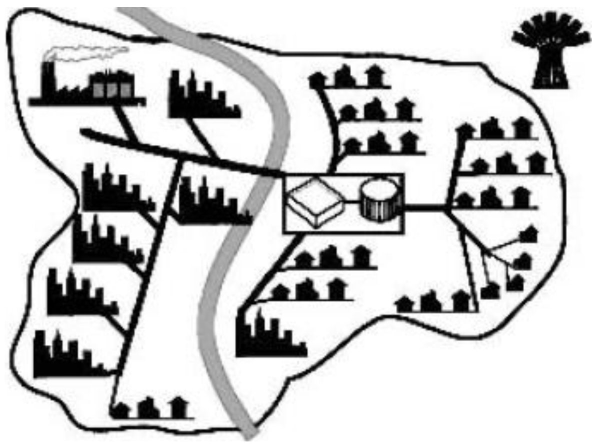

Figure 2.21: A centralized wastewater treatment system (Wanasen, 2003)

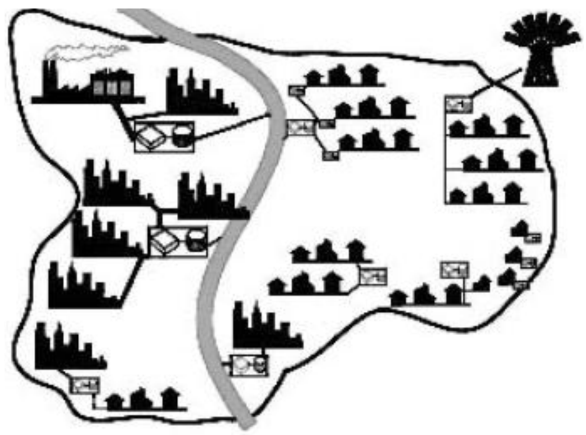

Figure 2.22: A DWWT system (Wanasen, 2003)

Decentralized wastewater treatment systems require limited funds for operation and maintenance. Hence the technologies that have been introduced for DWWT system require low operation and maintenance, and as little energy as possible.

There are many DWWT system options, such as intermittent sand filters, disposal fields, Imhoff tank, grey water systems and many others. The principal wastewater management options available for unsewered areas are shown in the table below.

Table 2.1: Wastewater management opt. for unsewered areas (Metcalf Eddy, 1991)

SOURCE OF WASTEWATER	WASTEWATER TREATMENT AND/OR CONTAINMENT	WASTEWATER DISPOSAL
Residential areas: Combined wastewater Black water Grey water Public facilities Commercial establishments Industries	Primary treatment: Septic tank Imhoff tank Secondary treatment: Aerobic/anaerobic units Aerobic units are 1) Intermittent sand filter 2) Re-circulating granular medium filter 3) Constructed wetlands Onsite containment: Holding tank Privy	Subsurface disposal: Disposal fields Seepage beds Disposal trenches Mound systems Evapotranspiration/percolation Others: beds/ponds Drip application Wetland (marsh) Discharge to water bodies

Most suitable decentralized wastewater treatment method is using very simple and cost effective natural wastewater treatment systems. There are different kinds of natural wastewater treatment methods which is economical and decent effective. Also they require long times and wide surface areas to become effective.

2.2.1 Natural wastewater treatment systems

Natural wastewater treatment systems are simple, cost-effective and efficient methods to purify the growing amount of wastewater produced by society. They can be applied as secondary or tertiary purification treatment, allowing the removal of most of the bacteria, microorganism and the destruction of the organic matter. There are plenty of different methods. Among them wetland, lagoon purification and storage in tanks gave good results in terms of yield and are quite diffused all over the world.

Their extreme simplicity in building, operation and maintenance make these systems competitive with the conventional (sewer) wastewater treatment methods.

Generally these systems are used all over the world for the purification of wastewater from industry, household and agriculture.

Main features

The main features of natural wastewater treatments are:

Simplicity: Design and construction of the plants are very simple. Even small building companies can build them and unqualified staff can carry out their maintenance operations.

Cost-effectiveness: Cost of building and maintenance of the plants is low. Because they require almost no energetic consumption or waste treatment, they are much more convenient than the conventional (biological) wastewater plants during the operational phase. Also because mechanical devices are not used in these treatments, thus maintenance costs are reduced. There is no limiting factor except availability and the cost of land to place the treatment plants.

Efficiency: The efficiency is highly dependent on climatic conditions (it is lower with low temperatures). Thus natural wastewater treatment plants are generally rather efficient for the removal of the pollutants.

Reliability: Natural systems are very reliable even in extreme operating conditions. They can adsorb a wide variety of hydraulic and organic feed.

Table 2.2: Advantages and disadvantages of decentralised wastewater treatment systems (Naturgerechte, 2001)

Type	Kind of treatment	Kind of wastewater treated	Advantages	Disadvantages
Septic tank	sedimentation, sludge stabilisation	wastewater with main pollution by settleable solids, esp. domestic	simple, durable, little space because of underground construction	tow treatment efficiency, effluent not odourless
Imhoff tank	sedimentation, sludge stabilisation	wastewater with main pollution by settleable solids, esp. domestic	durable, little space because of underground construction, odourless effluent	less simple than septic tank, needs very regular desludging
Anaerobic filter	anaerobic degradation of suspended and dissolved solids	pre-settled domestic and industrial wastewater of narrow COD/BOO ratio	simple and fairly durable if well constructed and wastewater has been property pre-treated, high treatment efficiency, little permanent space required because of underground construction	costly in construction because of special filter material, blockage of filter possible, effluent smells slightly despite high treatment efficiency
Baffled septic tank	anaerobic degradation of suspended and dissolved solids	pre-settled domestic and industrial wastewater of narrow COD/BOD ratio, suitable for strong industrial wastewater	simple and durable, high treatment efficiency, little permanent space required because of underground construction, hardly any blockage, relatively cheap compared to anaerobic filter	requires larger space for construction, less efficient with weak wastewater, longer start-up phase than anaerobic filter
Constructed wetlands (Horizontal gravel filter)	aerobic-facultative-anaerobic degradation or dissolved and fines suspended solids, pathogen removal	domestic and weakly polluted industrial wastewater after removal of settleable and most suspended solids by pre-treatment	high treatment efficiency if properly constructed, pleasant landscaping possible, no wastewater above ground, cheap in construction if filter material is locally available, no odour nuisance	high permanent space requirement, costly if right quality of gravel is not available, great knowledge and care required during construction, intensive maintenance and supervision durinq first years
Anaerobic pond	sedimentation. anaerobic degradation and sludge stabilisation	heavily and medium polluted industrial wastewater	simple in construction, flexible with respect to degree of treatment, low maintenance requirements	wastewater pond occupies open land, there is always some odour, at times strong, mosquitoes are difficult to control
Aerobic pond	aerobic degradation, pathogen removal	weakly polluted, mostly pre-treated wastewater from domestic and industrial sources	simple in construction, reliable in performance if properly dimensioned, high pathogen removal rate, can be integrated well into natural environment, fish farming possible if large in size and loading is low	large permanent space requirement, mosquitoes and odour can become a nuisance if undersized; algae can raise effluent BOD

2.2.2 Ponds and Lagoons

Wastewater treatment can be accomplished using ponds. Ponds are relatively easy to build and manage, can accommodate large fluctuations in flow, and can also provide treatment that approaches conventional systems (producing a highly purified effluent) at much lower cost.

Table 2.3: General Specifications For Ponds (Carl E. 1999)

POND TYPES	FLOW REGIME	S.FACE AREA m^2	DEPTH m	LRT (days)	BOD_5 (mg/m^2-day)	BOD_5 (%) REMOVAL	ALGAL CONC. (mg/l)
Aerobic (low-rate)	İntermittent mixing	40.468	0,91-1,21	10-40	6.725-13.450	80-95	40-100
Aerobic (high-rate)	İntermittent mixing	2.023-8.093	0,30-0,45	4-6	8.966-17.933	80-95	100-260
Aerobic (maturation)	İntermittent mixing	8.093-40.468	0,91-1,52	5-20	1.681	60-80	5-10
Facultative	Mixed (surface layer)	8.093-40.468	1,21-2,43	5-30	5.604-20.175	80-95	5-20
Anaerobic	No mixing	2.023-8.093	2,43-4,87	20-50	22.417-56.042	50-85	0-5
Aerated lagoon	Completely mixed	8.093-40.468	1,82-6,09	3-10		80-95	

2.2.2.1 Stabilization Ponds

Stabilization ponds are large shallow basins used for wastewater treatment by natural processes involving the use of algae and bacteria to accomplish biological oxidation of organic matter. Mixing is usually provided by natural processes such as wind, heat, or fermentation; however, mixing can be induced by mechanical or diffused aeration.

When wastewater enters the stabilization pond several processes begin to occur. These include settling, aerobic decomposition, anaerobic decomposition, and photosynthesis. Solids in the wastewater will settle to the bottom of the pond. In addition to the solids in the wastewater entering the pond, solids, which are produced by the biological activity, will also settle to the bottom. Eventually this will reduce the detention time and the performance of the pond. When this occurs (usually 20 to 30 years) the pond will have to be replaced or cleaned.

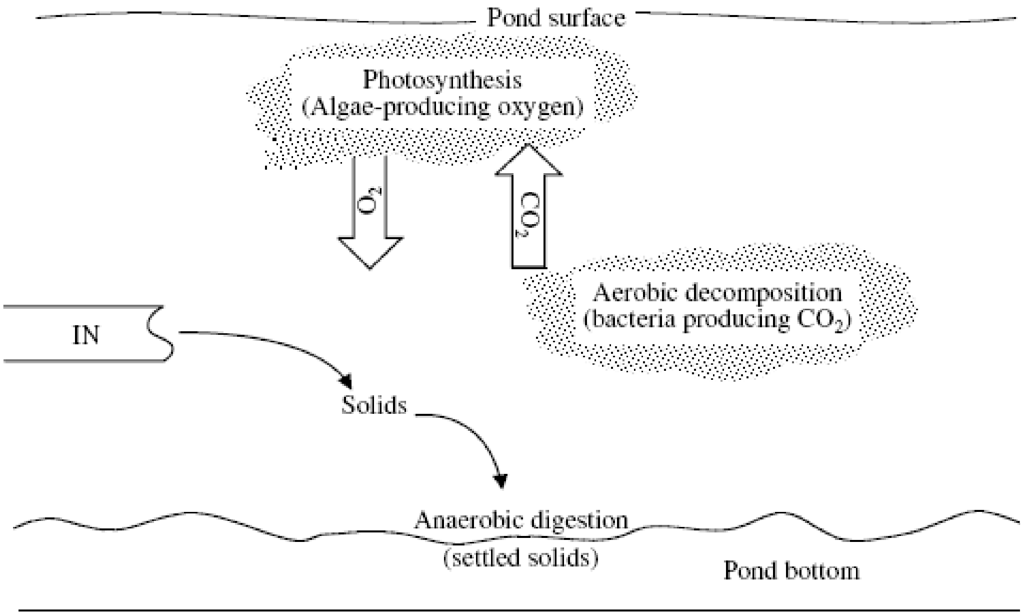

Figure 2.23: Stabilization pond processes (Spellman, 2003)

In states where stabilization-pond-treatment processes are commonly used, regulations govern pond design, installation, and operation. A minimum retention time of 60 days is often required for flow-through facultative ponds receiving untreated wastewater (Metcalf and Eddy, 1991).

Frequently, retention times as high as 120 days are specified. However, even with a low retention time of 30 days, a high degree of coliform removal is ensured. Other typical standards include embankment slopes (1:3 to 1:4), organic loading rate (2.2 to 5.5 g BOD/m^2-day, depending on climate), and permissible seepage through the bottom (0 to 6 mm/day). In some climates, treatment facilities can operate ponds without discharge to surface waters (McGhee, 1991).

When compared with other wastewater treatment systems involving biological treatment, a stabilization pond treatment system is the simplest to operate and maintain. Operation and maintenance activities include collecting and testing samples for DO and pH, removing weeds and other debris (scum) from the pond, mowing the berms, repairing erosion, and removing burrowing animals.

2.2.2.2 Organic Loading

The amount of BOD per unit area (or volume) per unit of time; usually expressed as [kg/m^2/day] or [kg/m^3/day].

2.2.2.3 Aerobic Ponds

In aerobic ponds, which are not widely used, oxygen is present throughout the pond. All biological activity is aerobic decomposition which is done by bacteria and algae in suspension under aerobic conditions.

There are two basic types of aerobic ponds. In one type, the objective is to maximize algae production. These aerobic ponds generally operate at depths of 0.15 m to 0.45 m.

In the other type of aerobic ponds, the amount of oxygen produced is maximized, and depths range to 1.5 m. Shallower depths encourage rooted aquatic plant growth, interfering with the treatment process. However, greater depths can interfere with mixing and oxygen transport from the surface. To achieve the best results with aerobic ponds, wastewater treatment facilities should provide mixing with pumps or surface aerators.

Environmental engineers adjust the pond loading rate to reflect the oxygen available from photosynthesis and atmospheric reaeration. Frequently, environmental engineers design large aerobic pond systems as completely mixed reactors, with two or three reactors in series.

Another design approach involves the use of a first-order, removal-rate equation developed by Wehner and Wilhelm (Metcalf and Eddy, 1991). This equation describes the substrate removal for an arbitrary flow-through pattern that lies somewhere between completely mixed and plug-flow as follows:

$S/S_0 = 4ae^{-(1/2d)}/[(1 + a)^2 e^{-(a/2d)} – (1 – a)^2 e^{-(a/2d)}]$

where:

S = Eff. substrate concentration, mg/l	u = fluid velocity (m/h)
S_0 = Inf. substrate concentration, mg/l	L = characteristic length (m)
$a = (1 + 4ktd)^{1/2}$	k = first-order reaction constant (h^{-1})
d = dispersion factor (D/uL)	t = retention time (h)
D = Axial dispersion coeff., (m^2/h)	

The term kt in the equation can be plotted as a function of S/S_0 for various dispersion factors (varying from zero for PF reactors to infinity for completely mixed reactors) to yield a graph that facilitates the use of the equation in designing ponds (Metcalf and Eddy, 1991).

The dispersion factor ranges from 0.1 to 2.0 for most stabilization ponds. For aerobic ponds, the dispersion factor is approximately 1.0 since completely mixed conditions usually prevail in these ponds for high performance. Depending on the operational and hydraulic characteristics of the pond, typical values for the overall first order BOD_5 removal-rate constant k range from 0.05 to 1.0 per day (Metcalf and Eddy, 1991).

Although aerobic pond efficiency is high (up to 95%) and most soluble BOD_5 is removed from influent wastewater, bacteria and algae in the effluent can exert a BOD_5 higher than that of the original waste. Hence, wastewater treatment facilities must apply methods of removing biomass from the effluent.

2.2.2.4 Facultative Ponds

The facultative pond is the most common type pond (based on processes occurring). Oxygen is present in the upper portions of the pond and aerobic processes are occurring. No oxygen is present in the lower levels of the pond where anoxic and anaerobic processes are occurring.

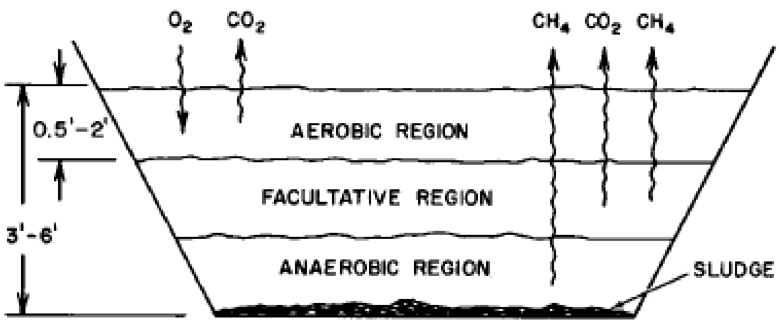

Figure 2.24: Elevation diagram of facultative lagoon strata and operation (Carl E, 1999)

The facultative pond contains three different zones, first zone is the surface zone where algae and bacteria grow up symbiotically, and second zone is an aerobic-anaerobic zone in the middle where facultative bacteria are responsible for waste conversion. The third zone is an anaerobic zone at the bottom sludge layer where anaerobic bacteria decompose accumulated organics in the lagoon.

Using the oxygen produced by algae growing near the surface, aerobic and facultative bacteria oxidize soluble and colloidal organics, producing carbon dioxide. This carbon dioxide is used by the algae as a carbon source.

Anaerobic waste conversion in the bottom zone produces dissolved organics and gases such as CH_4, CO_2, and H_2S that are either oxidized by aerobic bacteria or released to the atmosphere (Carl E, 1999).

Unlike aerobic ponds, facultative ponds promote settling of organics to the anaerobic zone. Therefore, quiescent conditions are required, and dispersion factors in facultative ponds vary from 0.3 to 1.0 (Metcalf and Eddy, 1991).

In cold climates, a portion of BOD_5 is stored in the accumulated sludge during the winter months. In the spring and summer as the temperature rises, accumulated BOD_5 is anaerobically converted. The end products of conversion (gases and acids) exert an oxygen demand on the wastewater. This demand can exceed the oxygen supply provided by algae and surface reaeration in the upper layer of the pond. In this case, wastewater treatment facilities should use surface aerators capable of satisfying 175 to 225% of the incoming BOD_5. The accumulation of sludge in the facultative pond can also lead to a higher SS concentration in the effluent, reducing overall pond performance (Carl E, 1999).

2.2.2.5 Anaerobic Ponds

Anaerobic ponds are normally used to treat high strength industrial wastes. There is no oxygen is in the pond and anaerobic decomposition is the only biological activity.

Anaerobic ponds treat high-strength wastewater with a high solids concentration which has enough organic loads to cause depletion of dissolved oxygen (O_2) and fixed oxygen (e.g. NO_3 or SO_4). This highly loaded and, consequently, anaerobic ponds that have particularly high odor emissions in the beginning until a heavy layer of scum has developed are often used as primary ponds in treating wastewater in tropical countries (Heinss-Strauss, 1998).

Anaerobic ponds are deep earthen ponds with depths to 9 m which maintain heat energy and anaerobic conditions. Influent waste settles to the bottom, and partially clarified effluent is discharged to another treatment process for further treatment. Anaerobic conditions are maintained throughout the depth of the pond except for the shallow surface zone (Carl E, 1999). Waste conversion is performed by a combination of precipitation and anaerobic metabolism of organic wastes to carbon dioxide, methane and other gases, acids, and cells. On the average, anaerobic ponds achieve BOD5 conversion efficiencies to 70%, and under optimum conditions, 85% efficiencies are possible (Metcalf and Eddy, Inc. 1991).

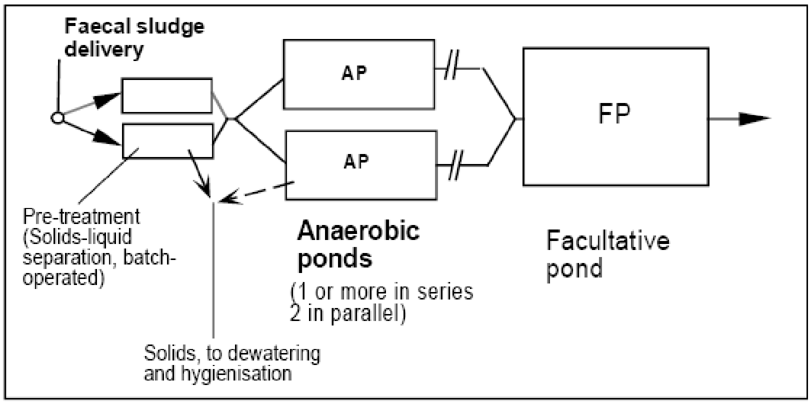

Figure 2.25: Schematic Drawing of a Waste Treatment System Treating Low to Medium-Strength Faecal Sludges (Montangero and Strauss, 2002)

Figure shows a Waste Treatment system suitable to treat low to medium-strength faecal sludges. It comprises pre-treatment units (tanks or ponds) for solids-liquid separation followed by a series of one or more anaerobic ponds and a facultative pond. This allows producing a liquid effluent apt for discharge into surface waters. Effluent use in agriculture is not possible due to its high salinity (Montangero and Strauss, 2002).

2.2.3 Constructed Wetlands

Wetlands obtain protection for water resources such as lakes, streams, and groundwater. Although naturally occurring wetlands have always served as ecological buffers, research and development of wetland treatment technology is a relatively recent phenomenon. Studies of the feasibility of using wetlands for wastewater treatment were initiated during the early 1950s in Germany. In the United States, wastewater to wetlands research began in the late 1960s and increased dramatically in scope during the 1970s (Thomas, 2001). As a result, the use of wetlands for water and wastewater treatment has gained considerable popularity worldwide.

In general, constructed wetlands are used for wastewater with a low suspended solids content and COD concentrations below 500 mg/l. It is an excellent technology for upgrading septic tank effluent to a very high quality.

There are three main types of constructed wetlands:

- Free Water Surface (FWS) Systems
- Subsurface Flow (SSF) Systems
- Vertical Flow Systems

Both types (FWS and SSF) consist of a channel or a basin with some sort of barrier to prevent seepage and utilize emergent aquatic vegetation as part of the treatment system. The difference between FWS and SSF wetlands is that SSF uses some kind of media as a major component (Suthersan,2001).

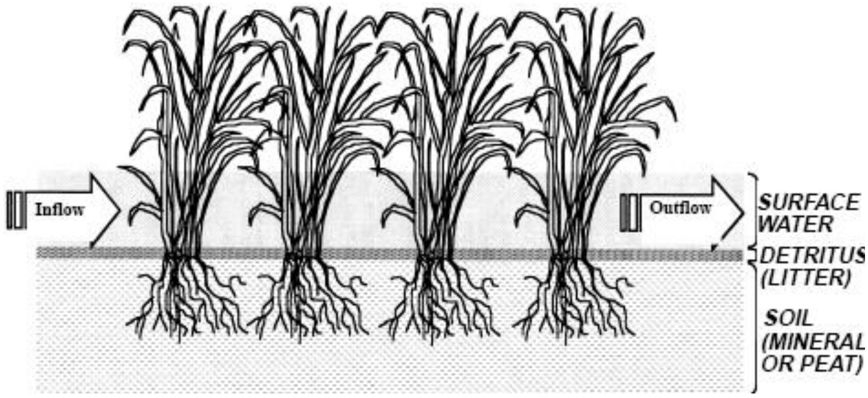

Figure 2.26: Free Water Surface (FWS) Systems (Thomas, 2001)

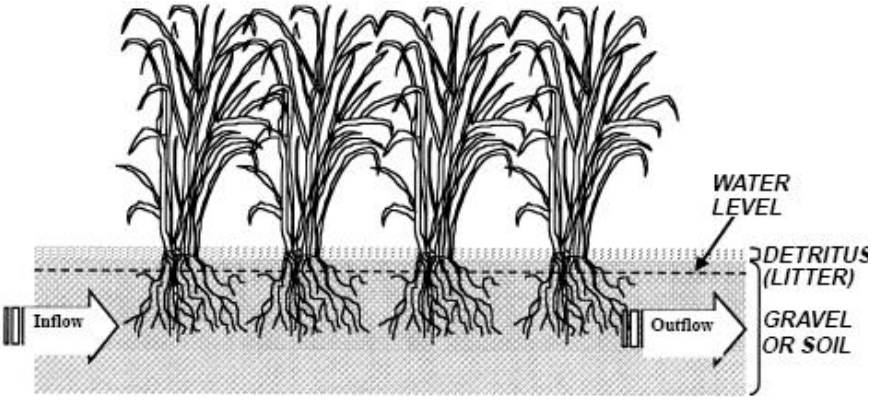

Figure 2.27: Subsurface Flow (SSF) Systems (Thomas, 2001)

2.2.3.1 Free Water Surface (FWS) Systems

FWS system design typically incorporates a shallow layer of surface water, flowing over mineral (sandy) or organic (peat) soils. These systems look like natural wetlands both in the way they look and the way the wastewater is treated. Hence natural wetlands, both forested and herbaceous, have also been effectively used as FWS system. They maintain a shallow depth 10 to 45 centimeters of water and wastewater. Wetland plants play an important role in filtering wastes and providing surface area for bacteria, which enhances treatment and regulates flow (McComas, 2000).

The size and configuration of the FWS system vary dramatically in size, from less than 1 ha to greater than 1000 ha, based on estimated wastewater volume, the strength of the wastewater to be treated daily, and estimates of how long the wastewater needs to remain in the wetland to be treated. Large FWS wetlands are even being used as a nutrient control technology to treat runoff from entire regional watersheds. Effectiveness depends on the wetland size (volume), location in the watershed, and configuration of inlet and outlet structures (Thomas, 2001).

2.2.3.2 Subsurface Flow (SSF) Systems

The SSF systems are the most common type of constructed wetland used to treat household waste on-site. Hence they require less land area than FWS systems and are usually designed to blend in with the landscape. With these systems, the wastewater is treated below ground, the surface of the flowing water is beneath the surface of the top layer of the medium so it is less likely to release odors or attract mosquitoes or pests and it continues to provide effective treatment of most wastewater components through the winter in temperate climates.

Subsurface flow wetlands differ from FWS wetlands in that they incorporate a suitable depth (1.5 – 3.0 feet) of a rock or gravel matrix that the wastewater is passed through in a horizontal or vertical fashion. In horizontal flow systems the top layer of the bed will remain dry unless the matrix clogs. The matrix media also support the root structure of the emergent vegetation.

Subsurface flow wetlands also can be operated in a vertical flow fashion which can reduce matrix clogging problems and enhance certain contaminant removal processes such as nitrification (Thomas, 2001).

2.2.3.3 Vertical Flow Systems

Vertical flow constructed wetlands are vegetated systems in which the flow ofwater is vertical rather than horizontal as in FWS and SSF wetlands.

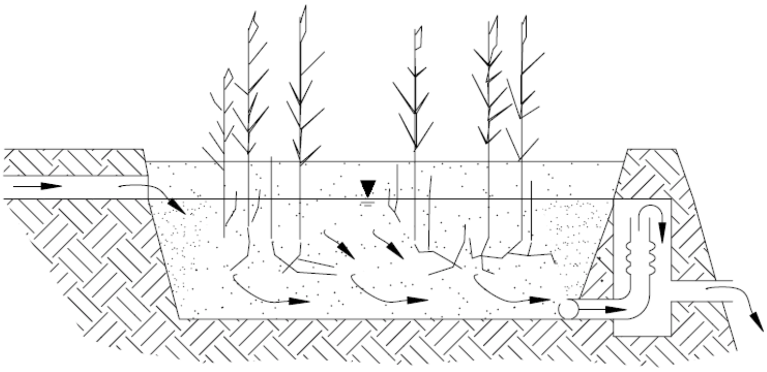

Figure 2.28: Vertical flow constructed wetland.

Polluted water is applied at time intervals over the entire surface of the wetland. The water flows through a permeable medium and is collected at the bottom. The intermittent application allows the cell to drain completely before the next application. This type of operation allows for much more oxygen transfer than typical SSF systems and thus may be a good option for treatment of wastewaters with a relatively high oxygen demand and high levels of ammonia through nitrification. But also greatly increases the mechanical and operational requirements of the system over the more traditional wetland treatment processes (Suthersan,2001).

2.2.4 Septic Tanks

Septic tanks receive raw sewage, allow it to settle, and pass the relatively clear liquid to the adsorption field, which is the next stage of treatment. The remaining solids digest slowly in the bottom of the tank.

Anaerobic decomposition, which takes place in the absence of free oxygen in a septic tank, is a slow process. To maintain practical detention times (6 to 8 h or more), the reactions cannot be carried far. Therefore, the effluent is often, contains a multitude of microorganisms and organic materials that require further decomposition (Carl E, 1999).

It’s usually constructed by concrete (often prefabricated), occasionally prefabricated steel.

(a) (b)

Figure 2.29: Typical Conventional Septic Tanks: (a) Concrete type with reinforcing steel (under construction) and (b) fibreglass type

2.2.4.1 Septic Tank Design

Usually a two-compartment design arranged in a series is preferred (Figure 2.30). The first chamber should contain two-thirds and the second chamber should contain one-third of the total volume. The liquid depth should be between 1.2 and 2 m.

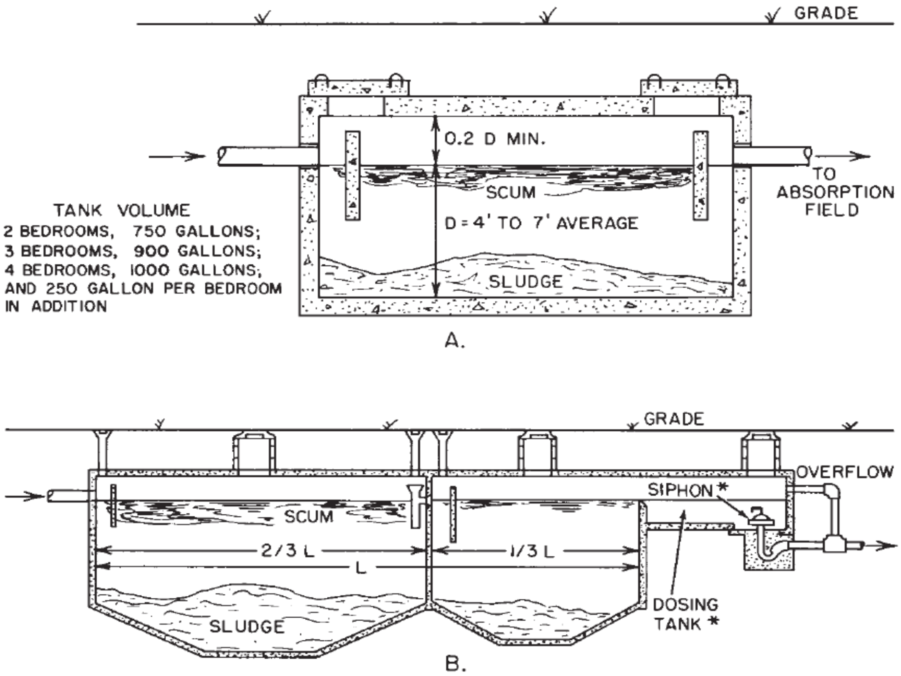

Figure 2.30: Septic tank configurations. A. Typical household septic tank; B. Typical large institutional septic tank with dosing siphon.

For large fields, uniform distribution is obtained by periodic flooding of the field followed by periodic drying. Dosing tanks are used to flood these fields; they collect the sewage, and automatic bell siphons or pumps transport the waste to the field.

Septic Tanks which removes 15 to 25% of BOD and 40 to 60% of SS additionally requires usually subsurface drainage fields, occasionally intermittent sand filters or lagoons.

2.2.5 Two-Story (Imhoff) Tanks

The two-story or Imhoff tank is similar to a septic tank in the removal of settleable solids and the anaerobic digestion of solids. The difference is that the two story tank consists of a settling compartment where sedimentation is accomplished, there is little or no decomposition and often remains aerobic, a lower compartment where settled solids and anaerobic digestion takes place, and gas vents. Solids removed from the wastewater by settling pass from the settling compartment into the digestion compartment through a slot in the bottom of the settling compartment. The design of the slot prevents solids from returning to the settling compartment. Solids decompose anaerobically in the digestion section. Gases produced as a result of the solids decomposition are released through the gas vents running along each side of the settling compartment.

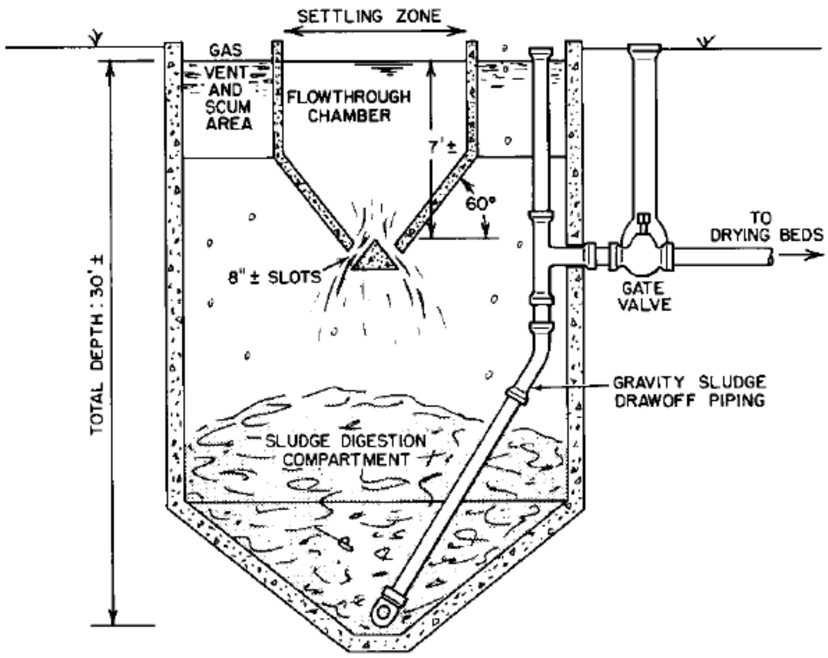

Figure 2.31: Imhoff tank configuration

The Imhoff tank has been developed for pre-treatment of wastewater in small treatment systems. Larger storage volumes for sludge and shorter sludge removal intervals have to be considered when adapting the design to FS treatment.

The volume foreseen for settling (above the inclined walls) can be kept similar to the design for wastewater. Recommended design values: overall depth 2-3m; sludge accumulation depth 0.5–1 m; hydraulic retention time in settling compartment 4-8 h; sludge removal intervals 1-4 weeks, volume of accumulated sludge per incoming solids load 5-9 L/kg TS (Spellman, 2003).

The operation of Imhoff tanks is not complex. They are less efficient than settling basins and heated-sludge digestion tanks. The newer treatment methods offer more efficient alternatives to Imhoff tanks, but in small treatment units, they do provide efficient solids separation without mechanical or electrical equipment.

Imhoff Tanks which removes 25 to 35% of BOD and 40 to 60% of SS additionally requires usually trickling filters, occasionally intermittent sand filters or lagoons (Carl E, 1999).

2.2.6 Baffled Septic Tank

The baffled septic tanks, also known as "baffled reactor", are suitable for all kinds of wastewater, preferably for those with a high percentage of non-settleable suspended solids and low COD/BOD ratio.

The baffled reactor is a combination of several anaerobic process principles; the septic tank, the fluidised bed reactor and the UASB. Its upflow velocity which should not exceed 2 m/h, limits its design. Based on a given hydraulic retention time, the upflow velocity increases in direct relation with the reactor height. Reactor height cannot serve as a variable parameter to design the reactor for the required hydraulic retention time (HRT) so that the limited upflow velocity results in large but shallow tanks.

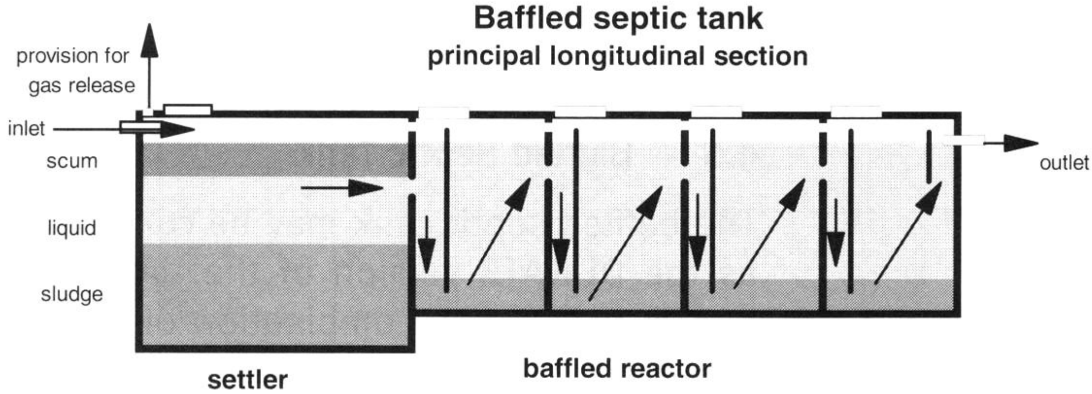

Figure 2.32: Schematic diagram of baffled reactor

The baffled septic tank is ideal for decentralised wastewater treatment because it is simple to build and simple to operate. Hydraulic and organic shock loads have little effect on treatment efficiency. The baffled septic tank consists of at least four chambers in series. The last chamber can have a filter in its upper part in order to retain eventual solid particles. The first compartment is always a settling chamber for larger solids and impurities, followed by a series of upflow chambers. The water stream between chambers is directed by baffle walls that form a down-shaft or by down-pipes that are placed on partition walls. A settler can also follow the baffled septic tank as posttreatment.

2.2.7 Anaerobic Filter

In an anaerobic filter reactor, the growth-supporting media is submerged in the wastewater. Anaerobic microorganisms grow on the media surface as well as inside the void spaces among the media particles. The media entraps the Suspended Solid present in the influent wastewater that can be fed into the reactor from the bottom (upflow filter) or the top (downflow filter).

Periodically backwashing the filter solves bed-clogging and high-head-loss problems caused by the accumulation of biological and inert solids.

Because it can retain a high concentration of active biomass within the system for an extended time period, the anaerobic filter can easily adapt to varied operating conditions (e.g., without significant changes in effluent quality and gas production due to fluctuations in parameters such as pH, temperature, loading rate, and influent composition). Also, intermittent shutdowns and complications in industrial treatment will not damage the filter since it can be fully recovered when it is restarted at a full load (Carl E, 1999).

The following expressions describe the overall substrate utilization rate for a completely mixed anaerobic filter:

$$R_o = 5\,(kSX_s)/(K_s + S) + (\eta k'S)/(K_s + S)$$

$$k' = \rho k A \delta$$

where:

Ro = the overall substrate utilization rate, mass/volume-time

Xs = suspended biomass concentration, mass/volume

η = the effectiveness factor that defines the degree of diffusional limitations of the biofilm

k = the maximum substrate utilization rate in the biofilm, mass/volume-time

ρ = the biofilm dry density, mass/volume

A = total biofilm surface area per unit filter volume, l/length

δ = biofilm thickness, length

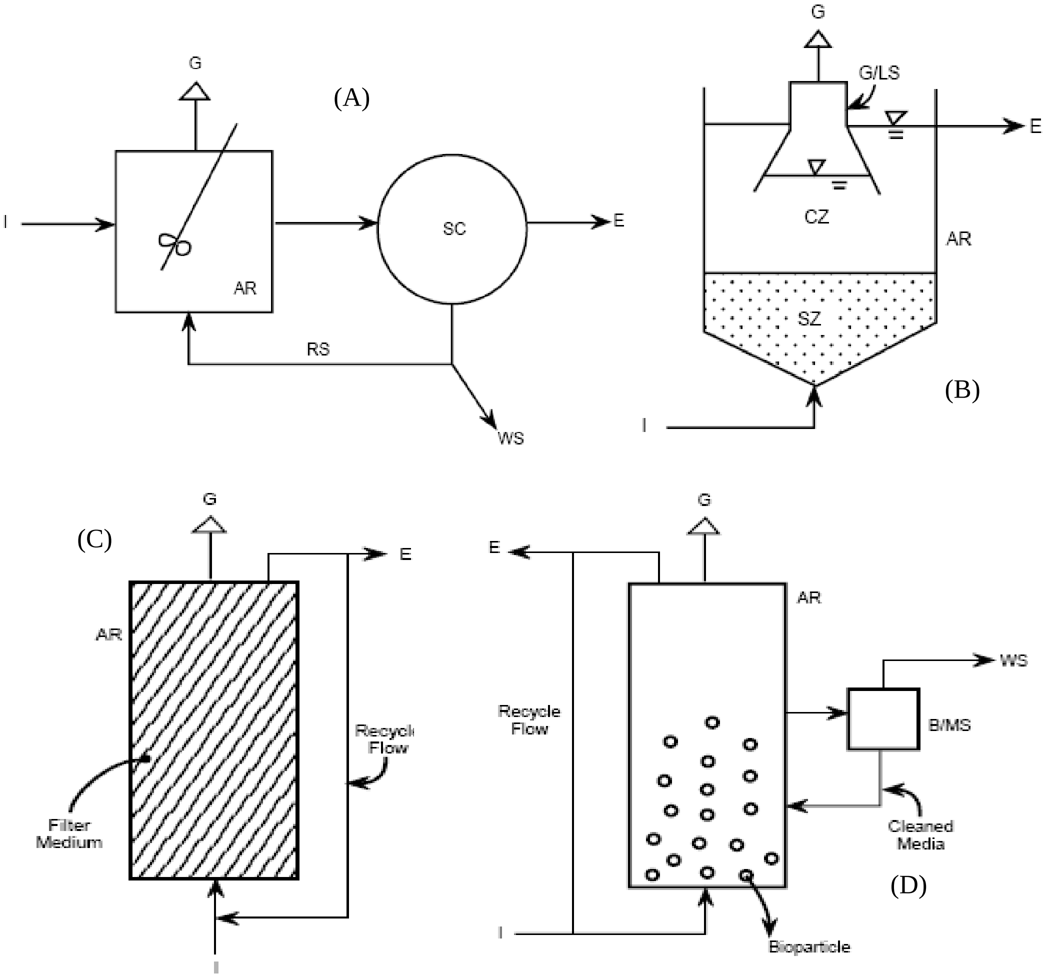

Figure 2.33: Process schemes of anaerobic treatment processes. A, Anaerobic contact process; B, Upflow sludge blanket reactor; C, Anaerobic filter; D**,** Anaerobic fluidized bed reactor.

Key:

AR: Anaerobic reactor

B/MS: Biofilm/media separator

CZ: Clarification zone

E: Effluent

G: biogas

G/LS: Gas–liquid separator

I: Influent

RS: Return sludge

SC: Secondary clarifier

SZ: Sludge zone

WS: Waste sludge

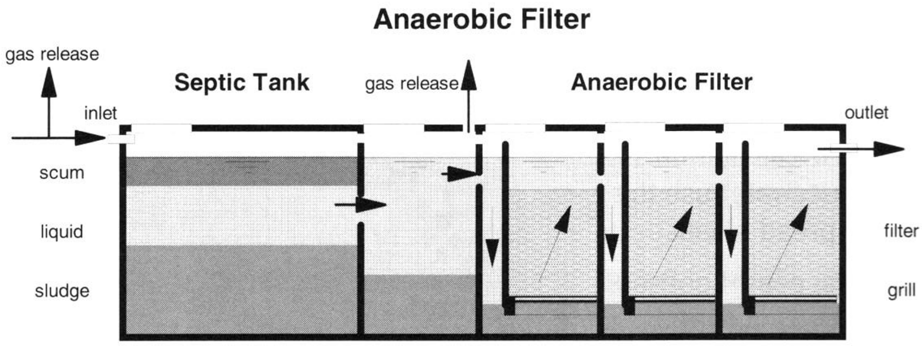

Figure 2.34: Schematic diagram of anaerobic reactor

2.3 Sakhnin System

Sakhnin is an Arab town located in the Galilee region of Israel, with a population of approximately 21,000. The Sakhnin Municipality operates a training center on wastewater treatment, and agricultural and energy conservation technologies in association with the Towns' Association for Environmental Quality (TAEQ). The training center is located at the Wastewater Treatment Pilot Site, and includes an educational lab and a training team connected with local schools. The Municipality and TAEQ work closely with the Consortium's partners on pilot facilitation. They also provide office space and administrative resources to the project.

The Wastewater Treatment Pilot Site began operations in July 1999. Its main objective is to test effective and appropriate wastewater treatment technologies for rural areas. Land availability, population size, climatic conditions and socioeconomic considerations were all taken into account in the selection of the treatment technologies to be studied. These factors all pointed to the need for wastewater treatment in Sakhnin that was extensive, reliable, simple, low-cost and low-impact. However, researchers found that the usual combination of extensive treatment units (anaerobic ponds, facultative ponds, and a reservoir) results in high water losses, due to the high, evaporation rates in the Mediterranean region.

The Sakhnin pilot site was designed with the goal of alleviating these problems and of implementing a replicable, comprehensive model of appropriate technology for wastewater treatment and reuse for sustainable agriculture in rural areas of the Middle East.

Established in 1997 the Appropriate Technology Consortium (ATC) is a cooperative effort of Israeli, Palestinian, and Egyptian non-governmental organizations (NGOs), research scientists, consultants, and municipalities to establish low-cost, efficient, and replicable wastewater treatment and reuse systems in rural areas of the Middle East.

Prior to the development of the ATC site at Sakhnin, the existing basic full-scale facility consisted of the following units:

- Two anaerobic ponds (sedimentation ponds which are based on biological activity without oxygen consumption), each with a volume of 5,000 cubic meters;
- One facultative pond (a pond where biological activity is, combined with anaerobic and aerobic bacteria) with a volume of 5,000 cubic meters;
- One reservoir with a volume of 150,000 cubic meters.

Treated wastewater, or effluent, from the reservoir was also already being used to irrigate local olive trees.

ATC incorporated the raw sewage and effluent from different stages of the already-existing facilities into a variety of treatment schemes in order to evaluate their performance.

2.3.1 The ATC Pilot Units

The following extensive and semi-extensive treatment units have been installed and are currently being studied by ATC at the pilot.

- Vertical Aerobic Beds
- An Up-flow Anaerobic Sludge Blanket (UASB) Reactor
- A Horizontal Subsurface Flow, Constructed Wetland
- An Intermittent Sand Filter
- A Wastewater Reservoir

Most of these treatment units are well-established technologies. However, evaluation of their performance under rural Middle East conditions is essential. The selected treatment system will be an integrated system consisting of several units. The pilot site was designed to be flexible, thus allowing for the study, of different combinations of the units.

2.3.2 Treatment Stages

As depicted below in Figure, wastewater treatment at the Sakhnin pilot site occurs in three stages. The units under study at each of these stages are described in detail below.

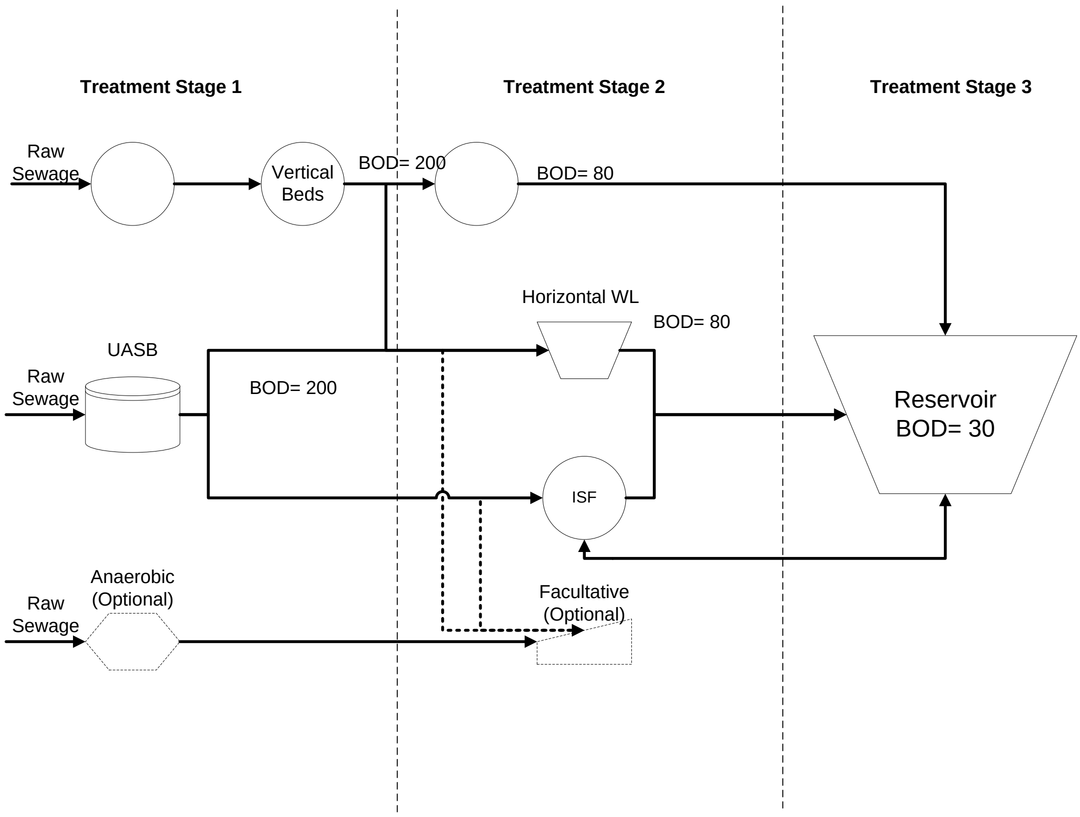

Figure 2.35: Treatment stages in Sakhnin

2.3.2.1 Treatment Stage 1

Raw sewage is received from Sakhnin's sewage collection system. It enters the pre-treatment anaerobic pond, or sedimentation pond, where grit and settleable organic solids are removed. After approximately two to three hours, the sewage is pumped into one of two semi-intensive units, either the Vertical " Aerobic Bed or the Up Flow Anaerobic Sludge Blanket (UASB), where it receives initial treatment aimed at reducing the biological oxygen demand (BOD) to approximately 200 mg/l.

Vertical Aerobic Beds (Unsaturated Flow Biological Filter with Passive Air Pump System)

Three Vertical Aerobic Beds exist at the site. In this system, a “passive air” pump, driven by a fill and draw hydraulic operation cycle, provides for aerobic conditions. Wastewater trickles through the "bed," which is composed of two layers of gravel:

- An upper layer of small sized gravel which serves as the support media for the microorganisms that degrade the organic matter in the wastewater;
- A coarse gravel layer in the bottom of the bed to allow for efficient drainage.

Figure 2.36: Vertical Aerobic Beds

Up Flow Anaerobic Sludge Blanket Reactor (UASB)

The Up Flow Anaerobic Sludge Blanket Reactor (UASB) is an extensive wastewater treatment unit that was developed in the early 1970s. The unit at the Sakhnin site is a modified simpler version (RALF) with conic geometry and without the typical eomptex separation device of most UASBs. Its success lies in the establishment of a dense sludge bed in the bottom of the reactor, in which the biological process takes place. This sludge bed is formed through the accumulation of solids and bacterial growth.

Sewage is pumped from the anaerobic pond after an initial few-hours of retention time. It enters from the bottom side of the reactor and is collected by an overflow channel surrounding the upper circumference of the reactor. A-baffle inside the top of the reactor prevents scum release from the reactor as the effluent moves to Stage 2.

Figure 2.37: Up Flow Anaerobic Sludge Blanket Reactor (UASB)

2.3.2.2 Treatment Stage 2

Following completion of initial treatment in Stage 1, the sewage flows to Stage 2 for further treatment. Currently, effluent enters either the Constructed Wetland or the Intermittent Sand Filter unit for Stage 2 processing.

Constructed Wetland

A natural wetland is an ecosystem where the water surface is near the ground surface long enough each year to maintain saturated, soil conditions and related vegetation. A "constructed wetland" is a wetland specifically built for the purpose of pollution control and waste management, at a location other than that of existing natural wetlands. (USEPA)

Figure 2.38: Constructed Wetland

Figure 2.39: Constructed Wetland (different angle)

Figure 2.40: Series of other wetlands

Figure 2.41: Closer view

At the Sakhnin site, treatment of effluent takes place in the Constructed Wetland, following treatment in either the UASB or the Vertical Beds. The Wetland consists of a gravel bed through which the wastewater flows horizontally. The gravel serves as the support media for microorganisms as well as for the roots of the plants growing there. The removal of contaminants from the wastewater is obtained through anaerobic biological degradation, adsorption, sedimentation and filtration. As long as the water level is kept below the surface of the media, there is little risk of problematic side effects such as odors or insects.

Intermittent Sand Filter (ISF)

Sand filters have been used for several decades for treating both freshwater and wastewater. Intermittent sand filtration doses the wastewater onto a sand bed "intermittently", and is used to treat effluent following other types of pretreatment processes, such as aerobic ponds. The ISF at the Sakhnin site has been designed to provide high-quality removal of contaminants, pathogen, reduction, and nitrification of wastewater effluent. Wastewater flows downwards through the four layers of different sizes of sand and gravel (ranging from 0.4 -20 mm). The ISF removes contaminants in the wastewater through physical filtration and biological processes. ISF units are known for their ability to produce a high quality of effluent that can be used, for irrigation.

Figure 2.42: Intermittent Sand Filter (ISF)

2.3.2.3 Final Treatment Stage

After completing Stage 2, all sewage enters the final stage of the treatment process. At this stage, it receives final cleaning to prepare it for reuse or for returning it to the surrounding environment. At the Sakhnin pilot site, the final stage consists of a reservoir.

Wastewater Reservoir

The reservoir is always the last treatment stage. Besides serving as an operational reservoir, it provides for final polishing of the effluent prior to its reuse. When designed and operated properly, stabilization reservoirs can remove 90% of the BOD and detergents, five orders of magnitude of fecal coliforms, and other pollutants including heavy metals, refractory organics, and general toxicity.

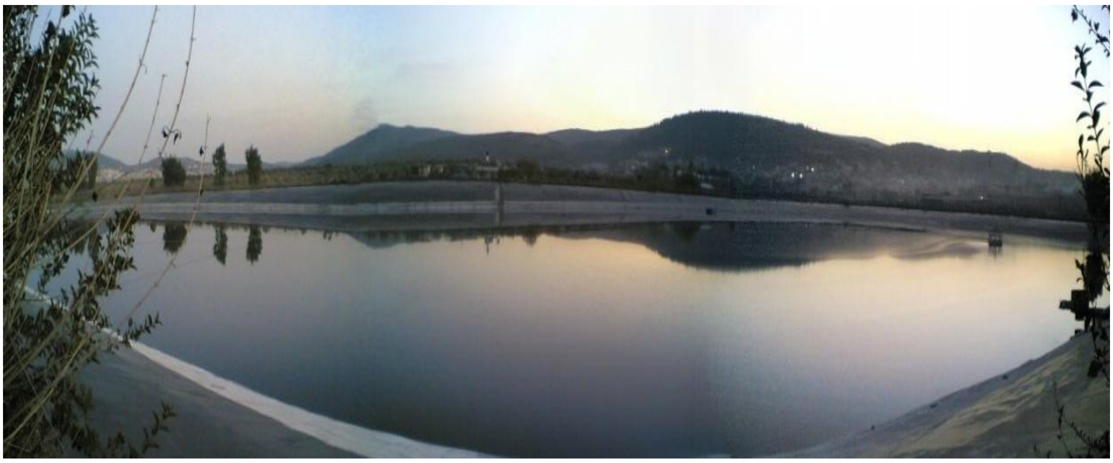

Figure 2.43: Wastewater Reservoir

3. EXPERIMENTAL SETUPS CONCERNING UPGRADING TECHNOLOGIES FOR TREATMENT METHODS CURRENTLY BEING USED

The scientific study was divided into 4 main technical tasks. Each "task" is devoted to the upgrading of one of the four "traditional" steps of WWTP of Sakhnin city. **Figure 3.1** shows top view scheme of the experimental area.

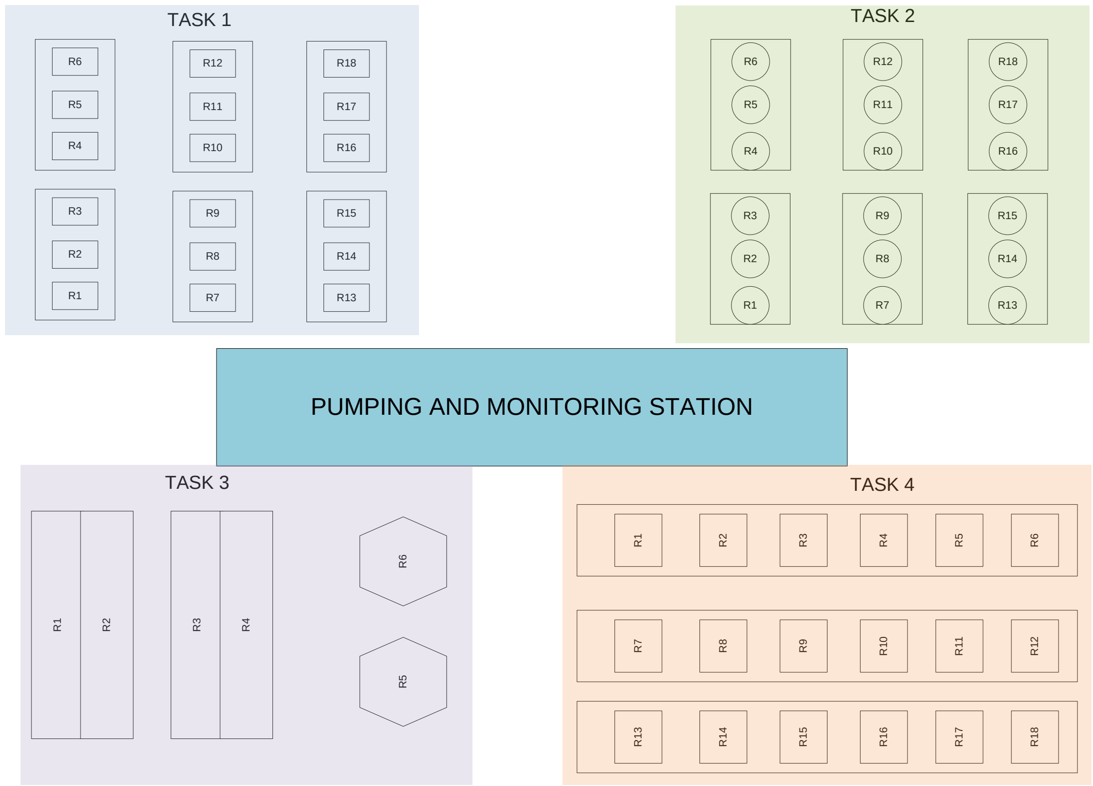

Figure 3.1: Scheme of the experimental area (Top View)

Each tasks influent comes from the treatment system which is belonged to. All inlet water was kept in 200lt tanks before fed into tasks (Figure 3.2). Only Task 1 had additional special sedimentation and pre-Filtration which was needed for effective treatment experiments (Figure 3.8).

Figure 3.2: Waste Water Storage Tanks for All Tasks

3.1 TASK 1 Anaerobic lagoon (Equipment Installation for Upgrading WWT Technologies by Adding Plastic Cover)

Equipment required was installed for the experimental research and development work related to upgrading the anaerobic settling pond by the addition of plastic covering. This was expected to improve the effluent quality produced by an experimental model of the Sakhnin WWT plant and reduce its environmental nuisances.

Figure 3.3: Anaerobic Lagoon

Figure 3.4: Second Anaerobic Lagoon

18 plastic tanks, 120 liters in volume each, were fed with effluent wastewater taken from the WWT plant at about the same hydraulic retention time of the full scale lagoons. Nine of the tanks were covered with a floating bell of polyethylene sheet while the other nine will served as a reference. Scheme and pictures of the tanks used are down below (Figure 3.5 to Figure 3.8).

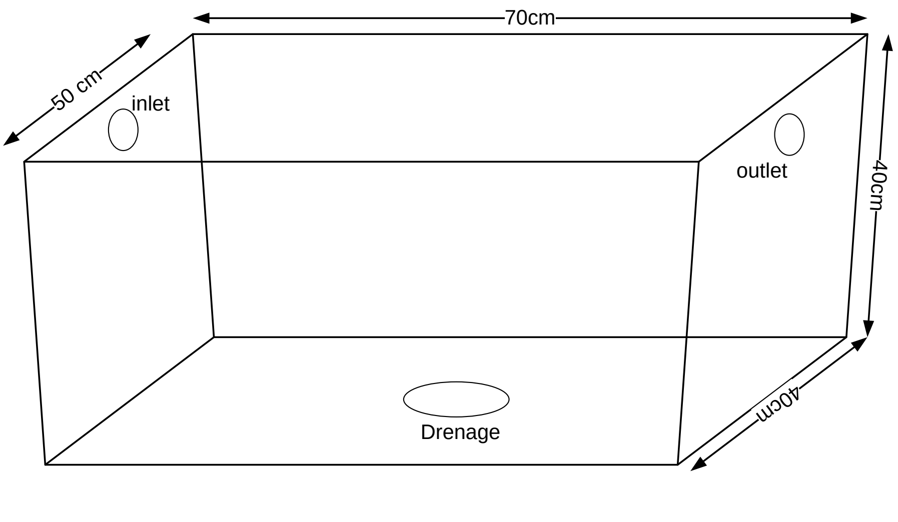

Figure 3.5: Scheme of the tank used for "task 1" and “task 4”

Figure 3.6: Experimental area of Task 1

Figure 3.7: Anaerobic tank covered with a covered floating bell of polyethylene sheet

Figure 3.8: Sedimentation and Pre-Filtration for Task 1

After covering the anaerobic ponds with the plastic covers, it was expected that improvements will be evident in all progress parameters in the experimental tanks, relative to the reference. Expected results for Task 1 are;

1. Improved reduction of pollutants (COD, BOD, etc.) in the experimental tanks relative to the reference tanks.
2. Reduction or complete elimination of odors near the covered tanks.
3. Heating of experimental tanks relative to the reference tanks.
4. Significant biogas production.
5. Temperature increase in the covered tanks, by solar heating.

It was assumed that the WWT plant of Sakhnin will continue to operate and supply raw effluents. Only constraint was the availability of raw wastewater from the city of Sakhnin.

Table 3.1: Equipment installation and Retention times for Anaerobic tanks

Reactor Number	Plastic cover	Retention Time (day)	Color of pl. cover
1	+	**10**	Black
2	+	**8**	White
3	+	**4**	White
4	+	**8**	White
5	+	**8**	Black
6	+	**6**	Black
7	+	**4**	Black
8	+	**2**	Black
9	+	**4**	White
10	**-**	**10**	-
11	**-**	**8**	-
12	**-**	**4**	-
13	+	**8**	Black
14	**-**	**8**	-
15	**-**	**6**	-
16	**-**	**4**	-
17	**-**	**2**	-
18	+	**4**	Black

3.2 TASK 2 Facultative Lagoon (Equipment Instillation for Upgrading of WWT Technologies by Applying an Intermittently Fed Bio-Filter (IF-BF)Technology)

Experimental equipment required was intalled for research and development on the possibility of reducing the organic load applied to the seasonal reservoir by replacing the existing facultative pond step by an intermittently fed bio-filter (IF-BF).

Figure 3.9: Facultative Lagoon

Figure 3.10: Facultative Lagoon

Nine pairs of intermittently fed ponds was constructed, filled with various types of biomass materials as the fixed medium. During the 3 years of the project, various types of biomass materials and various operation regime methods will be tested. In this study first 8 months of the project will be discussed.

Plastic cylindirical tanks of 120 liter volume was used to simulate the ponds (Figure 3.11 to Figure 3.16). Effluents from the facultative lagoon of the WWT plant were fed into the tanks at retention times of 2-10 days. The horticultural value of the biomass, after discharging the tanks will be tested. The quality of effluent was followed through conventional wastewater analytical procedures, and compared to the quality of the WWT plant facultative pond.

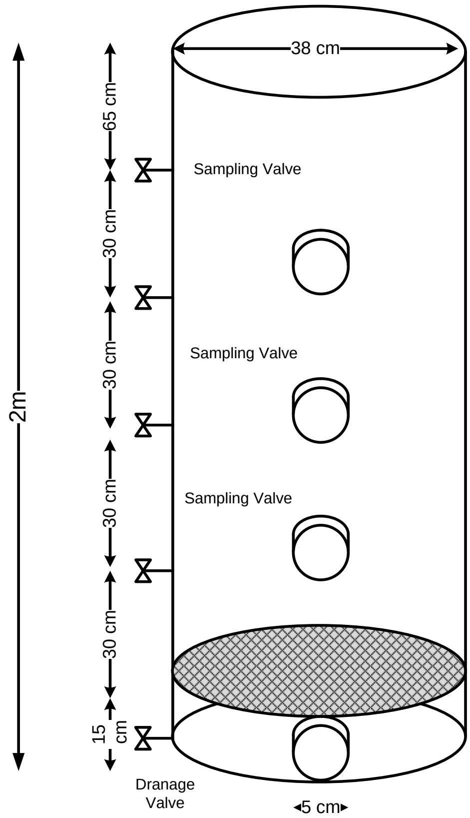

Figure 3.11: Scheme of the Intermittently Fed Bio-Filter

Figure 3.12: The Array of Task 2

Figure 3.13: Intermittently Fed Bio-Filter (1.5 m filter left hand side, 2.5 m filter right hand side)

Figure 3.14: Close View of the Bio-Filter

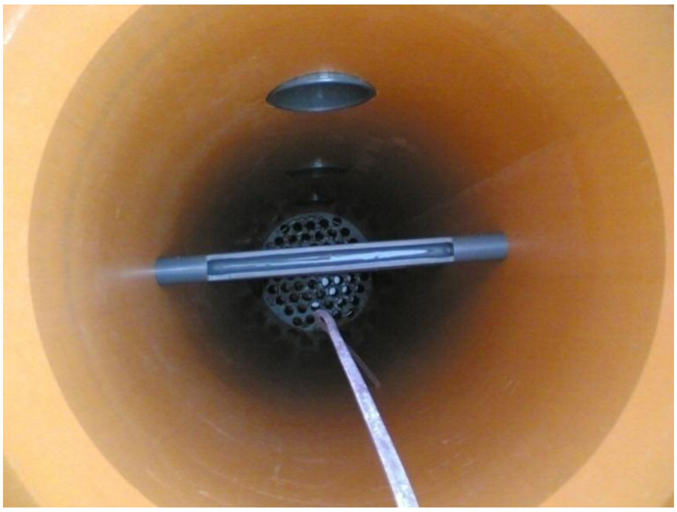

Figure 3.15: Inside of Tank

Figure 3.16: Filling woodchips

Expected results for Task 2 is;

1. Reduced pollutant concentration in the effluent of the IF-BF.
2. Obtaining stabilized biomass from the IF-BF for agricultural use.
3. Saving surface area for the need of other treatment systems.
4. Utilization of various vegetative wastes (biomass) from the area for the needs of the WWT plant and for compost-like material.

It was assumed that the WWT plant of Sakhnin will continue to operate and supply raw effluents. Only constraint was the availability of raw wastewater from the city of Sakhnin.

Table 3.2: Equipment installation and Retention times for Bio-filters

Reactor Number	Height Of Biofilter (m)	Type Of Wood Chips	Size Of Wood Chips (cm)	Retention Tıme (Days)
1	2.5	**evilo**	10	**6**
2	2.5	**evilo**	5	**6**
3	2.5	**enip**	10	**6**
4	1.5	**evilo**	10	**6**
5	1.5	**evilo**	5	**6**
6	1.5	**enip**	10	**6**
7	2.5	**enip**	5	**6**
8	2.5	**enip**	5	**6**
9	2.5	**enip**	5	**6**
10	1.5	**enip**	5	**6**
11	1.5	**enip**	5	**6**
12	1.5	**enip**	5	**6**
13	2.5	**enip**	10	**10**
14	2.5	**enip**	5	**10**
15	2.5	**enip**	5	**3**
16	1.5	**enip**	10	**10**
17	1.5	**enip**	5	**10**
18	1.5	**enip**	5	**3**

3.3 TASK 3 Seasonal Reservoir (Equipment Installation to Intensify WWT processes in the seasonal reservoir)

Experimental equipment required was installed for the research and development work on the enhancement of wastewater purification processes in the seasonal reservoir (SR) by controlling the hydraulic flow pattern.

Figure 3.17: Seasonal Reservoir

In the existing 150,000 m^3 reservoir of the Sakhnin WWT plant, partitions would be installed in a way that would direct the water flow in the pond in a plug flow pattern. The water quality along the constructed water channels (Figure 3.18 and Figure 3.19) formed by the partitions to simulate, under controlled experimental conditions, the flow, aeration, and biomass patterns in the SR will be monitored with reference to water quality situation in the non plug flow area of the reservoir.

In part of the channels, air diffusers were added to enhance biological activity. In some of the channels bundles of plastic strips were dipped to supply large surface area for the development of active biomass attached to the plastic strips.

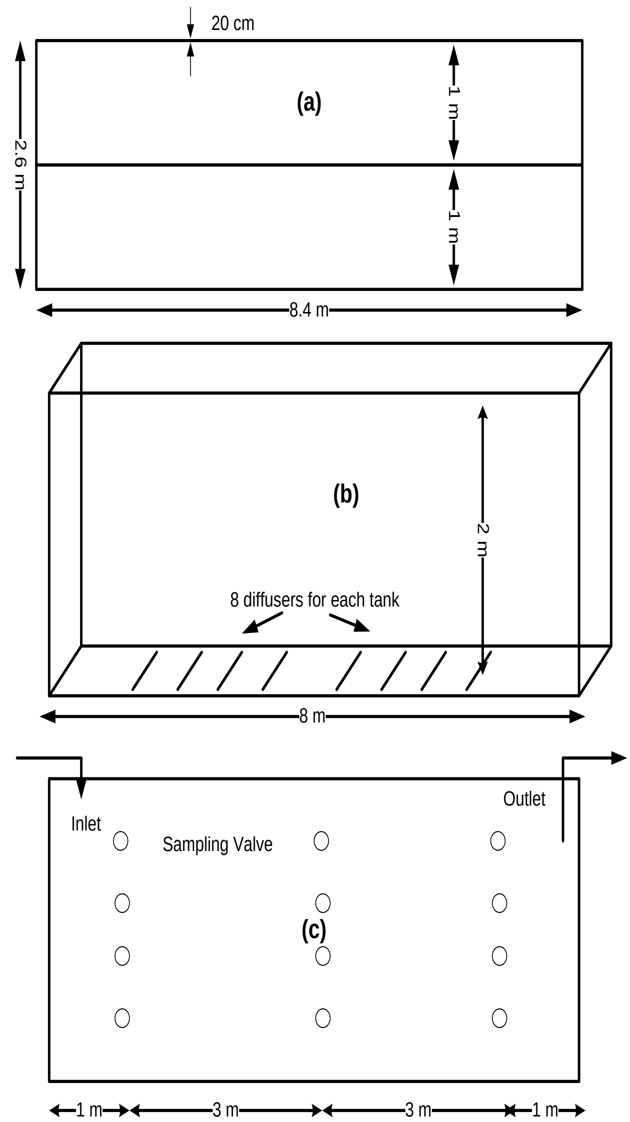

Figure 3.18: Scheme of the Rectangular Tanks (a) Top View, (b) Inside, (c) Side View

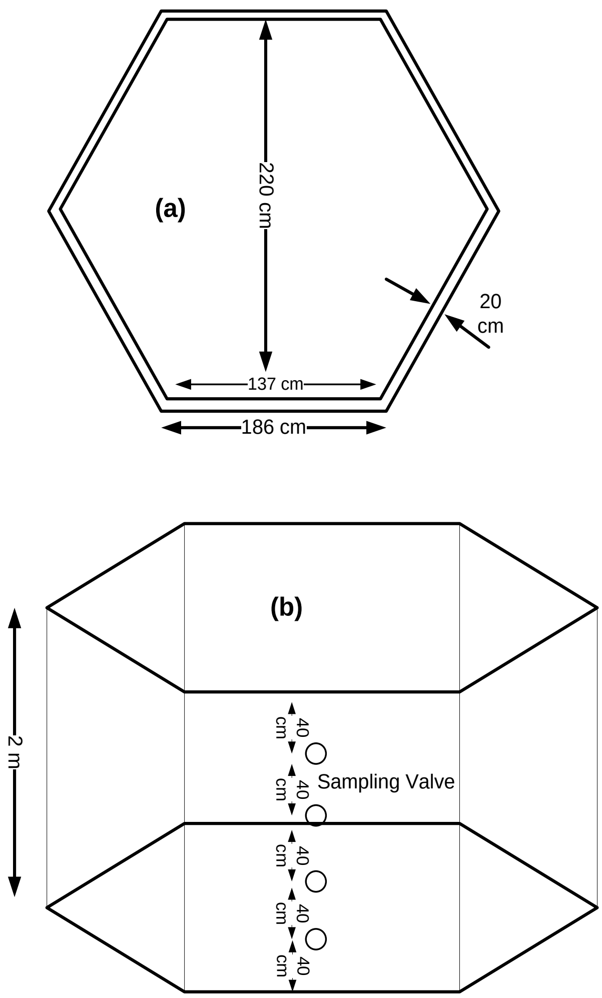

Figure 3.19: Scheme of the Hexagonal Tank (a) Top View, (b) Side View

Figure 3.20: 6 Reactors of TASK 3

Figure 3.21: TASK 3 Reactors (Different Angle)

Figure 3.22: Concrete Tunnels- Simulation of a Long Axis Seasonal Reservoir

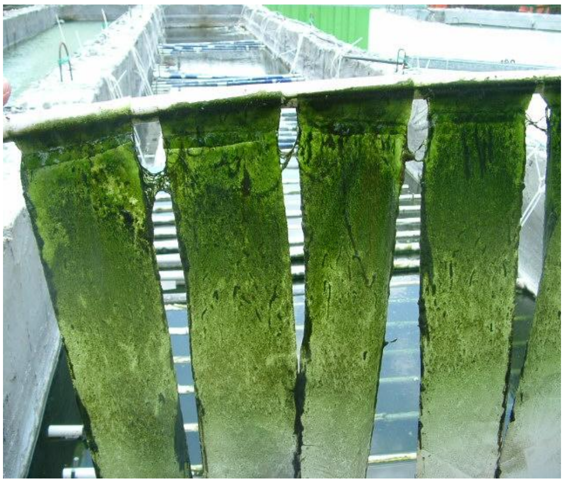

Figure 3.23: Concrete Tunnels Pond Showing the Synthetic Cloth Ribbons Used as Fixed Medium for Biomass Attachment

Figure 3.24: Concrete Hexagonal Pond - Simulation of Rectangular Shaped Seasonal Reservoir

Figure 3.25: Concrete Tunnels- Simulation of Long Axis Shaped Seasonal Reservoir

Biomass development and fixation on the plastic strips will be followed microscopically and by following respiration activity of strip samples. Equipment for measuring parameters such as DO, pH, EC, and light intensity will be installed.

Expected results for Task 3 is;

1. Improvement in pollution removal indicators.
2. Construction of a field experimental unit in the SR.
3. Installation of an operative laboratory experimental system for controlled water reservoir simulation.

It was assumed that the WWT plant of Sakhnin will continue to operate and supply raw effluents. Only constraint was the availability of space in the Sakhnin WWT plant seasonal reservoir to install the equipment.

Table 3.3: Equipment installation and Retention times for SR tanks

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT1 (day)	Retention Time (day)
1	Rectangular	-	-	**60**	**30**
2	Rectangular	-	+	**60**	**30**
3	Rectangular	+	-	**60**	**30**
4	Rectangular	+	+	**60**	**30**
5	Hexagonal	-	-	**60**	**30**
6	Hexagonal	-	+	**60**	**30**
Retention Time is 60 days in winter and 30 days in summer					

3.4 TASK 4 Wetlands (Equipment Installation to Intensify WWT processes in Wetlands)

Experimental equipment required was installed for the research and development related to the reduction of the organic pollutant concentrations in the WWT plant effluent in order to permit unlimited irrigation reuse of the wetland technology.

Existing wetland (WL) experimental facilities in the area of the SRDC will be used for controlled experiments on effluent quality improvement by feeding the WL units with the Seasonal Reservoir effluent (pumped from the SR outlet) according to the experimental program.

Various plants with different sensitivities and different removal capacities to the various pollutants were tested. Using plants with specific commercial or decorative values will also be tested. A greenhouse in the Galilee – Israel, specializing in hydrophobic plants with commercial value will supply the plants for the experimental constructed wetland ponds.

18 plastic tanks, 120 liters in volume each were used with sampling equipments (Figure 3.26 and Figure 3.27). Nine of the tanks filled with three layers of of different sized rock and gravel, other nine tanks filled with two layers of different and bigger sized rock and gravel (Figure 3.28 and Figure 3.29).

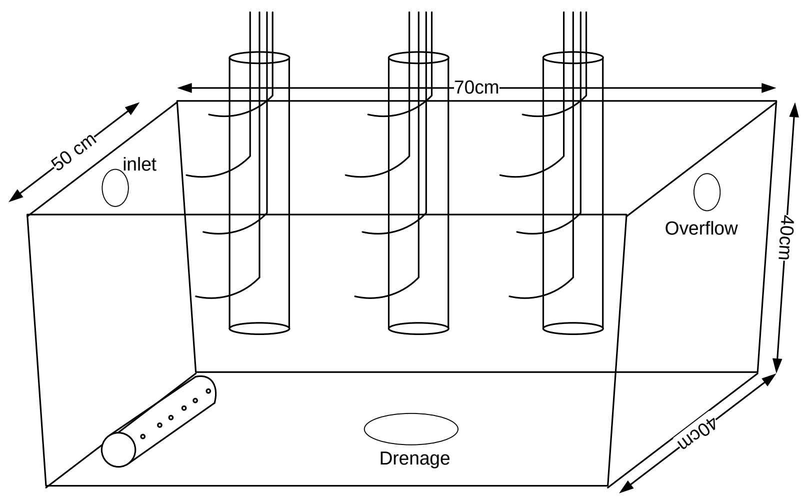

Figure 3.26: Scheme of Wetland Tank

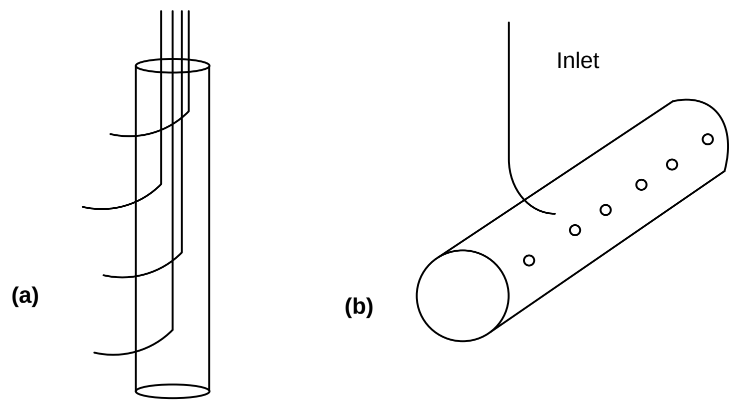

Figure 3.27: (a) Sampling Unit (to take samples from different zones) (b) Subsurface water distribute equipment

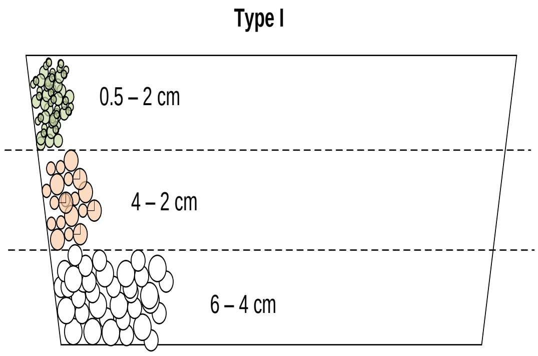

Figure 3.28: Scheme of Type I Wetland Tank

Figure 3.30: The Array of Task 4 Simulation Tanks (Constructed Wetlands)

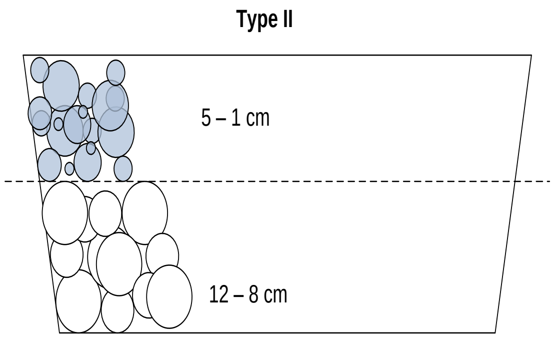

Figure 3.29: Scheme of Type II Wetland Tank

Figure 3.31: Single CWL Simulation Tank

Plants from the greenhouse were planted in the experimental ponds instead of the existing plants. Development of the plants were monitored by conventional vegetative biomass follow-up procedures and measures (weight and height of plants, leave size, crop weight, etc.).

Expected results for Task 4 is;

1. Removal of suspended solids.
2. Removal of BOD and COD.

3. Removal of nitrogen and phosphorus.
4. Permitting use of the water for drip irrigation without clogging interruptions or discharge of excess water to the environment without risking water resources.

The constructed wetland ponds in the SRDC will be available for the LIFE project. Only constraint was the availability of treated effluent from the seasonal reservoir and wetlands of the Sakhnin WWT plant.

Table 3.4: Equipment installation and Retention times for each tank

Reactor Number	Medium Type	Plants Type	Retention Time (day)	Aeration
1	Type I	Cane	5	+
2	Type I	Cane	5	-
3	Type I	Reed	5	+
4	Type I	Reed	5	-
5	Type I	Sugar Cane	5	+
6	Type I	Sugar Cane	5	-
7	Type I	Sugar Cane	3	+
8	Type I	Sugar Cane	3	-
9	Type I	Reed/Cane	5	+
10	Type II	Cane	5	+
11	Type II	Cane	5	-
12	Type II	Reed	5	+
13	Type II	Reed	5	-
14	Type II	Sugar Cane	5	+
15	Type II	Sugar Cane	5	-
16	Type II	Sugar Cane	3	+
17	Type II	Sugar Cane	3	-
18	Type II	Reed/Cane	5	+

Table 3.5: Table showing reactor numbers for each task

Task Number	Reactor Numbers																	
	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13	R14	R15	R16	R17	R18
T1	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
T2	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
T3	X	X	X	X	X	X												
T4	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X

3.5 Experimental Protocol

This study contains the first 8 months results and discussion including start-up period of the experimental setups which are going to be tested for 3 years.

3.5.1 Influent

Influent is a mix of industrial and domestic wastewater of Sakhnin community where most of the people usually earn their life by farming. They have different kinds of vegetable and fruit farms but mostly olive trees which are common in such geographic conditions. Hence olive oil factories mainly determines characteristic of influent and have major contribution in pollution. Each tasks influent comes from the treatment system which is belonged to but wetlands (Figure 3.32). Hence influent characteristics for each task will be given in Chapter 4.

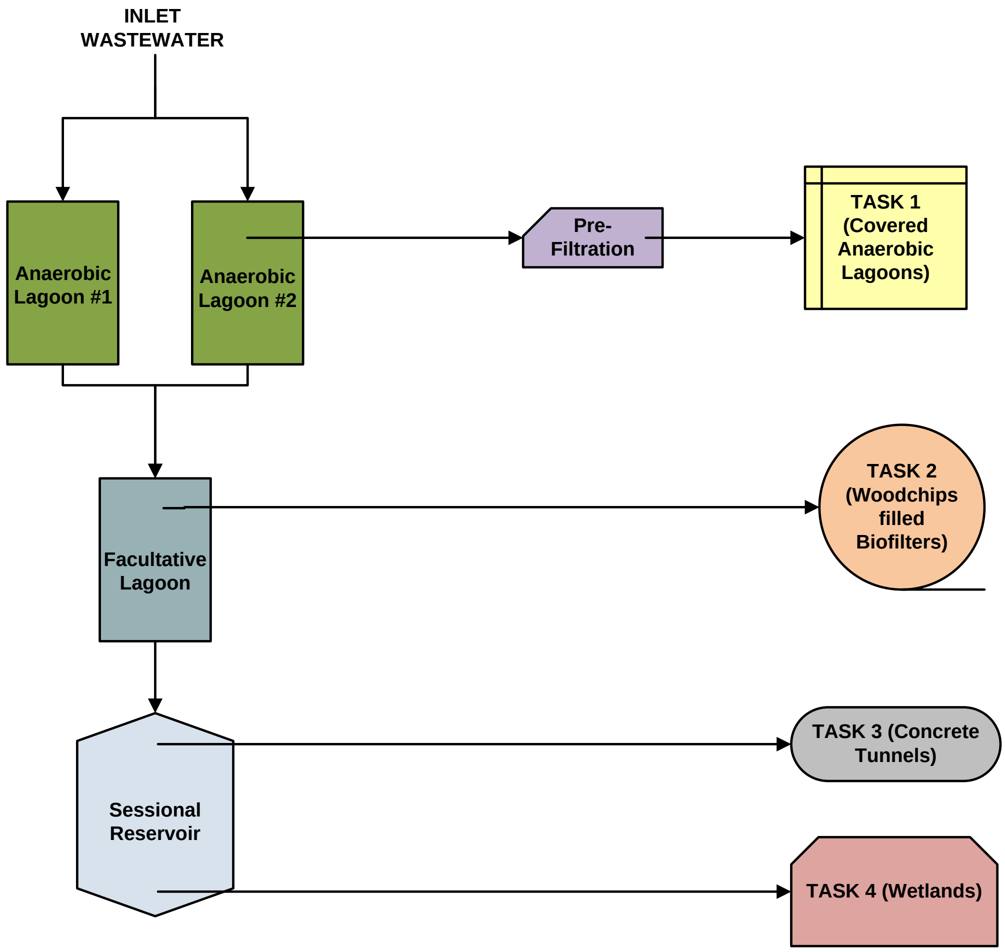

Figure 3.32: Wastewater flow scheme

The temperature has not been measured during the experiments. It was same for all tasks according to weather conditions. It was in the range of 25 to 28 ^{o}C in day time and 12 to 14 ^{o}C at nights. It should be noted that precipitation varies from approximately 700 mm in the northern part of Israel where experimental area was built. The annual evaporation is 180-200 millimeter.

3.5.2 Sampling Periods

In order to monitor and examine effectiveness and potential of different designs, i.e. medium types, covers, etc; twenty one liter of wastewater samples from the system were collected usually twice a week (40 liters a week) with an exception of delays between 3 to 6 days depending on weather conditions, specific holidays, and maintenance processes. The samples then were analyzed at the laboratories of the Galilee Society according to the standard methods of analysis.

3.5.3 Parameters

In order to assess the effectiveness of the upgraded wastewater treatment system, it was necessary to analyze the treated wastewater according to various parameters previously decided upon, including pH, COD (Chemical Oxygen Demand), BOD (Biological Oxygen Demand), TKN (Total Kjeldahl Nitrogen), NH_4^+ (Ammonium Ion), PO_4^{-3} (Phosphate) concentrations, conductivity, TSS (Total Suspended Solids), and VSS (Volatile Suspended Solids).

3.5.4 Analyzing Methods

Some of the analyses were done according to conventional wastewater analytical methods.

3.5.4.1 pH Analysis

A Hanna HI 8521 pH meter (Hanna Instruments, Italy) was used for measuring pH values. Before each measurement it was calibrated at room temperature using buffers of pH 4.0 and 7.0.

Figure 3.33: pH meter

3.5.4.2 COD Analysis

For all COD analyses "HACH DR/2010 Spectrophotometer" used with HACH COD kit.

2 ml of samples were poured into COD digestion reagent vials. Vials then located into a thermal block. The block was left for the analysis for two hours at 150ºC. After cooling phase, COD measurement was done by the spectrophotometer.

Figure 3.34: COD Reactor

Figure 3.35: Spectrophotometer

3.5.4.3 BOD Analysis

For BOD_5t (BOD total) analysis WTW OxiTop IS 12 BOD analyze kit is used.

Bottles which have samples, 2 tablets of sodium hydroxide and magnetic stirring rod were kept in a special refrigerator for 5 days at 20^{o}C.

Figure 3.36: BOD analyze kit

BODs (BOD soluable) was measured with YSI Model 58 Field DO Meter and YSI 5905 BOD Probe.

Figure 3.37: Dissolved Oxygen Meter with BOD Probe

3.5.4.4 TSS and VSS Analysis

Filter papers which was already dried at 105 ^{o}C for 30-40 min. and waited in desiccator for another 20-30 min. were weighted on a sensitive scale.

Figure 3.38: Sensitive Scale

After weight was noted, papers were put in the filtering contrivance.

Figure 3.39: Filtering Contrivance

Than filter paper waited 60 min. for dry out in the heater and another 20 min. in the desiccator. After drying process filter paper was weighted again and TSS calculated using the following formula;

$$TSS = \frac{Second\ Weight\ (mg) - First\ Weight(mg)}{Sample\ Volume\ (ml)}$$

To calculate VSS values, filter papers were combusted at 500 ^oC for 20 min. in heater and waited in desiccator for another 20 min. than difference calculated as VSS value.

3.5.4.5 NH_4^+ Analysis

Ammonium analysis were done according to colorimetric method of standart methods (1998). Two reagents were used. 15gr sodium stirate tribasic, 15gr sodium phosphate tribasic and 1.5gr EDTA-Na were mixed in 500ml water. 31.5gr phenol and 0.1gr sodium nitroproside added to this solution to have first reagent.

For second reagent, 16gr sodium hydroxide mixed in 1 lt water and 30 ml sodium hypochlorite 2.7-3.5% added (until the volume with this reagent to 50ml). The wave length used is 635nm.

Figure 3.40: The spectrophotometer used for analysis.

3.5.4.6 PO_4^{-3} Analysis

This analyze were done according to standard methods (1998) 4-152. 2 reagents, the ammomium molybdate and the stannous chloride reagent were used. The wavelength is 880nm.

Figure 3.41: The spectrophotometer used for analysis and vials.

3.5.4.7 TKN Analysis

For 5 ml sample, 5ml digestion solution added and heated for 20min. one drop of indicator methyl red added, then NAOH 1N added until the color get yellow. After addition of diluted acid until the solution gets pink color, NaOH 0.2N added to solution until it gets yellow color again. Water added to solution until 250 ml, then same method used which is used for NH_4^+ analysis.

Figure 3.42: The setup used for TKN analysis.

4. RESULTS

At all experiments generally two different systems compared to see which one is more effective. Covered and uncovered tanks in TASK 1 for instance. For both type systems, different setups used such as different retention times, volumes, materials etc. considering operating conditions could be changed by time. For example retention time for a tank could be 10 days for a week and 15 days for another week due to different conditions. Hence in this study, average values of these different conditions discussed.

4.1 Effectivness of Experimental Designs on TASK 1 (Anaerobic lagoon) Parameters

4.1.1 pH

pH is an important measure for anaerobic reactors to check the stability. During the study, pH values of outlet were in the range of 6 - 9,4 with an average of 7,65 which is acceptable for general discharge rules (For proper treatment, wastewater pH should normally be in the range of 6.5 to 9.0 (ideally 6.5 to 8.0) *(Spellman, 2003).*

36 pH readings of inlet has shown in Figure 4.1. pH value of inlet wastewater to Task 1 changed in the range of 6,69 - 7,67 with an average of 7,14.

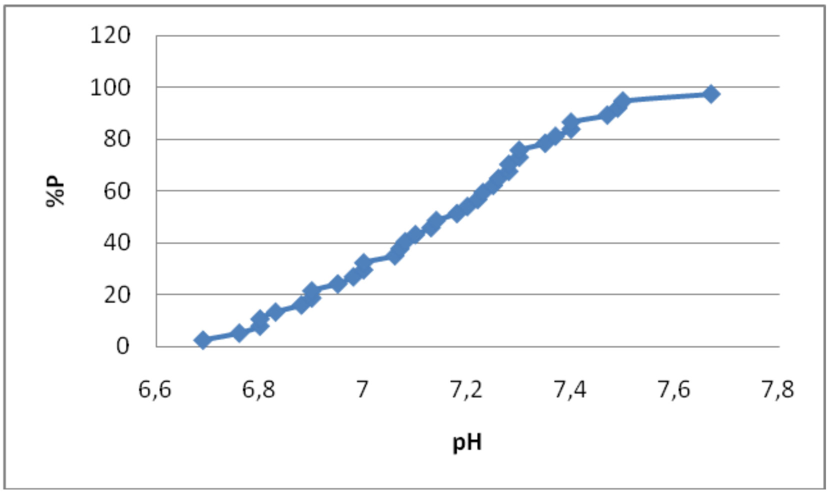

Figure 4.1: Inlet pH

As shown in the Table 4.1, cover color and retention time do not have any major affect on the pH values.

Table 4.1: pH values of covered tanks

Tank #	Color	RT (day)	Range	Average	Standard Deviation
2	White	8	7,2-8,75	7,61	0,49
5	Black	8	6,8-8,2	7,5	0,41
3	White	4	6,7-7,7	7,23	0,32
7	Black	4	6,9-8,57	7,62	0,53
4	White	8	6,89-8,2	7,44	0,43
13	Black	8	6,87-8,1	7,49	0,41
9	White	4	6,9-8,2	7,63	0,36
18	Black	4	6,74-8	7,36	0,38
8	Black	2	6,9-7,94	7,4	0,29
6	Black	6	7-7,8	7,46	0,23
1	Black	10	7,1-8,1	7,57	0,33

While minimum pH values were almost the same, maximum pH values reached approximately 9,0. As shown in the Table 4.2 tanks without cover reached a little higher pH values than the covered ones. It should be noted that tables above also includes values from startup period. Hence to make a accurate decision first and last two months experiment values were examined.

Table 4.2: pH values of uncovered tanks

Tank #	Color	RT (day)	Range	Average	Standard Deviation
10	-	10	6,7-9,4	8,03	0,71
11	-	8	6,7-8,5	7,92	0,60
12	-	4	6-9	7,87	0,82
14	-	8	6,33-8,9	7,83	0,85
15	-	6	6,8-8,4	7,76	0,59
16	-	4	6,7-9,1	8,08	0,69
17	-	2	6,7-9,2	7,91	0,82

As seen in the Table 4.3 at first two month pH values were slightly higher as a result of startup period. Eventually at last two months pH values for both covered and uncovered systems were equal while inlet averages were also same (pH 7) showing there is no difference between covered and uncovered tanks.

Table 4.3: Average values of first and last two months

Tank Specification	First Two Months Average (Inlet = 7)	Last Two Months Average (Inlet = 7)
Covered	7,5	7,1
Uncovered	7,8	7,1

As an example, Tank 9 is shown in Figure 4.2 For both systems since their pH values were equal and Tank 9's pH closest to the overall average of pH values.

As a result, covered tanks have no advantage over uncovered tanks in terms of pH.

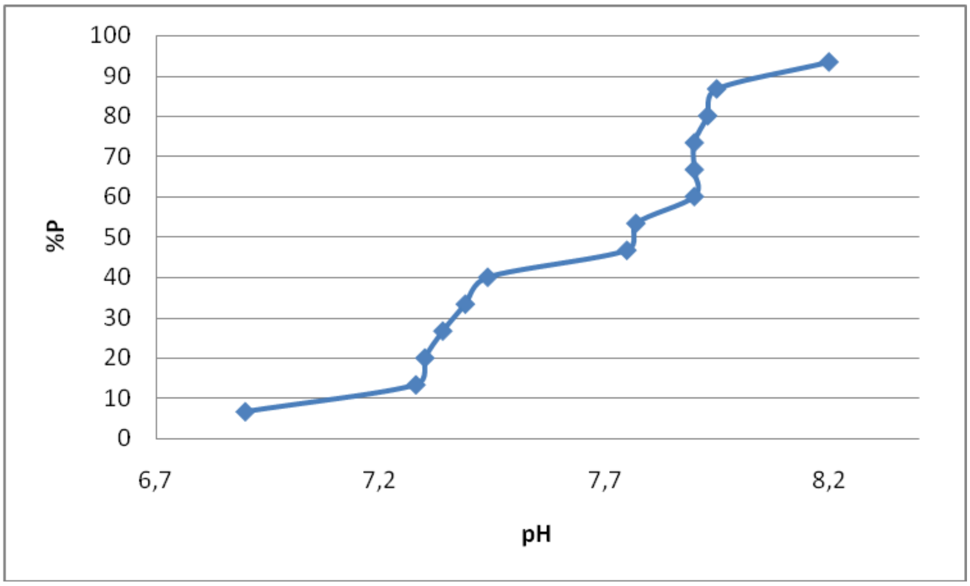

Figure 4.2: Outlet pH graph of Tank 9

4.1.2 Conductivity

Conductivity is a measure used to determine mineralization, variation or changes in water quality, corrosive effect etc. Corrosion of metallic surfaces by water that is high in dissolved solids causes problems in equipments. Effluent water will be used for irrigation and high dissolved solids can be a problem. Conductivity is measured in mS (millisiemens). Conductivities of inlet water were ranged in 1075-2170 mS with an average of 1741 mS (Figure 4.3).

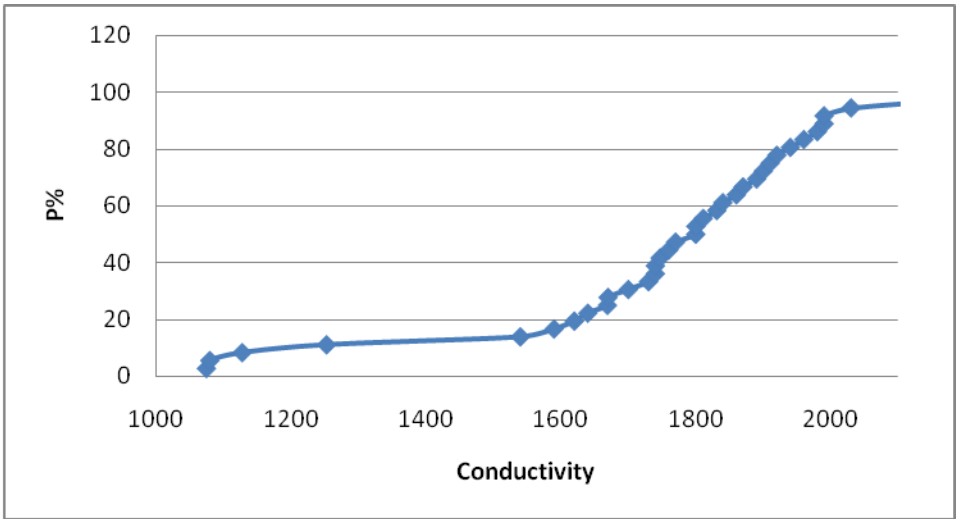

Figure 4.3: Inlet Conductivity

In general, water for irrigation purposes must have a low to medium salinity level (i.e. electrical conductivity of 600 to 1700 mS) (Aiello, 2007).

In all tanks average conductivity decreased from 1741 to 1481 mS which indicates that water quality has been raised 15%. Values are acceptable to reuse water for irrigation.

Table 4.4: Conductivity values of covered tanks

Tank #	Color	RT (day)	Range (mS)	Average (mS)	Standard Deviation
2	White	8	988-2060	1512	367
5	Black	8	724-1760	1281	429
3	White	4	1040-2010	1630	352
7	Black	4	656-1920	1392	474
4	White	8	960-1960	1573	373
13	Black	8	990-2050	1541	405
9	White	4	580-1900	1428	420
18	Black	4	990-2130	1618	397
8	Black	2	897-1900	1528	391
6	Black	6	1024-1910	1478	334
1	Black	10	988-2060	1512	399

Table 4.5: Conductivity values of uncovered tanks

Tank #	Color	RT (day)	Range (mS)	Average (mS)	Standard Deviation
10	-	10	795-2510	1473	490
11	-	8	970-2140	1473	465
12	-	4	910-2380	1460	471
14	-	8	840-2340	1434	525
15	-	6	940-2270	1498	467
16	-	4	850-1993	1433	407
17	-	2	783-1871	1411	419

Table 4.6: Removal rate comparison

Tank Specification	Average Cond. Inlet (mS)	Average Cond. Effluent (mS)	Removal Rate
Covered	1741	1499	14%
Uncovered	1741	1454	16%

If average values of first and last two month period were checked, there was no difference between two systems considering also inlet average value belongs to that period (Table 4.7).

Table 4.7: Average conductivity values of first and last two months

Tank Specification	First Two Months Average (mS) / Removal % (Inlet = 1807)	Last Two Months Average (mS) / Removal % (Inlet = 1332)
Covered	1720 / 5%	1212 / 9%
Uncovered	1591 / 12%	1202 / 10%

Average value of Tank 6 is the closest to overall average value. Hence graph of Tank 6 is selected as representative tank (Figure 4.4).

In conclusion, covered tanks gave better results than covered tanks.

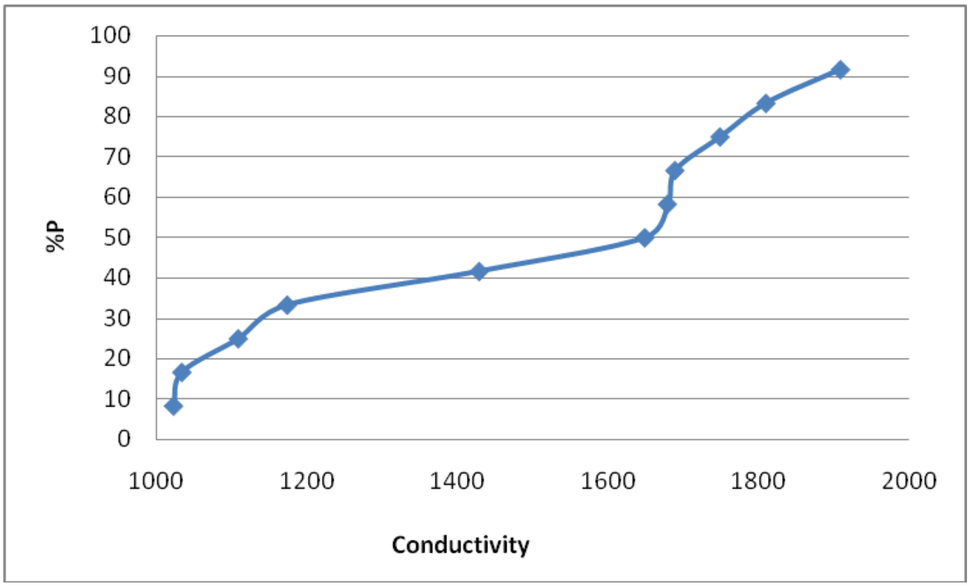

Figure 4.4: Conductivity of Tank 6

4.1.3 COD

COD is a basic parameter to determine oxygen-consumption capacity of inorganic and organic matter present in water or wastewater. During the study, inlet COD values ranged in 117 – 682 mg/l with an average of 482 mg/l (Figure 4.5).

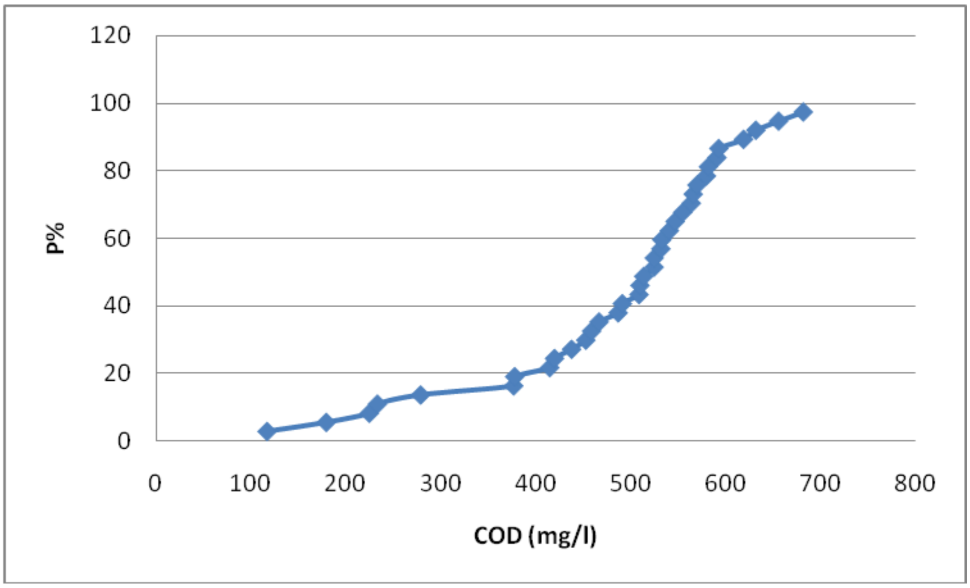

Figure 4.5: COD Inlet

As seen in the Tables 4.8 and 4.9, covered tanks have given better results than uncovered tanks and values generally under 400 mg/l. In order to determine COD removal rate of covered and uncovered tanks; average COD values were compared by considering inlet average value was 482 mg/l. Average COD value for covered ones was 303 mg/l while uncovered tanks average was 320 mg/l.

Table 4.8: COD values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standart Deviation
2	White	8	143-464	275	114
5	Black	8	78-452	271	144
3	White	4	97-537	327	138
7	Black	4	78-456	269	136
4	White	8	95-548	342	143
13	Black	8	78-588	318	172
9	White	4	49-671	335	194
18	Black	4	100-568	326	136
8	Black	2	75-452	291	114
6	Black	6	80-601	400	144
1	Black	10	80-467	274	130

Table 4.9: COD values of uncovered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standart Deviation
10	-	10	61-549	332	141
11	**-**	8	27-582	275	186
12	**-**	4	44-783	314	209
14	**-**	8	100-624	333	200
15	**-**	6	140-845	384	242
16	**-**	4	41-791	319	213
17	**-**	2	97-627	282	179

As seen in the Table 4.10, covered tanks were slightly more effective on COD removal.

Table 4.10: COD removal rate comparison

Tank Specification	Average CODt Inlet (mg/l)	Average Effluent CODt (mg/l)	Removal Rate
Covered	482	303	37%
Uncovered		320	34%

It was clear that covered tanks provided good anaerobic condition which is effective on COD and nitrogen removal by anaerobic oxidation (hydrolyses, fermentation, methanogenesis phases).

Standart COD value for evaluation is characterized by CODt (Total), hence CODt is considered instead of CODs (Soluble). Results of CODs experiments are included in the appendix for more information.

Table 4.11: Average values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 578)	Last Two Months Average(mg/l) / Removal % (Inlet = 434)
Covered	374 / 35%	300 / 31%
Uncovered	422 / 26%	336 / 23%

Tank 2 has given the best result between covered tanks and Tank 10 was the best between uncovered tanks (Figure 4.6 and Figure 4.7).

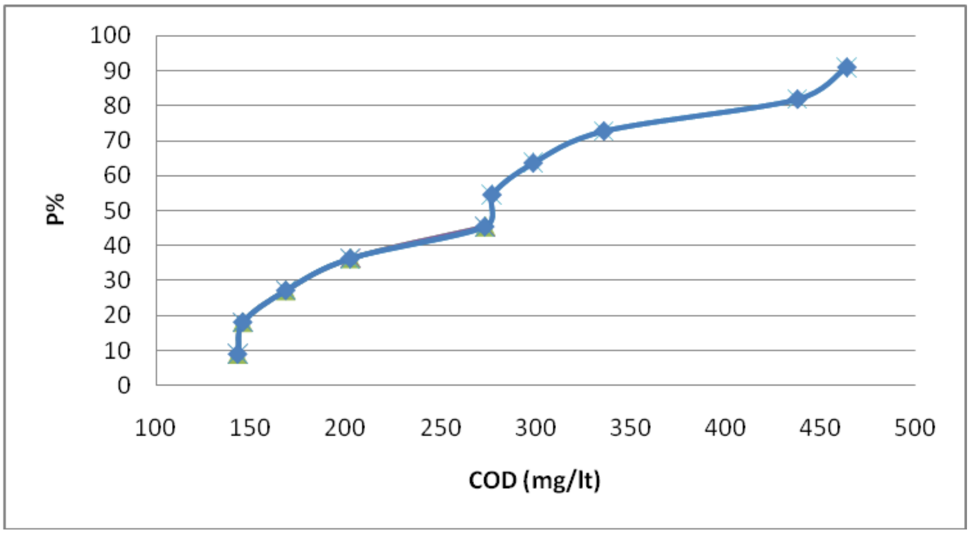

Figure 4.6: Outlet graphic of Tank 2 (Covered)

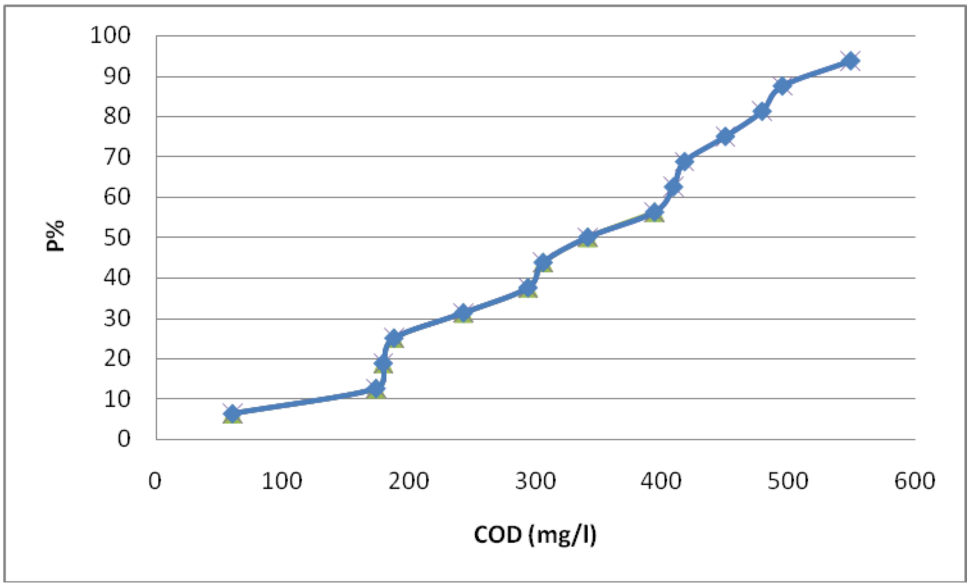

Figure 4.7: Outlet graphic of Tank 10 (Uncovered)

4.1.4 BOD_5

BOD is a standard measure of wastewater strength that quantifies the oxygen consumed in a stated period of time; usually at 20°C and 5 days. Inlet BOD value ranged in 230 – 534 mg/l with an average of 382 mg/l (Figure 4.8).

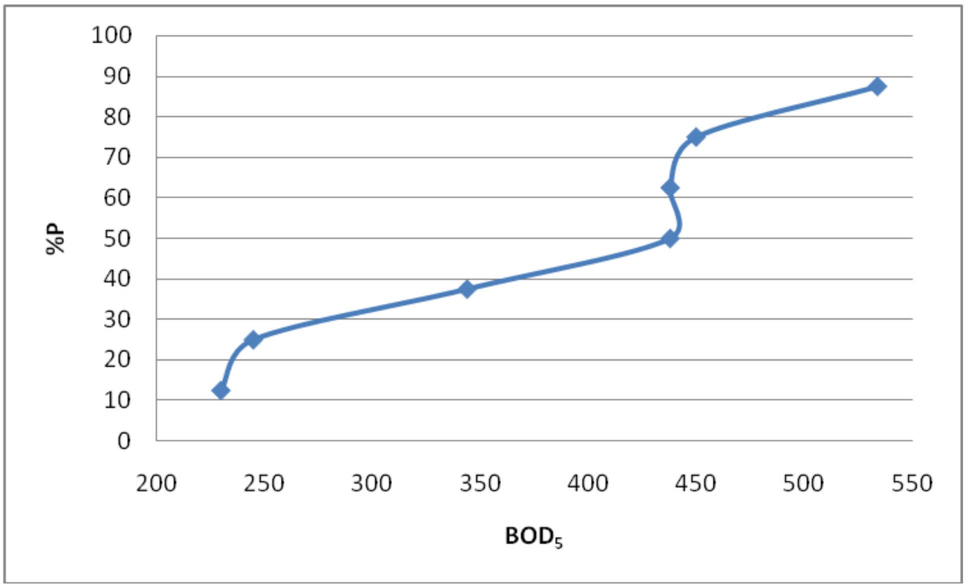

Figure 4.8: Inlet BOD

Same as the removal rate of COD, covered tanks removed BOD better than uncovered tanks with considering average values of effluent. However number of experiments was not quite enough to make a complete comparison between two different experimental setups (Table 4.12 and Table 4.13). Average BOD_5 value for covered tanks was 230 mg/l while uncovered tanks average value was 234 mg/l (Table 4.14).

Table 4.12: BOD_5 values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
2	White	8	175	175	-
5	Black	8	190	190	-
3	White	4	-	-	-
7	Black	4	-	-	-
4	White	8	170-335	252	117
13	Black	8	-	-	-
9	White	4	205	205	-
18	Black	4	-	-	-
8	Black	2	309	309	-
6	Black	6	-	-	-
1	Black	10	190-315	252	88

Table 4.13: BOD_5 values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
10	-	10	215-312	263	69
11	**-**	8	-	-	-
12	**-**	4	205	205	-
14	**-**	8	-	-	-
15	**-**	6	-	-	-
16	**-**	4	-	-	-
17	**-**	2	-	-	-

Table 4.14: BOD removal rate comparison

Tank Specification	Average BOD Inlet (mg/l)	Average Effluent BOD (mg/l)	Removal Rate
Covered	382	230	40%
Uncovered		234	39%

As shown in Table 4.12, covered tanks were slightly effective which had better results in COD removal. Another comparison made between covered tanks to investigate which color was more effective; there was no significant difference at all. Due to lack of available number of experiments it was not possible to compare first and last two months period also (Table 4.15).

Table 4.15: Average values of first and last two months

Tank Specification	First Two Months Average (mg/l)	Last Two Months Average (mg/l)
Covered	-	206
Uncovered	210	-

4.1.5 TSS

Total suspended solids (TSS) include all particles suspended in water which will not pass through a filter. As level of TSS increase, a water body begins to lose its ability to support a diversity of aquatic life. Suspended solids absorb heat from sunlight, which increases water temperature and subsequently decreases levels of dissolved oxygen (warmer water holds less oxygen than cooler water) (DEQ, 2002). Inlet TSS value ranged in 55 – 204 mg/l with an average of 123 mg/l (Figure 4.9).

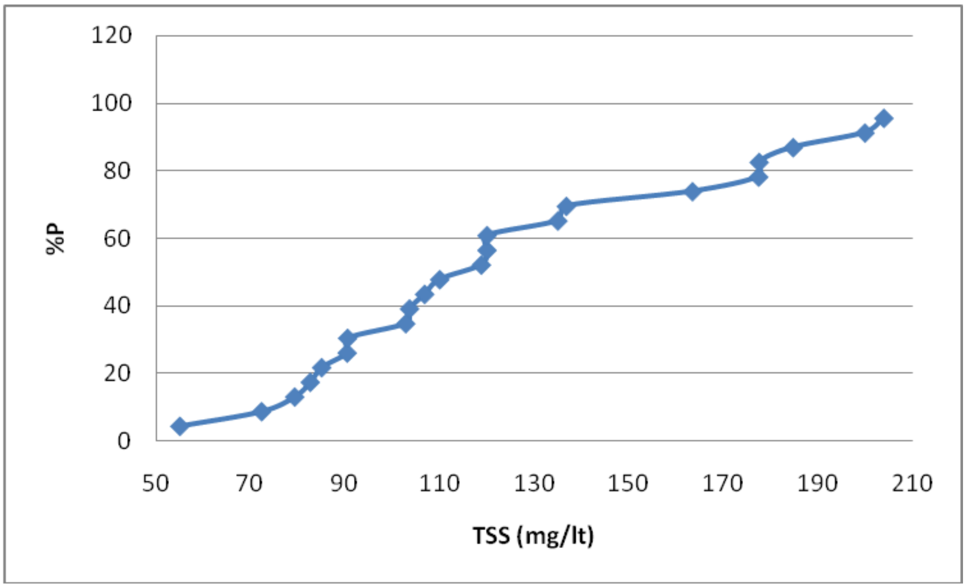

Figure 4.9: Inlet TSS

The effect of open air condition on uncovered tanks appeared once again and subsequently the TSS value increased in uncovered tanks instead of a decrease (Table 4.16 and Table 4.17).

Table 4.16: TSS values of covered tanks

Tank #	Color	RT (day)	Range(mg/l)	Average(mg/l)	St.Deviation
2	White	8	8-131	56	43
5	Black	8	10-124	72	46
3	White	4	49-255	91	64
7	Black	4	50-153	92	46
4	White	8	50-120	78	23
13	Black	8	61-208	100	55
9	White	4	50-376	146	123
18	Black	4	40-174	102	51
8	Black	2	40-86	64	19
6	Black	6	38-112	74	30
1	Black	10	40-108	70	25

Table 4.17: TSS values of uncovered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
10	-	10	120-468	210	137
11	-	8	50-423	176	170
12	-	4	45-470	211	162
14	-	8	52-145	86	51
15	-	6	20-70	45	35
16	-	4	46-536	183	236
17	-	2	60-768	275	331

Covered tanks average effluent value was 86 mg/l while it was 169 mg/l for uncovered tanks. Covered tanks have removed 31% of TSS, while uncovered tanks were unsuccessful (Table 4.18). It should be noted again that the tables above include values from the startup period. Last two months values were better and still covered tanks performed better than uncovered tanks (Table 4.19). The difference between two systems may because of scattered biomass.

Table 4.18: TSS removal rate comparison

Tank Specification	Average TSS Inlet (mg/l)	Average Effluent TSS (mg/l)	Removal Rate
Covered	123	86	31%
Uncovered		169	-37%

Table 4.19: Average values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 131)	Last Two Months Average(mg/l) / Removal % (Inlet = 167)
Covered	101 / 23%	60 / 64%
Uncovered	104 / 21%	82 / 51%

Tank 2 was the best between covered tanks in terms of TSS removal. In uncovered tanks Tank 15 was the best but it should be noted that the experiment number for this tank was only two. Hence to be more precise about the distribution of values by time Tank 10 is selected (Figure 4.10 and Figure 4.11).

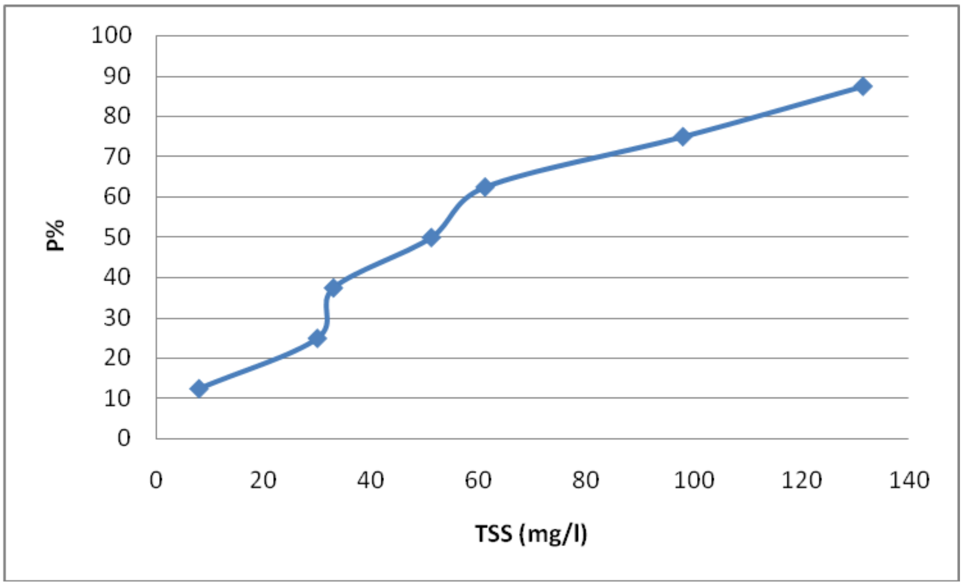

Figure 4.10: Outlet TSS value of Tank 2 (Covered)

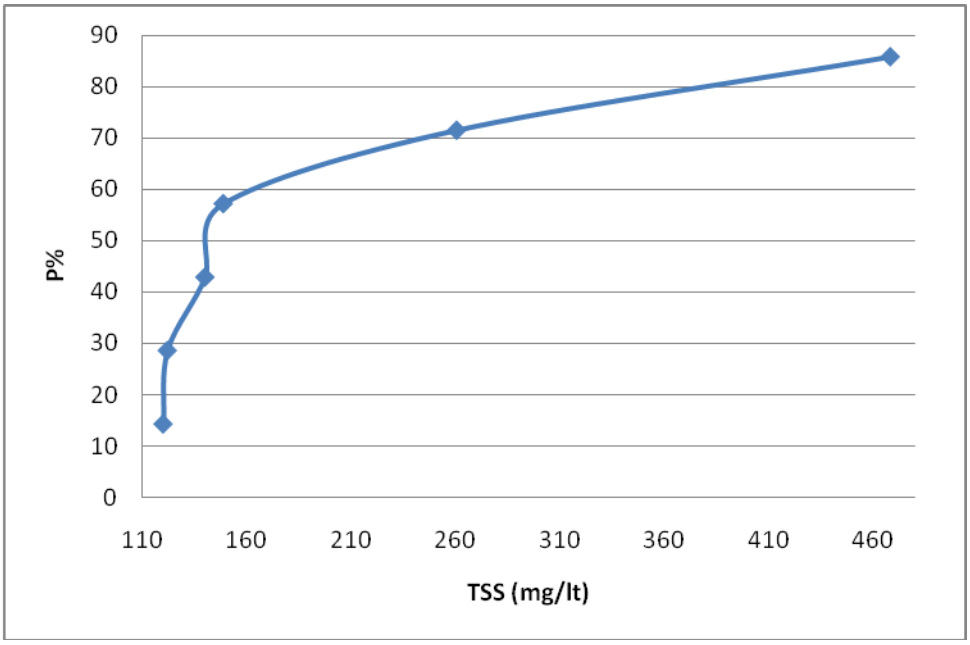

Figure 4.11: Outlet TSS value of Tank 10 (Uncovered)

4.1.6 VSS

Volatile suspended solids are organic content of suspended solids in a water or wastewater that lost on ignition (heating to 550 degrees C^{o}). VSS analyze is useful because it gives a general approximation of the amount of organic matter present in the solid fraction of wastewater. The loss of mass during combustion is not confined to organic material, and may include the decomposition or volatilization of some mineral salts (DEQ, 2002). During the study experiments VSS value ranged in 55-174 mg/l with an average of 106 mg/l (Figure 4.12).

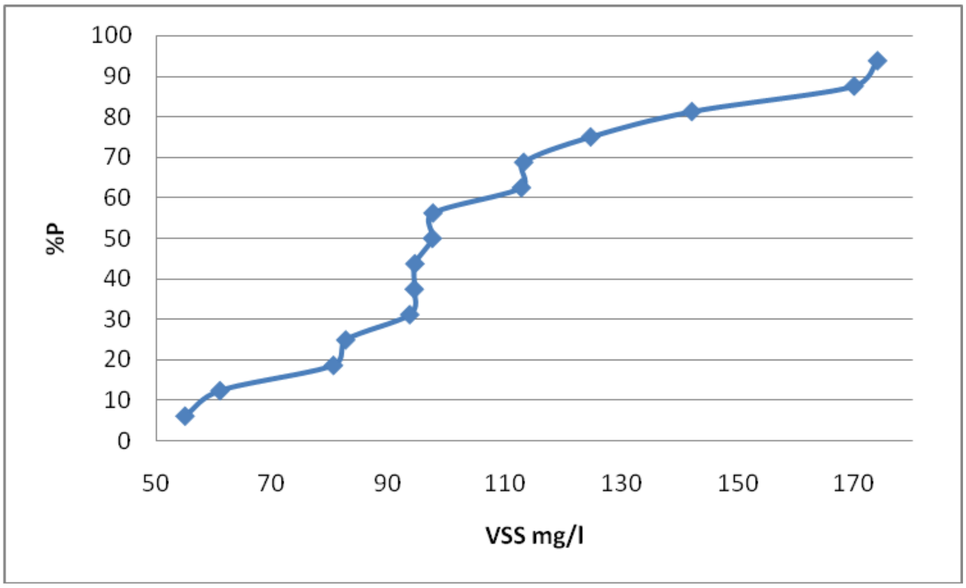

Figure 4.12: Inlet VSS

Uncovered tanks have reached higher values as expected due to open air condition. VSS values must be lower than TSS, since TSS value includes VSS. However, especially in uncovered VSS results, as seen in Table 4.20 and Table 4.21 some VSS values were higher than TSS. The reason to that was the number of experiments (some of the tanks only have two experiments) which effect average.

In the light of situations above, it is not possible to make a accurate comparison between two systems in terms of VSS.

Table 4.20: VSS values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
2	White	8	60-131	93	36
5	Black	8	74-111	94	18
3	White	4	48-223	101	82
7	Black	4	140	140	-
4	White	8	60-90	78	16
13	Black	8	95	95	-
9	White	4	98-348	207	128
18	Black	4	116-146	128	16
8	Black	2	-	-	-
6	Black	6	76-103	90	19
1	Black	10	46-90	66	19

Table 4.21: VSS values of uncovered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
10	-	10	140-417	264	140
11	**-**	8	392	392	-
12	**-**	4	128-426	260	124
14	**-**	8	-	-	-
15	**-**	6	-	-	-
16	**-**	4	497	497	-
17	**-**	2	112-731	421	438

4.1.7 NH_4^+

NH_4^+ is a form of ammonia found in solution, which can be used as the nitrogen source along with nitrate. Minimum value was 16 mg/l and maximum value was 103 mg/l with an average of 61 mg/l during the study (Figure 4.13).

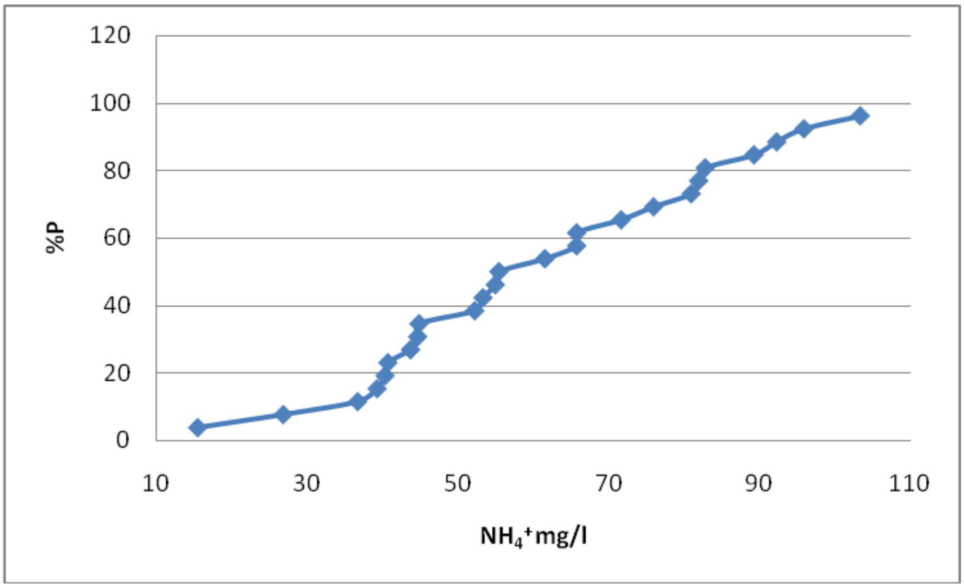

Figure 4.13: Inlet NH_4^+

As seen in the Table 4.22 and Table 4.23 uncovered tanks were more successful than covered tanks. pH determines the distribution between free ammonia and the ammonium ion (Carl E, 1999). As at pH levels higher than 7.5 ammonium ions (NH_4^+) mostly change into ammonia (NH_3) (Eldem and Öztürk, 2006), ammonium ions concentration level was lower than covered tanks which have lower pH values.

Table 4.22: NH_4^+ values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
2	White	8	15-85	40	22
5	Black	8	15-80	42	22
3	White	4	14-94	53	27
7	Black	4	9-91	46	35
4	White	8	14-73	48	21
13	Black	8	12-87	51	25
9	White	4	7-91	51	29
18	Black	4	13-93	55	29
8	Black	2	9-99	53	32
6	Black	6	15-89	52	28
1	Black	10	15-88	46	24

Table 4.23: NH_4^+ values of uncovered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
10	-	10	6-98	37	28
11	-	8	11-98	40	33
12	-	4	5-99	35	27
14	-	8	6-102	38	32
15	-	6	2-96	42	43
16	-	4	1-88	40	30
17	-	2	10-94	36	25

Table 4.24 shows the 18% difference between covered and uncovered tanks in terms of ammonium removal. There was no major difference at different retention times when compared between covered tanks to see which one is more effective.

Table 4.24: NH_4^+ removal rate comparison

Tank Specification	Average Inlet NH_4^+ (mg/l)	Average Effluent NH_4^+ (mg/l)	Removal Rate
Covered	61	49	20%
Uncovered		38	38%

When last two months values were checked, it is seen that whole system outlet value was better as expected (Table 4.25). Also again uncovered tanks were better than covered tanks.

Table 4.25: Average values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 57)	Last Two Months Average(mg/l) / Removal % (Inlet = 37)
Covered	44 / 23%	27 / 27%
Uncovered	52 / 9%	15 / 59%

Best performed tanks; Tank 2 and Tank 12's graph is down below (Figure 4.14 and Figure 4.15).

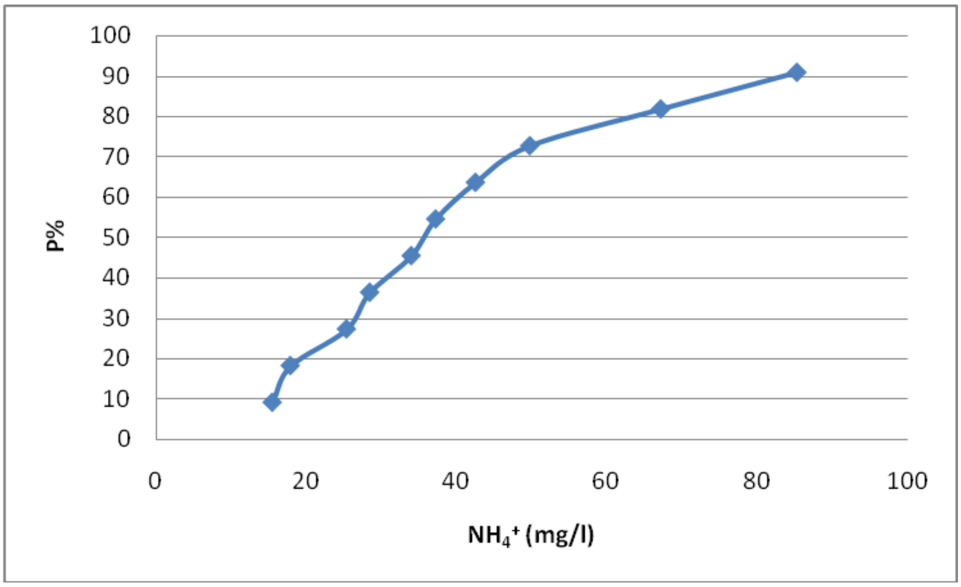

Figure 4.14: Outlet NH_4^+ graph for Tank 2 (Covered)

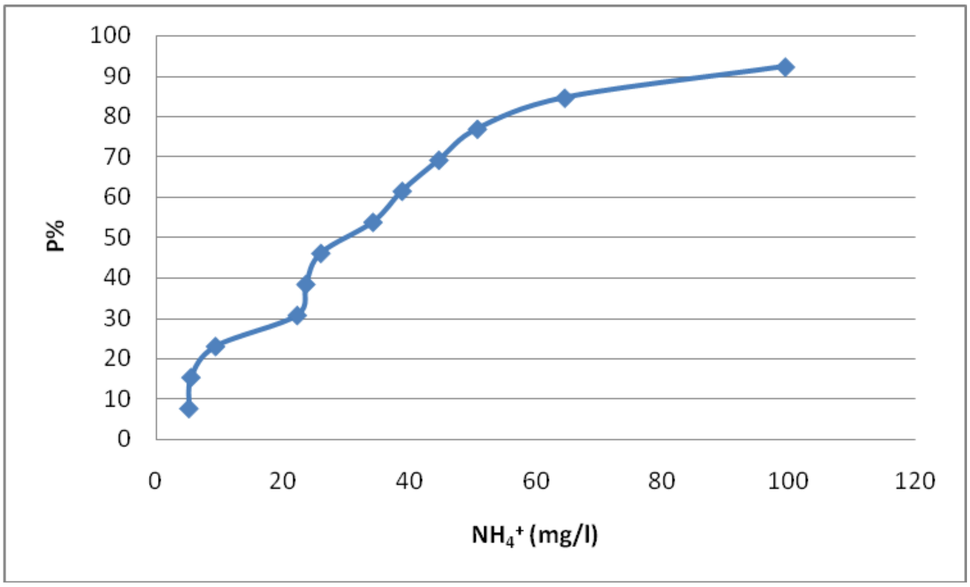

Figure 4.15: Outlet NH_4^+ graph for Tank 12 (Unovered)

4.1.8 PO_4^{-3}

Wastewater treatment normally requires removal of phosphorus to reduce algal growth. Algae may grow at PO_4^{-3} levels as low as 0.05 mg/L. Growth inhibition requires levels well below 0.5 mg/L. Since municipal wastes typically contain approximately 25 mg/L of phosphate (as orthophosphates, polyphosphates, and insoluble phosphates), the efficiency of phosphate removal must be quite high to prevent algal growth. (Stanley E, 2001). During the study, inlet PO_4^{-3} values ranged in 0.4 – 26 mg/l with an average of 14 mg/l (Figure 4.16).

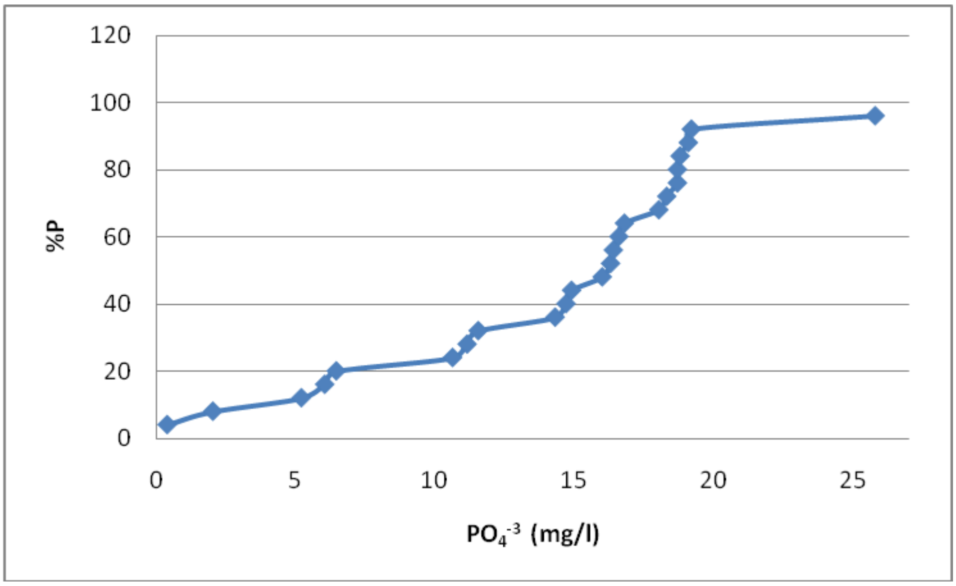

Figure 4.16: Inlet PO_4^{-3}

As shown in the Table 4.26 and Table 4.27 uncovered tanks performed better than covered tanks as in NH_4^+ removal which may be expected. There was no significant difference between covered tanks whether black or white covered.

Table 4.26: PO_4^{-3} values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
2	White	8	0,6-19	11	7
5	Black	8	2-20	12	8
3	White	4	0,5-17	12	7
7	Black	4	1-17	7	7
4	White	8	1-26	12	7
13	Black	8	1-20	12	8
9	White	4	1-24	12	8
18	Black	4	0-24	12	9
8	Black	2	0-15	9	7
6	Black	6	1-19	10	7
1	Black	10	0-21	13	8

Table 4.27: PO_4^{-3} values of uncovered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
10	-	10	1-27	10	7
11	**-**	8	1-19	10	8
12	**-**	4	0-24	11	6
14	**-**	8	0-23	9	9
15	**-**	6	0-24	11	8
16	**-**	4	0-20	8	8
17	**-**	2	0-19	7	7

There was 14% difference between covered and uncovered tanks in terms of PO_4^{-3} removal as shown in the Table 4.28 which is cannot be considered as major. As expected last two months values were better as seen in the Table 4.29.

Table 4.28: PO_4^{-3} removal rate comparison

Tank Specification	Average Inlet PO_4^{-3} (mg/l)	Average Effluent PO_4^{-3} (mg/l)	Removal Rate
Covered	14	11	21%
Uncovered		9	35%

Table 4.29: Average values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 19)	Last Two Months Average(mg/l) / Removal % (Inlet = 9)
Covered	17 / 11%	8 / 11%
Uncovered	15 / 21%	7 / 22%

As a representative graph, best performed Tank 7's graph is used below for both systems since there was no significant difference between two systems (Figure 4.17).

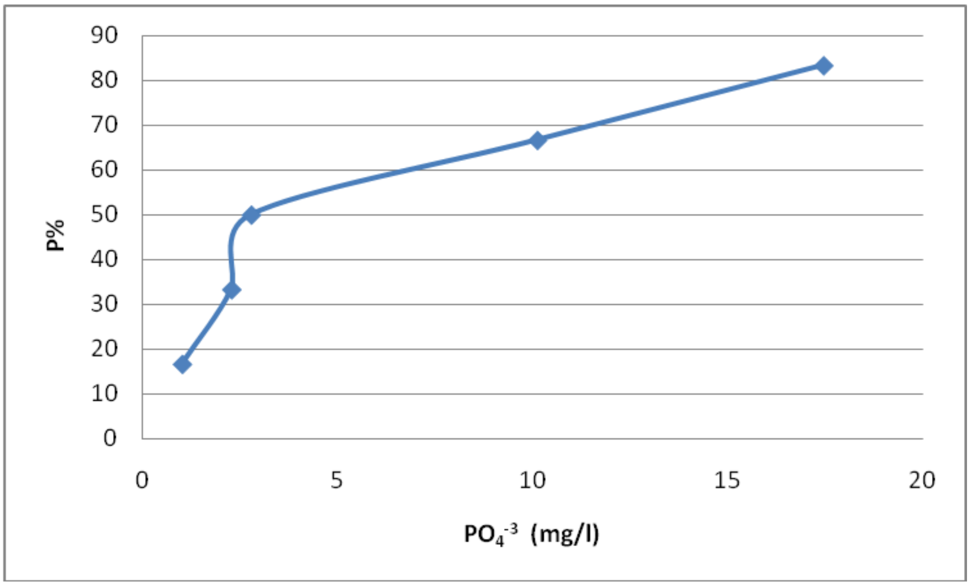

Figure 4.17: Outlet PO_4^{-3} graph of Tank 7

4.1.9 TKN

TKN is the sum of the organic plus ammonia nitrogen in a water sample which is determined by digesting and distilling the sample and then measuring the ammonia concentration in the distillate (Pankratz, 2001). Minimum value was 30 mg/l and maximum value was 201 mg/l with an average of 89 mg/l during the study (Figure 4.18).

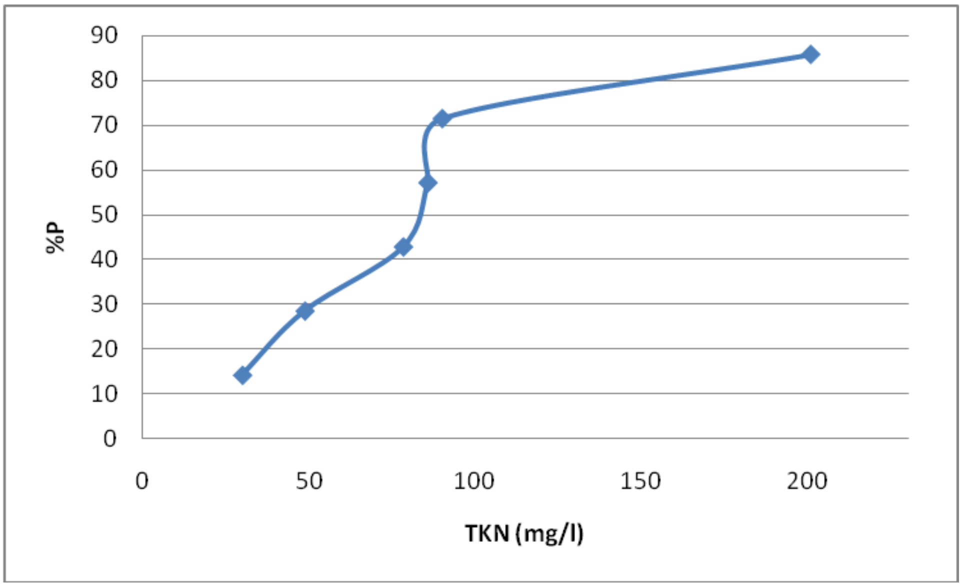

Figure 4.18: Inlet TKN

Number of experiments was not quite enough to make a complete comparison between tanks. Maximum number of experiments for a tank was two due to availability of laboratory conditions. Even tough as seen in the Table 4.30 and Table 4.31, it was clear that both covered and uncovered tanks were mostly successful in terms of TKN removal considering average inlet TKN value was 89 mg/l.

Table 4.30: TKN values of covered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
2	White	8	53	53	-
5	Black	8	65	65	-
3	White	4	0-68	34	48
7	Black	4	247	247	-
4	White	8	91	91	-
13	Black	8	75	75	-
9	White	4	164	164	-
18	Black	4	-	-	-
8	Black	2	27	27	-
6	Black	6	54-89	71	25
1	Black	10	56	56	-

Table 4.31: TKN values of uncovered tanks

Tank #	Color	RT (day)	Range (mg/l)	Average (mg/l)	Standard Deviation
10	-	10	185	185	-
11	-	8	65	65	-
12	-	4	-	-	-
14	-	8	-	-	-
15	-	6	-	-	-
16	-	4	-	-	-
17	-	2	-	-	-

4.2 Effectivness of Experimental Designs on TASK 2 (Facultative lagoon) Parameters

4.2.1 pH

15 pH readings of inlet have shown in Figure 4.19. pH value of inlet wastewater to Task 2 changed in the range of 6,81 - 7,6 with an average of 7,16.

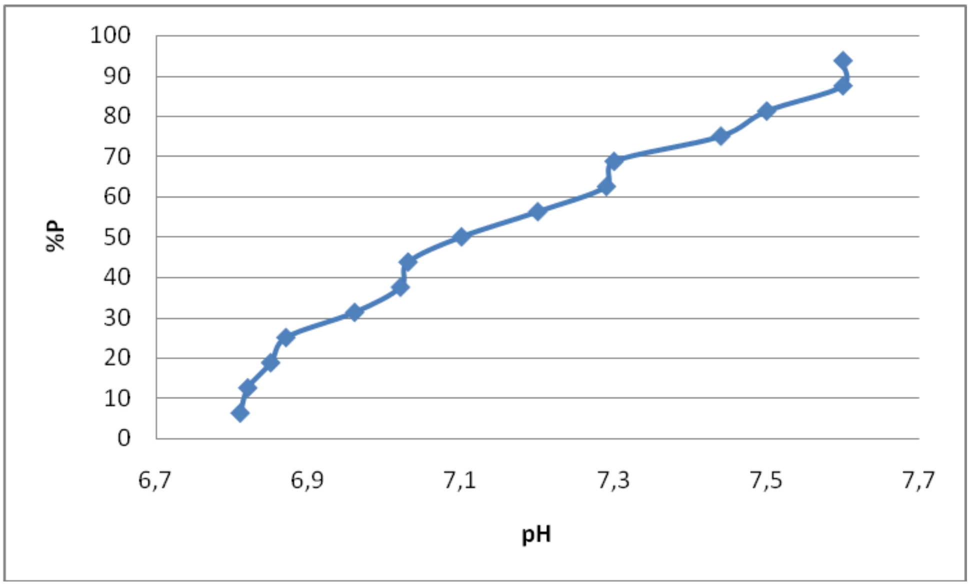

Figure 4.19: Inlet pH

Average pH values for olive woodchips filled bioreactors and pine woodchips filled bioreactors were 6,8 and 7,15 respectively. Even though different feedstock resulted different pH values, both values were in acceptable range for discharge water. Biological characteristic diversity of feedstock might cause this difference.

Table 4.32: pH values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range	Average	Standard Deviation
4	1,5	10	6	6,2-7,7	6,9	0,54
5		5	6	6,1-7,6	6,6	0,44
1	2,5	10	6	6,4-7,8	6,9	0,50
2		5	6	6,6-8,4	6,9	0,65

Table 4.33: pH values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range	Average	Standard Deviation
18	1,5	5	3	6,7-8	7,3	0,54
15	2,5	5	3	6,6-8,2	7,1	0,77
10	1,5	5	6	6,4-8	6,8	0,41
11		5	6	6,7-8	7,1	0,45
12		5	6	6,6-8,4	7,5	0,72
6		10	6	6,7-7,4	7	0,24
7	2,5	5	6	6,5-7,9	6,9	0,67
8		5	6	6,7-7,9	7,1	0,47
9		5	6	6,4-8,3	7,4	0,90
3		10	6	6,5-7,1	6,7	0,20
17	1,5	5	10	6,6-8,1	7,1	0,54
16		10	10	7,5-8,3	8,1	0,34
14	2,5	5	10	6,3-7,2	6,6	0,33
13		10	10	6,5-8,5	7,5	0,85

The difference between pH values was same at last two months (Table 4.34).

Table 4.34: Average pH values of first and last two months

Tank Specification	First Two Months Average (Inlet = 6,9)	Last Two Months Average (Inlet = 7,3)
Olive	6,3	7,1
Pine	6,8	7,4

As a reference Tank 11's outlet graph is given in Figure 4.20.

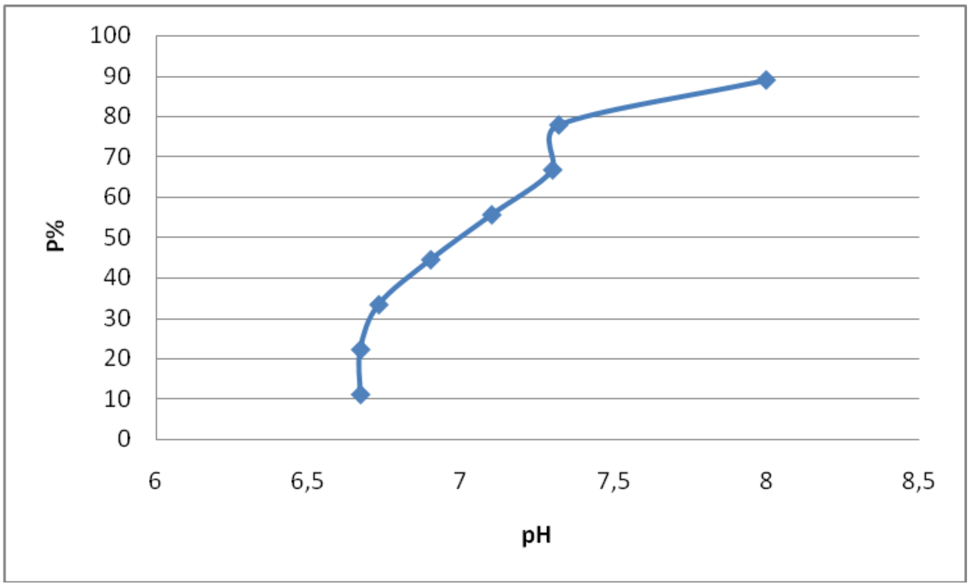

Figure 4.20: Outlet graphic of Tank 11

4.2.2 Conductivity

15 Conductivity readings of inlet have shown in Figure 4.21. Conductivities of inlet water were ranged in 1220-1865 mS with an average of 1602 mS.

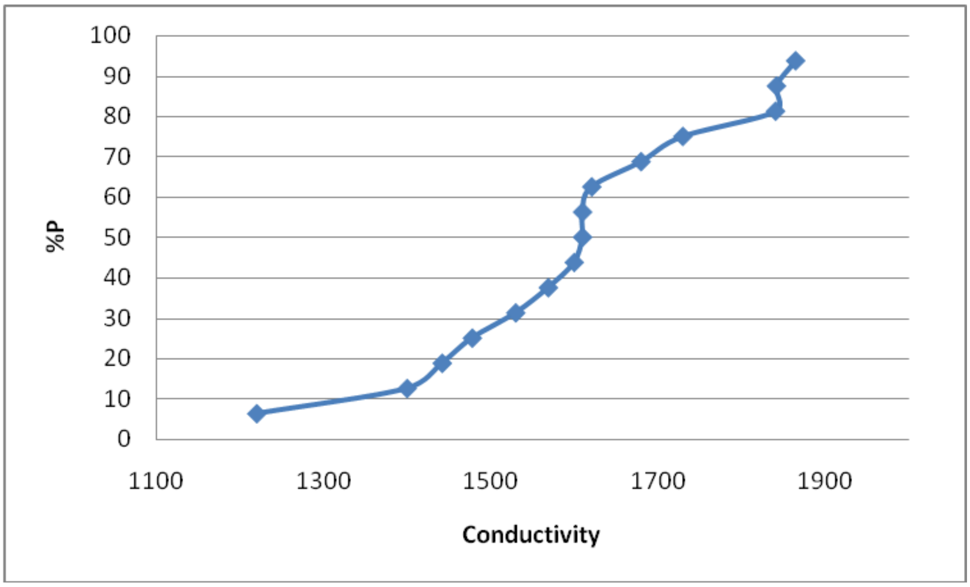

Figure 4.21: Inlet Conductivity

While inlet conductivity value was 1602 mS, both type of reactors decreased conductivity at around 1400 mS (1414 mS for olive type and 1400 mS for pine type) (Table 4.35 and Table 4.36). Total water quality has been raised up 13%. Values are acceptable to reuse water for irrigation.

Table 4.35: Conductivity values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mS)	Average (mS)	Standard Deviation
4	1,5	10	6	990-1817	1471	239
5		5	6	1000-1857	1448	287
1	2,5	10	6	930-1600	1306	235
2		5	6	1100-1680	1431	198

Table 4.36: Conductivity values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mS)	Average (mS)	Standard Deviation
18	1,5	5	3	1130-1789	1450	257
15	2,5	5	3	1000-1734	1341	243
10	1,5	5	6	1090-1844	1533	249
11		5	6	1020-1769	1424	242
12		5	6	1040-1739	1386	233
6		10	6	1000-1713	1437	220
7	2,5	5	6	960-1763	1315	333
8		5	6	1050-1880	1447	362
9		5	6	900-1790	1355	349
3		10	6	1030-1890	1546	309
17	1,5	5	10	905-1820	1422	315
16		10	10	944-1561	1325	234
14	2,5	5	10	1030-1720	1305	263
13		10	10	850-1716	1305	312

Conductivity of water has been greatly reduced as seen in the Table 4.37 after the startup period. Again there is no significant difference in results of both types of reactors.

Table 4.37: Average values of first and last two months

Tank Specification	First Two Months Average (mS) / Removal % (Inlet = 1745)	Last Two Months Average (mS) / Removal % (Inlet = 1366)
Olive	1664 / 5%	1158 / 15%
Pine	1698 / 3%	1167 / 15%

Only Tank 17 has been selected for representative tank for all tanks since average outlet values were almost same for both type of reactors (Figure 4.22).

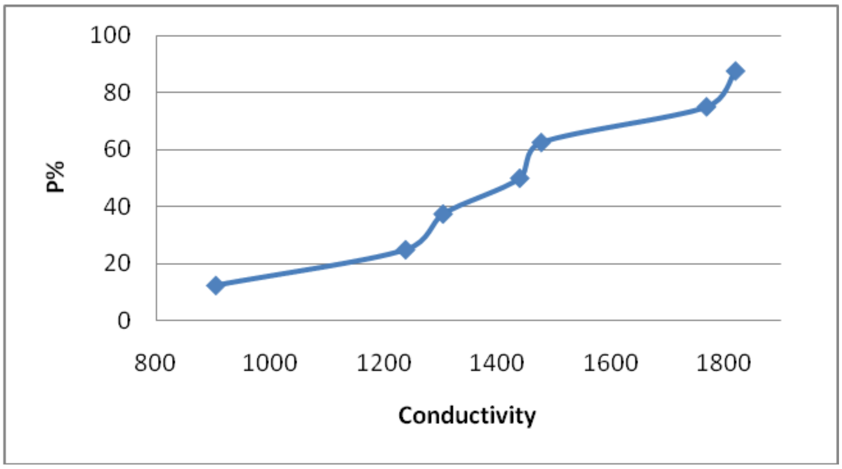

Figure 4.22: Conductivity outlet graphic of Tank 17

4.2.3 COD

15 COD readings of inlet have shown in Figure 4.23.

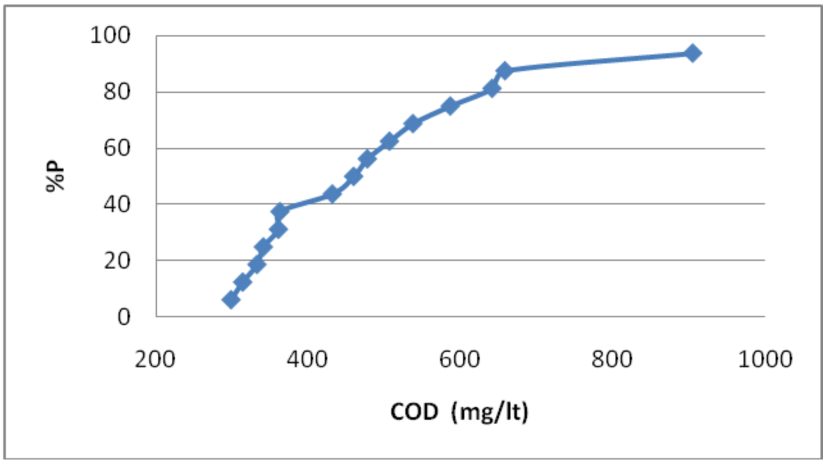

Figure 4.23: COD Inlet

During the study, inlet COD values ranged in 299 – 906 mg/l with an average of 481 mg/lt (Table 4.38 and Table 4.39). It has been noticed that highest COD value for Task 2 was measured 906 mg/lt while it was 682 mg/lt for Task 1. However average value felt down to 481 mg/lt in Task 2. This shows real case effectiveness of anaerobic and facultative lagoons.

Table 4.38: COD values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
4	1,5	10	6	92-1484	617	561
5		5	6	89-1491	625	557
1	2,5	10	6	103-392	252	105
2		5	6	89-359	200	109

Table 4.39: COD values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
18	1,5	5	3	83-830	375	274
15	2,5	5	3	108-894	331	313
10	1,5	5	6	126-694	327	224
11		5	6	129-649	333	192
12		5	6	160-1228	405	340
6		10	6	75-596	287	154
7	2,5	5	6	92-547	233	215
8		5	6	129-492	297	161
9		5	6	89-712	378	259
3		10	6	86-894	400	297
17	1,5	5	10	146-1277	470	421
16		10	10	149-1203	470	442
14	2,5	5	10	83-424	215	122
13		10	10	183-619	319	177

Average values were found as 423 mg/lt for olive type reactors and 346 mg/lt for pine type reactors. As seen in the Table 4.38 reactors that have 2,5m height have given better removal values compared to 1,5m ones. This can be caused of the volume of the 2,5m reactors which is capable of holding more woodchips. Microorganisms on different feedstock, number of experiments and filling of the reactors at different times might be the factors that vary widely and could be significantly effective on the results.

In order to clarify the difference in effectiveness of different types of reactors, removal rates have been compared in Table 4.40.

Table 4.40: COD removal rate comparison

Tank Specification	Average CODt Inlet (mg/l)	Average Effluent CODt (mg/l)	Removal Rate
Olive	481	423	12%
Pine		346	28%

As seen in table pine filled reactors have given noticeably better results than olive filled reactors.

To have more accurate result first and last two months outlet average values also compared. Table 4.41 shows the difference between startup period and last two months period. On the contrary this time olive woodchips filled reactors average is better than the other. Nonetheless difference was not major.

Table 4.41: Average values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 565)	Last Two Months Average(mg/l) / Removal % (Inlet = 324)
Olive	1310 / -131%	145 / 55%
Pine	568 / -1%	161 / 50%

Best performed tanks graph for both systems is down below (Figure 4.24 and Figure 4.25).

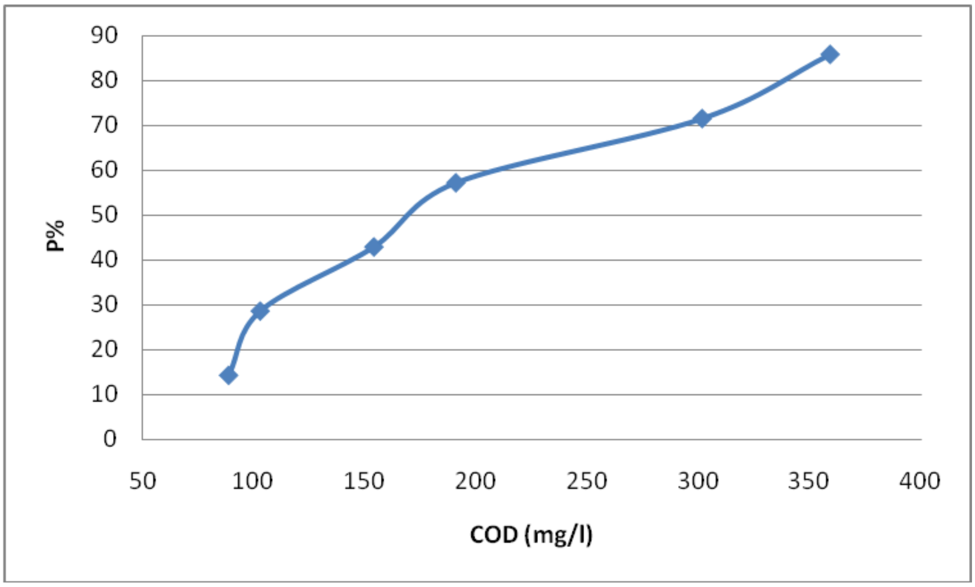

Figure 4.24: COD outlet graph of Tank 2 (Olive)

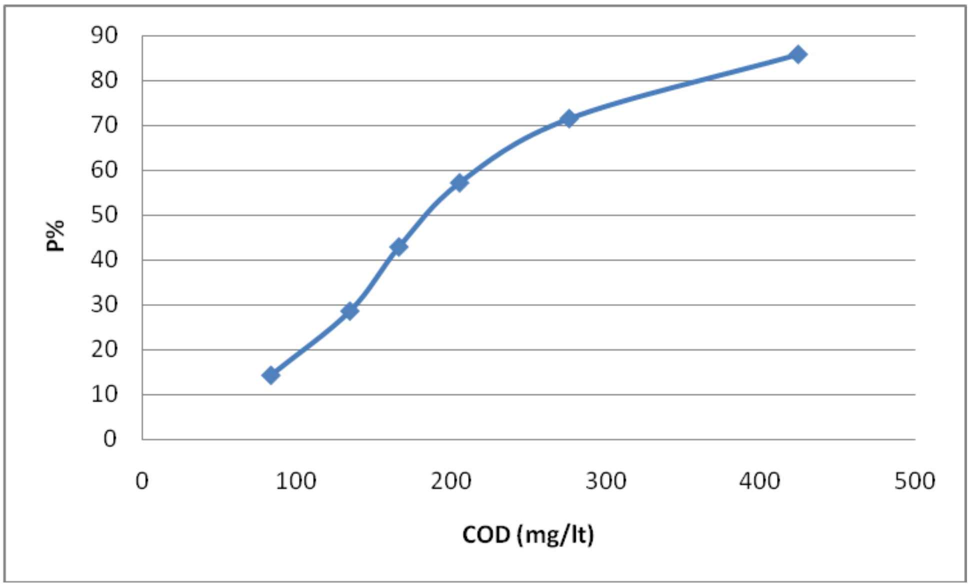

Figure 4.25: COD outlet graph of Tank 14 (Pine)

4.2.4 BOD_5

Inlet BOD value ranged in 170 – 496 mg/l with an average of 325 mg/l (Figure 4.26).

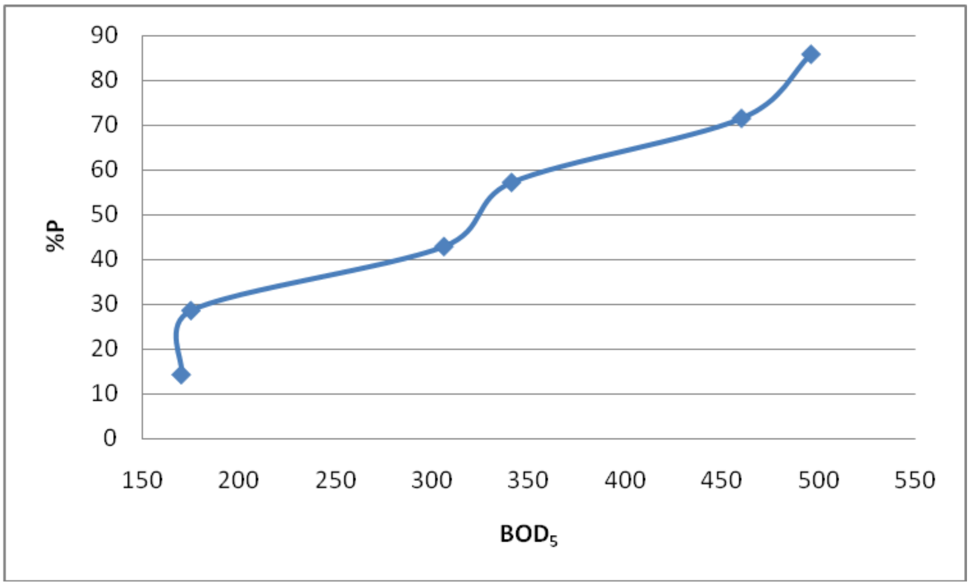

Figure 4.26: Inlet BOD

Average BOD_5 values were obtained as 235 mg/l and 322 mg/l for olive type and pine type reactors respectively. As it is seen on the Table 4.43, lack of experimental data and relatively higher BOD_5 values obtained from tanks 3 and 17 might cause deviation in resultant average values. Therefore, BOD_5 values can be misleading and erroneous.

Table 4.42: BOD values of olive woodchips filled bioreactors

Tank #	**Height Of Biofilter (m)**	**Size Of Wood Chips (cm)**	**RT (Day)**	**Range (mg/lt)**	**Average (mg/lt)**	**Standard Deviation**
4	1,5	10	6	230	230	-
5		5	6	235	235	-
1	2,5	10	6	205-377	291	122
2		5	6	135-235	185	71

Table 4.43: BOD values of pine woodchips filled bioreactors

<table>
<tr><th>Tank #</th><th>Height Of Biofilter (m)</th><th>Size Of Wood Chips (cm)</th><th>RT (Day)</th><th>Range (mg/lt)</th><th>Average (mg/lt)</th><th>Standard Deviation</th></tr>
<tr><td>18</td><td>1,5</td><td>5</td><td>3</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>15</td><td>2,5</td><td>5</td><td>3</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>10</td><td rowspan="4">1,5</td><td>5</td><td>6</td><td>250-460</td><td>374</td><td>110</td></tr>
<tr><td>11</td><td>5</td><td>6</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>12</td><td>5</td><td>6</td><td colspan="2">120</td><td>-</td></tr>
<tr><td>6</td><td>10</td><td>6</td><td colspan="2">170</td><td>-</td></tr>
<tr><td>7</td><td rowspan="4">2,5</td><td>5</td><td>6</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>8</td><td>5</td><td>6</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>9</td><td>5</td><td>6</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>3</td><td>10</td><td>6</td><td colspan="2">480</td><td>-</td></tr>
<tr><td>17</td><td rowspan="2">1,5</td><td>5</td><td>10</td><td>170-520</td><td>470</td><td>247</td></tr>
<tr><td>16</td><td>10</td><td>10</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>14</td><td rowspan="2">2,5</td><td>5</td><td>10</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>13</td><td>10</td><td>10</td><td>-</td><td>-</td><td>-</td></tr>
</table>

Table 4.44 proves that results are erroneous and shows 1.0% for pine type reactors while it is 28% for olive type.

Table 4.44: BOD removal rate comparison

<table>
<tr><th>Tank Specification</th><th>Average BOD Inlet (mg/l)</th><th>Average Effluent BOD (mg/l)</th><th>Removal Rate</th></tr>
<tr><td>Olive</td><td rowspan="2">325</td><td>235</td><td>28%</td></tr>
<tr><td>Pine</td><td>322</td><td>1.0%</td></tr>
</table>

4.2.5 TSS

Inlet TSS value ranged in 63 – 236 mg/l with an average of 127 mg/l (Figure 4.27).

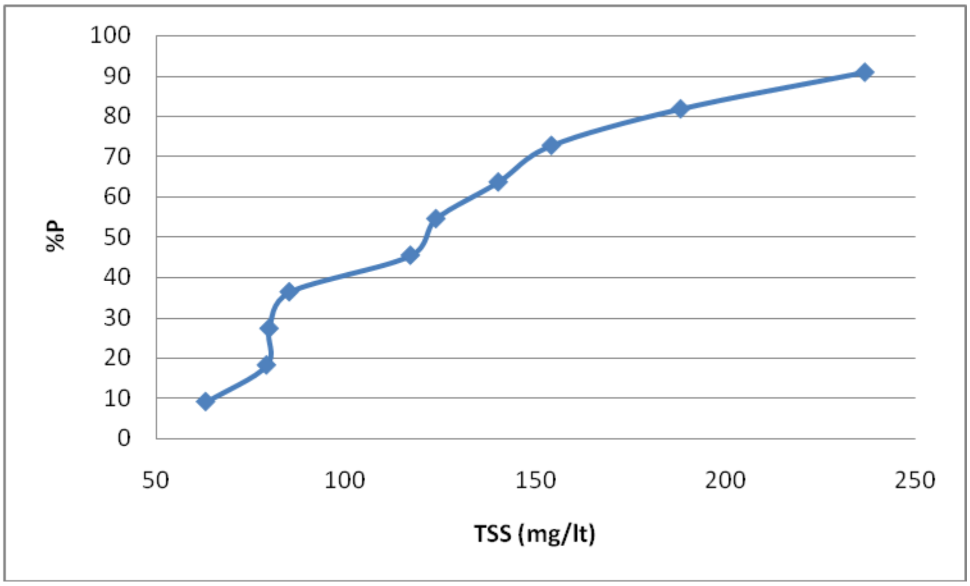

Figure 4.27: Inlet TSS

Average values and specifications are shown in the (Table 4.45 and Table 4.46).

Table 4.45: TSS values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
4	1,5	10	6	18-317	134	117
5		5	6	38-65	47	10
1	2,5	10	6	29-138	83	77
2		5	6	61-90	75	20

Table 4.46: TSS values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
18	1,5	5	3	73-315	194	171
15	2,5	5	3	9	9	-
10	1,5	5	6	38-153	78	45
11		5	6	46	46	0
12		5	6	38-56	48	12
6		10	6	61-120	87	30
7	2,5	5	6	-	-	-
8		5	6	-	-	-
9		5	6	81	81	-
3		10	6	29-240	134	149
17	1,5	5	10	76-346	211	191
16		10	10	46	46	-
14	2,5	5	10	27	27	-
13		10	10	26	26	-

In general perspective, removal efficiencies of olive and pine type reactors occurred as 85 mg/l and 82 mg/l respectively (Table 4.47). At the matter of fact that inlet average TSS was 127 mg/l both reactor types showed good removal efficiencies. Once again, insufficient experimental data should be considered as a substantially effective factor on the results.

Table 4.47: TSS removal rate comparison

Tank Specification	Average TSS Inlet (mg/l)	Average Effluent TSS (mg/l)	Removal Rate
Olive	127	85	33%
Pine		82	35%

At the beginning olive woodchips filled reactors had more suspended solids than pine woodchips filled ones; this can be caused from the structure of olive tree. This situation changed later during the experiment period as seen in the Table 4.48. Olive type reactors performed better in time even though the difference between systems was not significant.

Table 4.48: Average values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 108)	Last Two Months Average(mg/l) / Removal % (Inlet = 112)
Olive	133 / -23%	62 / 45%
Pine	107 / 1%	70 / 37%

Tank 10's graph is down below to represent other tanks (Figure 4.28).

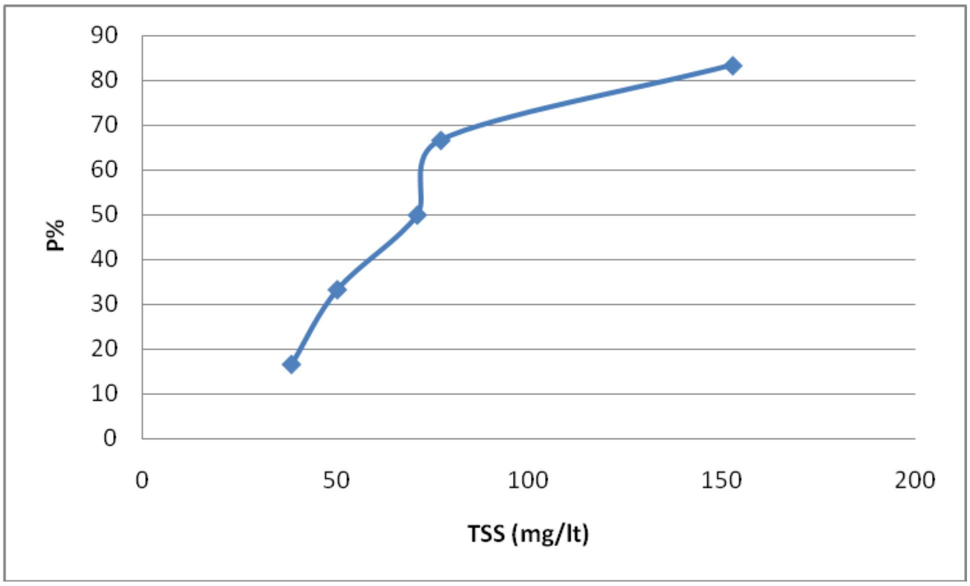

Figure 4.28: TSS outlet graph of Tank 10

4.2.6 VSS

During the study experiments VSS value ranged in 58-224 mg/l with an average of 109 mg/l (Figure 4.29).

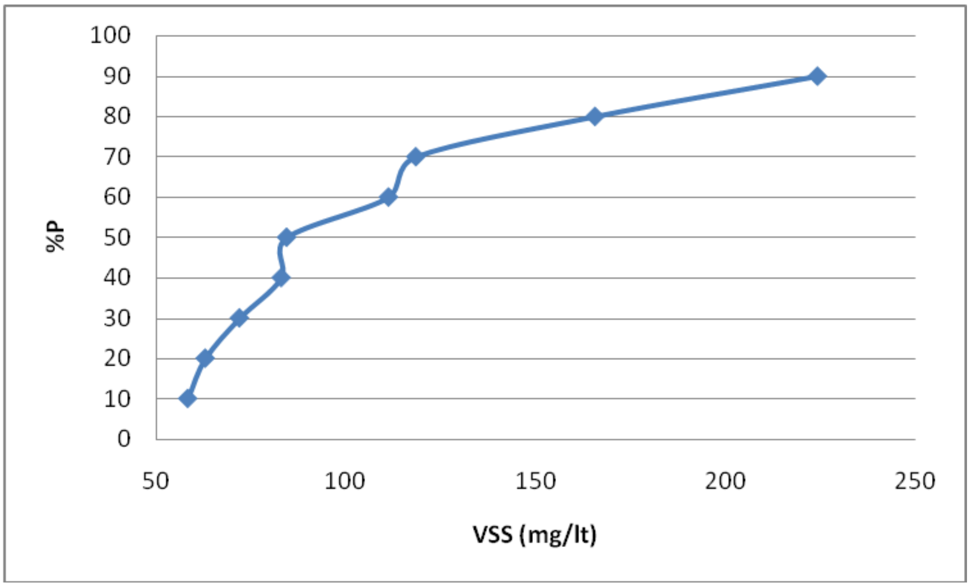

Figure 4.29: Inlet VSS

Average values and specifications are shown in the (Table 4.49).

Table 4.49: VSS values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
4	1,5	10	6	18-277	120	138
5		5	6	38-65	52	19
1	2,5	10	6	120	120	-
2		5	6	59	59	-

Table 4.50: VSS values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
18	1,5	5	3	30-161	96	92
15	2,5	5	3	9	9	-
10	1,5	5	6	38-140	76	45
11		5	6	-	-	-
12		5	6	35-68	49	17
6		10	6	81	81	-
7	2,5	5	6	44	44	-
8		5	6	-	-	-
9		5	6	79-197	153	64
3		10	6	29-301	179	138
17	1,5	5	10	37-278	124	134
16		10	10	-	-	-
14	2,5	5	10	27	27	-
13		10	10	26	26	-

Compared to average TSS value which was 127 mg/lt, volatile part (VSS) was found 109 mg/lt which is 86% of TSS. It is clearly seen that most of the TSS was volatile when we compare TSS versus VSS data, from Table 4.45 to Table 4.51. In this respect, olive and pine type reactors were investigated. Pine type reactors achieved 27% removal efficiency while it is 19% for olive type (Table 4.51).

Table 4.51: VSS removal rate comparison

Tank Specification	Average VSS Inlet (mg/l)	Average Effluent VSS (mg/l)	Removal Rate
Olive	109	88	19%
Pine		79	27%

Even though experimental data was not enough, pine type reactors efficiency is better than olive type reactors at the last two months (Table 4.52).

Table 4.52: Average VSS values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 96)	Last Two Months Average(mg/l) / Removal % (Inlet = 119)
Olive	277 / -189%	61 / 49%
Pine	180 / -87%	41 / 66%

4.2.7 NH_4^+

Minimum value was 14 mg/lt and maximum value was 88 mg/lt with an average of 56 mg/lt during the study. Most of the inlet values were in a good agreement at around 60 mg/lt (Figure 4.30).

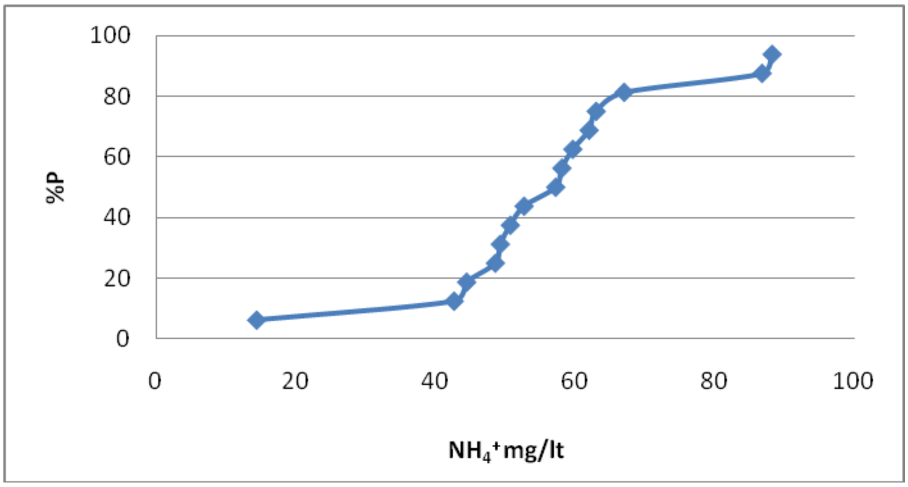

Figure 4.30: Inlet NH_4^+

Average values and specifications are shown in the (Table 4.53 and Table 4.54).

Table 4.53: NH_4^+ values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
4	1,5	10	6	16-82	44	21
5		5	6	1-51	29	20
1	2,5	10	6	4-64	21	22
2		5	6	23-68	44	16

Table 4.54: NH_4^+ values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
18	1,5	5	3	1-63	33	25
15	2,5	5	3	0-56	23	21
10	1,5	5	6	7-61	35	19
11		5	6	0-58	33	24
12		5	6	0-58	26	25
6		10	6	8-76	37	23
7	2,5	5	6	9-48	23	19
8		5	6	13-50	31	18
9		5	6	0-45	16	19
3		10	6	1-74	39	25
17	1,5	5	10	0-58	28	25
16		10	10	0-54	29	22
14	2,5	5	10	1-86	25	34
13		10	10	1-80	36	32

Average values were 34 mg/l for olive woodchips filled reactors and 30 mg/l for pine woodchips filled reactors. Average outlet of both systems was 32 mg/l which is almost half of the average inlet value 60 mg/l. As shown in the table below (Table 4.55) even there is no major efficiency difference between two systems, pine filled systems have given better results.

Table 4.55: NH_4^+ removal rate comparison

Tank Specification	Average NH_4^+ Inlet (mg/l)	Average Effluent NH_4^+ (mg/l)	Removal Rate
Olive	60	34	43%
Pine		30	50%

When Table 4.56 is checked, almost same percantage of difference was seen. As expected last two months values 50% better than the beginning period.

Table 4.56: Average NH_4^+ values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 50)	Last Two Months Average(mg/l) / Removal % (Inlet = 51)
Olive	48 / 4%	20 / 61%
Pine	47 / 6%	18 / 65%

Outlet graph of Tank 1 and Tank 15 is down below (Figure 4.31 and Figure 4.32).

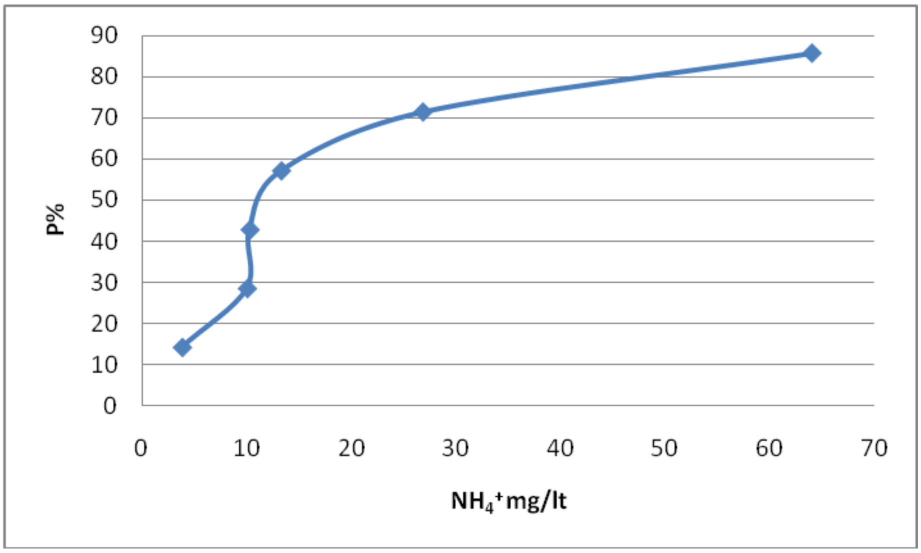

Figure 4.31: Outlet NH_4^+ graph of Tank 1

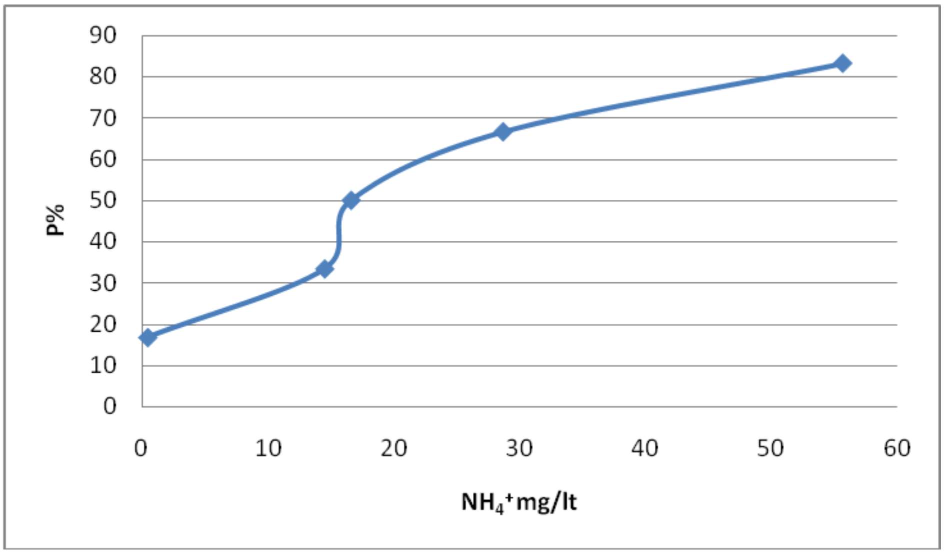

Figure 4.32: Outlet NH_4^+ graph of Tank 15

4.2.8 PO_4^{-3}

12 PO_4^{-3} readings of inlet have shown in Figure 4.33. During the study, inlet PO_4^{-3} values ranged in 3.4 – 22 mg/l with an average of 14 mg/l.

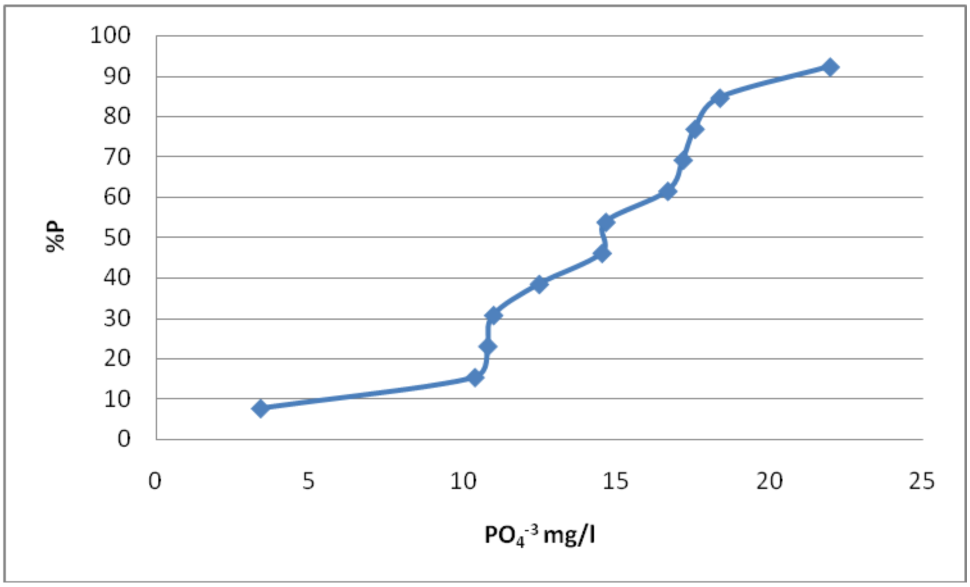

Figure 4.33: Inlet PO_4^{-3}

Average values and specifications are shown in the (Table 4.57 and Table 4.58).

Table 4.57: PO_4^{-3} values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
4	1,5	10	6	5-36	15	9
5		5	6	6-38	13	9
1	2,5	10	6	3-12	8	4
2		5	6	3-14	8	4

Table 4.58: PO_4^{-3} values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
18	1,5	5	3	9-44	19	12
15	2,5	5	3	10-30	17	8
10	1,5	5	6	7-30	15	7
11		5	6	10-18	14	3
12		5	6	6-35	14	9
6		10	6	5-35	13	9
7	2,5	5	6	8-14	11	3
8		5	6	10-13	11	1
9		5	6	8-26	14	6
3		10	6	10-24	15	5
17	1,5	5	10	4-36	15	10
16		10	10	8-25	16	7
14	2,5	5	10	7-25	14	7
13		10	10	5-18	10	4

Average values were 11 mg/l for olive woodchips filled reactors and 14 mg/l for pine woodchips filled reactors. Average value for pine type reactors may be misleading but it should be noted that these values also includes startup period values. High values influenced general average for pine type reactors as seen in Table 4.59.

Table 4.59: PO_4^{-3} removal rate comparison

Tank Specification	Average PO_4^{-3} Inlet (mg/l)	Average Effluent PO_4^{-3} (mg/l)	Removal Rate
Olive	14	11	21%
Pine		14	0%

It was clear that removing efficiency of both systems were equal considering Table 4.60 which shows the clear difference between the beginning and last two months period.

Table 4.60: Average PO_4^{-3} values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 17)	Last Two Months Average(mg/l) / Removal % (Inlet = 13)
Olive	13 / 23%	10 / 23%
Pine	14 / 18%	11 / 16%

Tank 2 and Tank 11's outlet PO_4^{-3} is down below (Figure 4.34 and Figure 4.35).

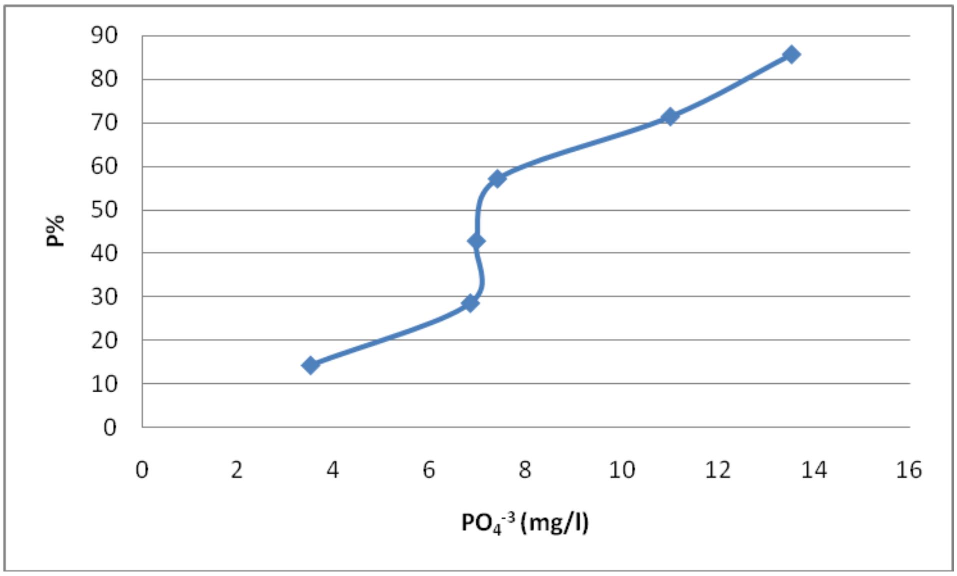

Figure 4.34: Outlet PO_4^{-3} graph of Tank 2

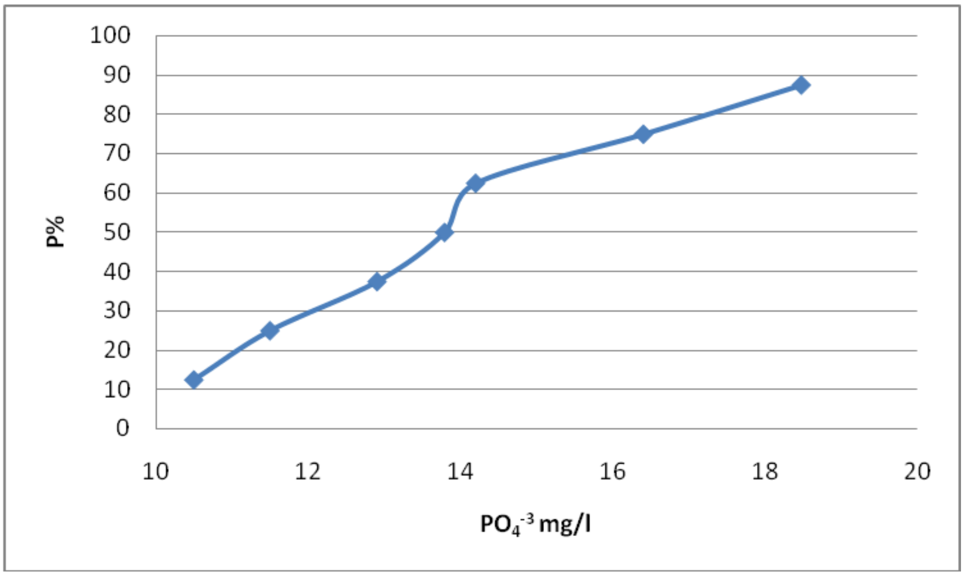

Figure 4.35: Outlet PO_4^{-3} graph of Tank 11

4.2.9 TKN

Minimum value was 46 mg/l and maximum value was 60 mg/l with an average of 52 mg/l during the study (Figure 4.36).

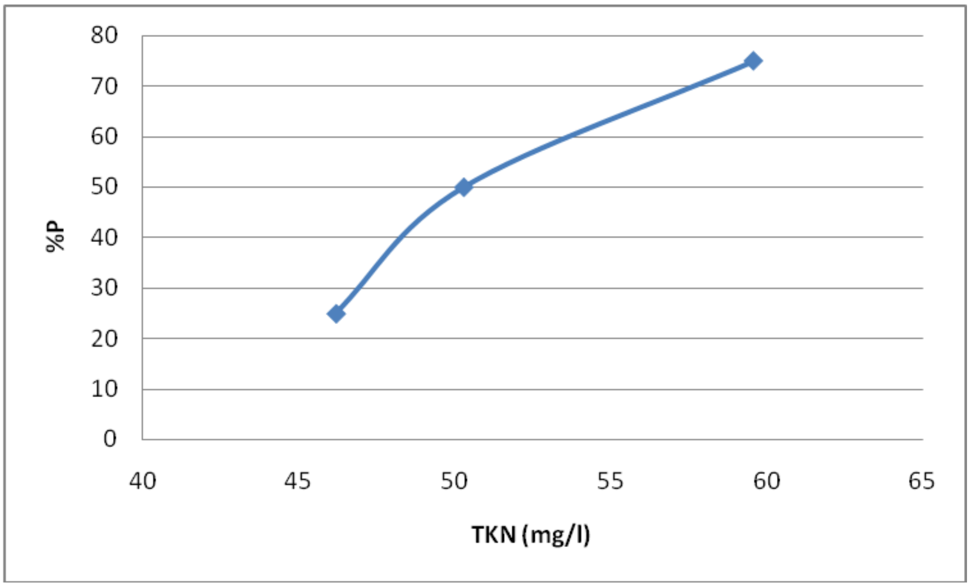

Figure 4.36: Inlet TKN

Due to availability of laboratory conditions number of experiments was maximum two. Hence values were not enough to make a complete comparison between two systems. However average value was 42 mg/l which is lower than inlet average value 60 mg/l.

Average values and specifications are shown in the (Table 4.61 and Table 4.62).

Table 4.61: TKN values of olive woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
4	1,5	10	6	32-42	34	7
5		5	6	41-50	45	7
1	2,5	10	6	44	44	-
2		5	6	-	-	-

Table 4.62: TKN values of pine woodchips filled bioreactors

Tank #	Height Of Biofilter (m)	Size Of Wood Chips (cm)	RT (Day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
18	1,5	5	3	-	-	-
15	2,5	5	3	-	-	-
10	1,5	5	6	59	59	-
11		5	6	-	-	-
12		5	6	-	-	-
6		10	6	46-65	55	13
7	2,5	5	6	-	-	-
8		5	6	-	-	-
9		5	6	-	-	-
3		10	6	-	-	-
17	1,5	5	10	4-36	15	-
16		10	10	-	-	-
14	2,5	5	10	-	-	-
13		10	10	-	-	-

Table 4.63: TKN removal rate comparison

Tank Specification	Average TKN Inlet (mg/l)	Average Effluent TKN (mg/l)	Removal Rate
Olive	52	41	21%
Pine		43	17%

4.3 Effectivness of Experimental Designs on TASK 3 (Seasonal Reservoir) Parameters

4.3.1 pH

37 pH readings of inlet have shown in Figure 4.37. During the study, inlet pH values ranged in 6 – 8 mg/l with an average of 7.

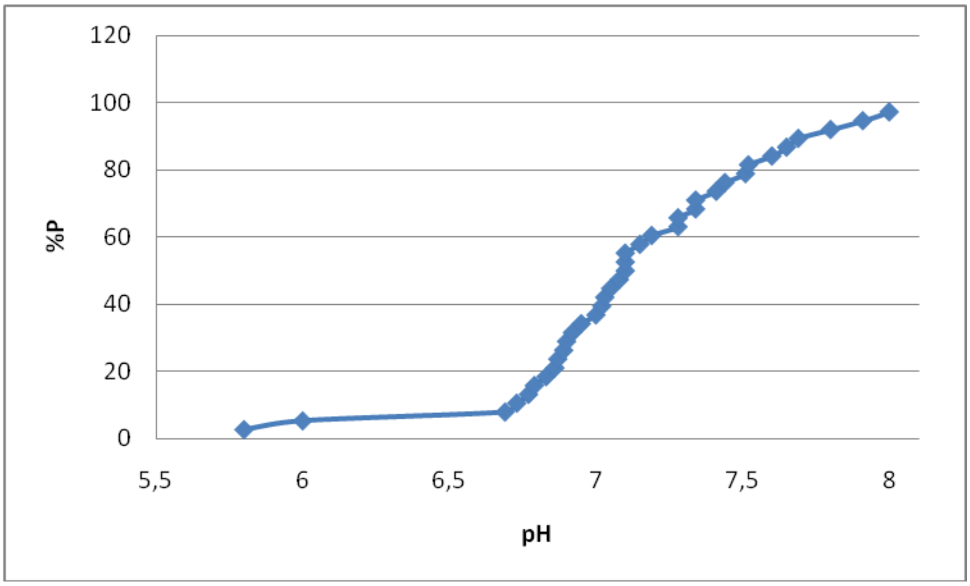

Figure 4.37: Inlet pH

All pH values were close to each other (Table 4.64). Even it was in the acceptable range, average outlet pH from all tanks was 8,3. Outlet pH values were slightly higher than inlet pH which can be omitted.

Table 4.64: pH values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	Retention Time (day)	Range	Average	Standard Deviation
1	Rectangular	-	-	**30**	6,7-9	8,2	0,57
5	Hexagonal		-	**30**	7,1-9	8,3	0,46
2	Rectangular		+	**30**	6,1-8,7	7,8	0,61
6	Hexagonal		+	**30**	7,1-9,3	8,4	0,60
3	Rectangular	+	-	**30**	7,3-9,2	8,7	0,53
4	Rectangular		+	**30**	7,4-9	8,4	0,40

Raise in pH values could be also observed from Table 4.65 in last two months. But the difference was not significant and could be omitted. Tank 4's graph is down below to represent other tanks (Figure 4.38).

Table 4.65: Average pH values of first and last two months

Outlet average for all 6 reactors	First Two Months Average (Inlet = 6,9)	Last Two Months Average (Inlet = 7,6)
	7,7	8,3

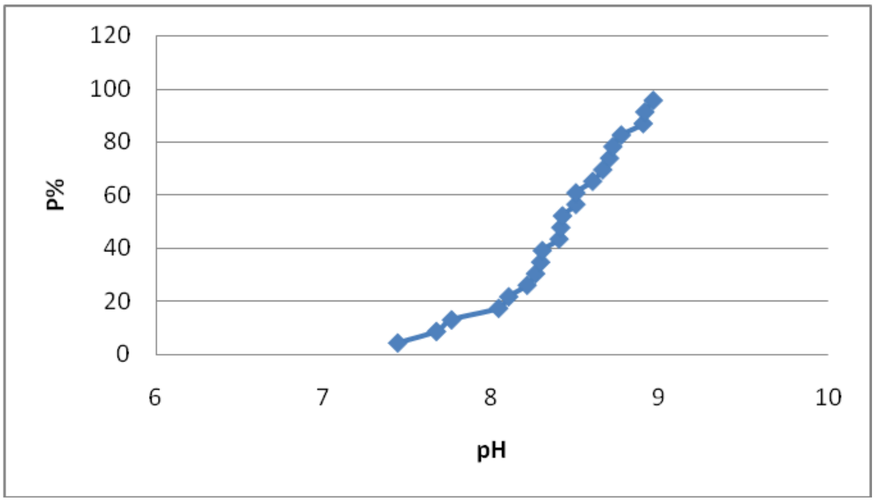

Figure 4.38: Outlet pH graph of Tank 4

4.3.2 Conductivity

36 Conductivity readings of inlet have shown in Figure 4.39. Conductivities of inlet water were ranged in 1230-1900 mS with an average of 1644 mS.

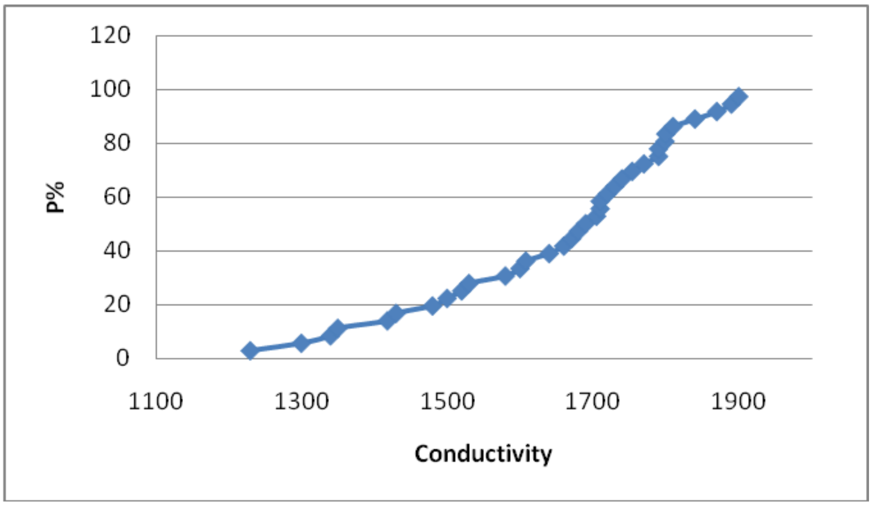

Figure 4.39: Inlet Conductivity

As seen in the Table 4.66 there is no increase in efficiency in conductivity removal rate. General outlet average value for all reactors was 1570mS which is 7% better than average inlet value 1644mS. However this values includes startup period values. Conductivity values were decreased by time to 1258mS overall as seen in the table below. It should be noted that, at the same time inlet average was 1352. Hence removal efficiency was same.

Table 4.66: Conductivity values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mS)	Average (mS)	Standard Deviation
1	Rectangular	-	-	**30**	1107-1870	1573	186
5	Hexagonal		-	**30**	1055-1770	1576	176
2	Rectangular		+	**30**	1109-1830	1659	231
6	Hexagonal		+	**30**	1160-1770	1507	186
3	Rectangular	+	-	**30**	1220-1675	1508	143
4	Rectangular		+	**30**	1190-1790	1600	157

Table 4.67: Average Conductivity values of first and last two months

Outlet average for all 6 reactors	**First Two Months Average(mS)/ Removal % (Inlet = 1757)**	**Last Two Months Average(mS) / Removal % (Inlet = 1352)**
	1648 / 6%	1258 / 7%

To represent all tanks Tank 4's graph is given (Figure 4.40).

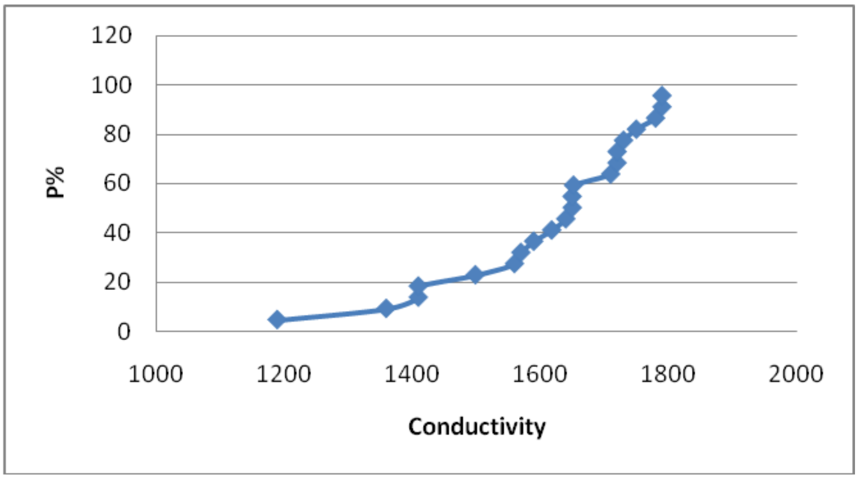

Figure 4.40: Outlet conductivity graph of Tank 4

4.3.3 COD

37 COD readings of inlet have shown in Figure 4.41. COD value of inlet wastewater to Task 3 changed in the range of 200 - 809 mg/lt with an average of 446 mg/lt.

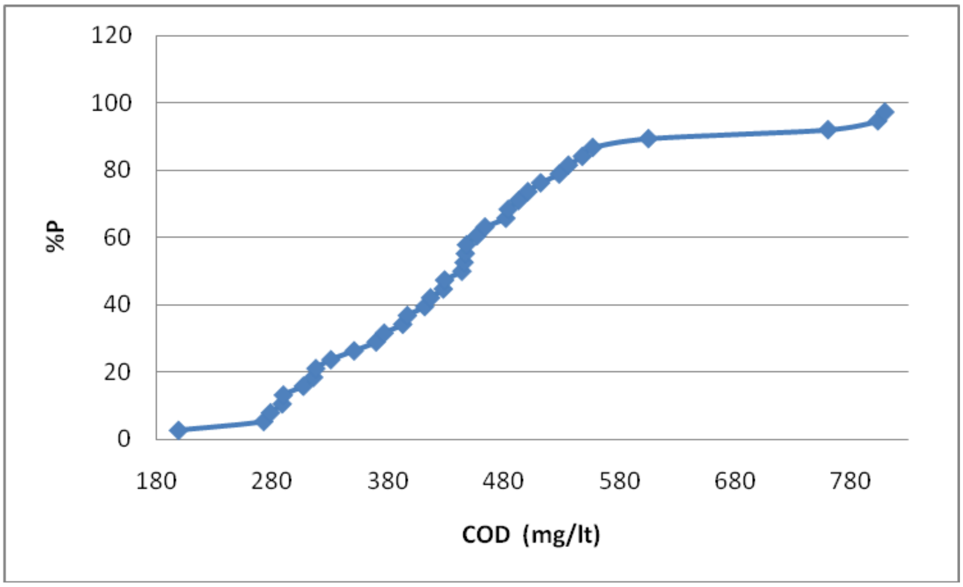

Figure 4.41: Inlet COD

Removal rate for all tanks were 44 % considering average inlet and outlet value which is 446 mg/l and 248 mg/l respectively. As Table 4.68 shows tanks that have fixed medium (Tank 3 and 4) have given better results. Also applying aeration noticeably increased COD removing performance of Tank 4. Tanks 3's removal rate was 51% and Tank 4's was 62%. At the same time it was obvious that rectangular tanks performs better than hexagonal shaped reactors.

Table 4.68: COD values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	61-690	247	179
5	Hexagonal		-	**30**	133-590	308	129
2	Rectangular		+	**30**	78-748	252	174
6	Hexagonal		+	**30**	120-945	295	191
3	Rectangular	+	-	**30**	112-615	219	130
4	Rectangular		+	**30**	39-469	169	117

Table 4.69 shows the clear difference between the startup period and the last two months. Overall system performance greatly increased.

Table 4.69: Average COD values of first and last two months

Outlet average for all 6 reactors	First Two Months Average(mg/l)/ Removal % (Inlet = 510)	Last Two Months Average(mg/l) / Removal % (Inlet = 303)
	449 / 12%	137 / 55%

Tank 4's performance was the best, in this manner it is selected as representative.

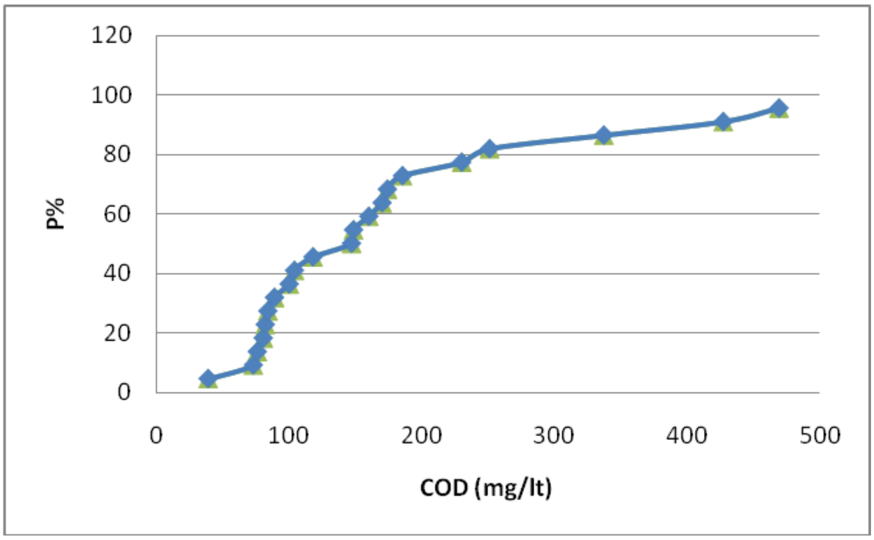

Figure 4.42 Outlet COD graph of Tank 4

4.3.4 BOD_5

Inlet BOD value ranged in 250 – 440 mg/l with an average of 384 mg/l Figure 4.43).

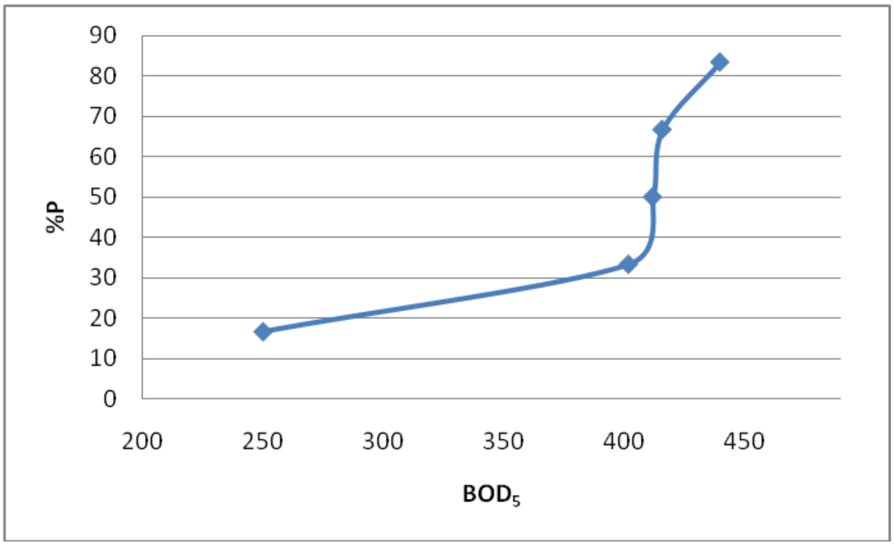

Figure 4.43: Inlet BOD

As the other tasks BOD_5 values were not enough due to lab availability. Maximum number of experiments for tanks was 3 and for inlet value was 5. Lack of experiment number has affected average values. However it was clear that reactor performances were very good as seen in the Table 4.70. Overall Task performance in terms of removal rate was 71%.

As in the COD removal Tank 4 was better than the other tanks. That shows airflow provides the oxygen needed to maintain aerobic conditions.

Table 4.70: BOD values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	30-50	79	43
5	Hexagonal		-	**30**	90-200	145	78
2	Rectangular		+	**30**	50-238	152	67
6	Hexagonal		+	**30**	50-250	118	114
3	Rectangular	+	-	**30**	30-357	164	148
4	Rectangular		+	**30**	15-30	22	11

4.3.5 TSS

During the study experiments TSS value ranged in 47-310 mg/l with an average of 105 mg/l (Figure 4.44).

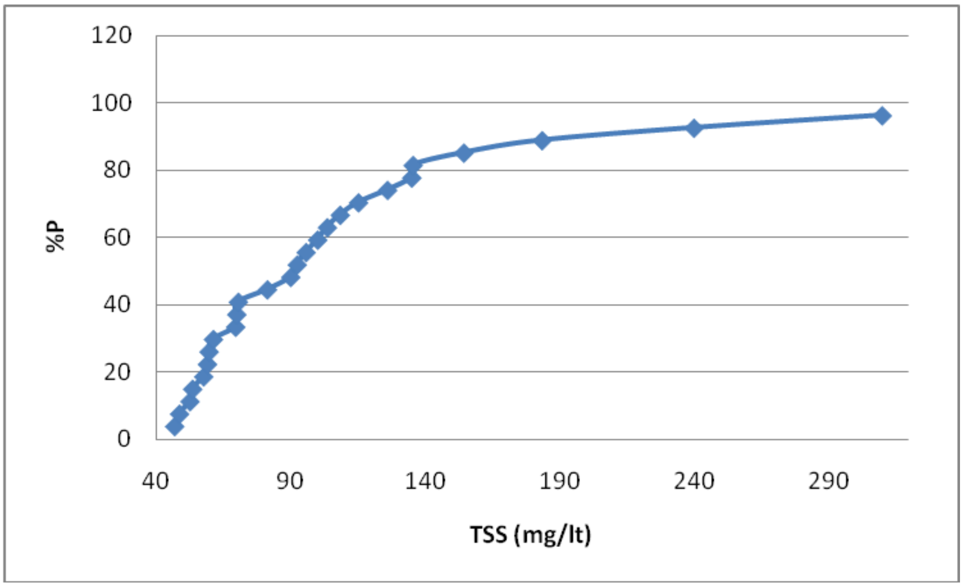

Figure 4.44: Inlet TSS

It seems hexagonal tanks has no effect on TSS removal according to the Table 4.71 while rectangular tanks performs better. But if Table 4.72 checked which shows the latest situation, overall system performance increased by time as expected. Especially Tank 4 which has fixed medium and aeration equipment both achieved best removing rate.

Table 4.71: TSS values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	7-405	80	98
5	Hexagonal		-	**30**	5-306	101	88
2	Rectangular		+	**30**	5-238	65	59
6	Hexagonal		+	**30**	5-329	105	90
3	Rectangular	+	-	**30**	15-267	67	67
4	Rectangular		+	**30**	10-77	30	19

Table 4.72: Average TSS values of first and last two months

Outlet average for all 6 reactors	**First Two Months Average(mg/l)/ Removal % (Inlet = 91)**	**Last Two Months Average(mg/l) / Removal % (Inlet = 117)**
	145 / -59%	18 / 85%

Best performed Tank 4's outlet graph is down below (Figure 4.45).

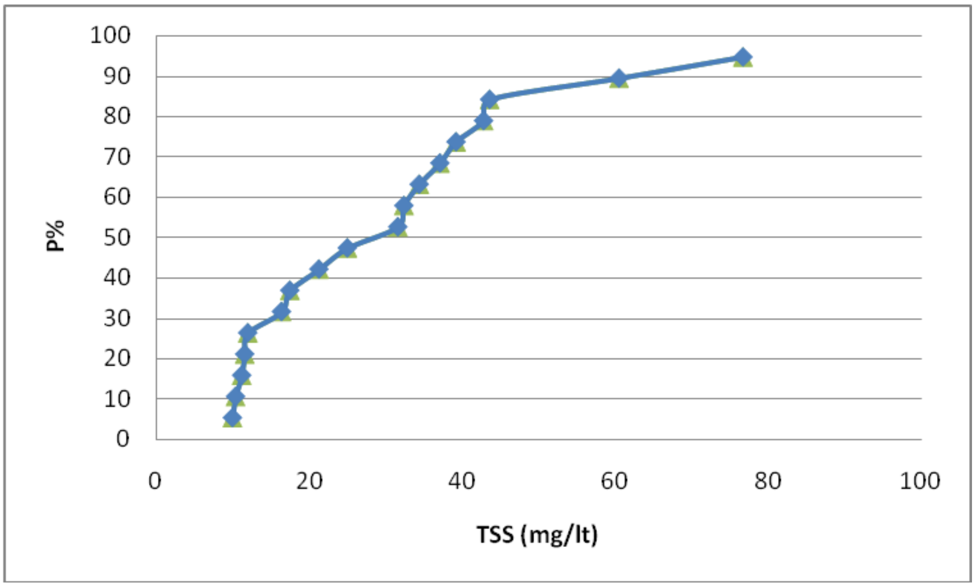

Figure 4.45: Outlet TSS graph of Tank 4

4.3.6 VSS

Inlet VSS value ranged in 13 – 310 mg/l with an average of 96 mg/l (Figure 4.46).

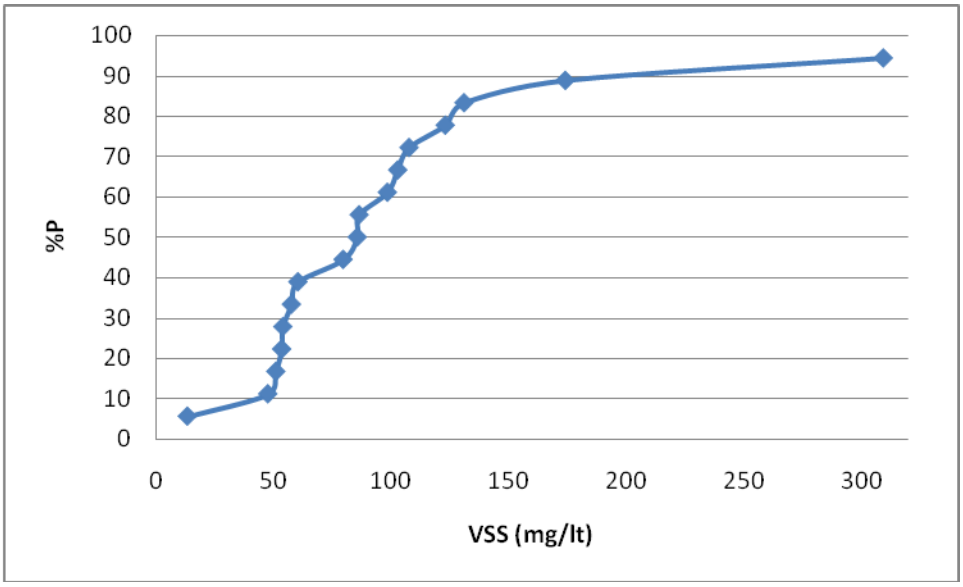

Figure 4.46: Inlet VSS

Inlet average value 96 mg/l shows that 91% of the water consist of volatile part considering inlet average TSS value was 105 mg/l. As seen in Table 4.73 rectangular tanks performed better than hexagonal tanks, while Tank 4 again achieved the best result. Also Tank 3 has a good efficiency which also has fixed medium equipment like Tank 4.

Table 4.73: VSS values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	7-396	82	104
5	Hexagonal		-	**30**	8-300	101	86
2	Rectangular		+	**30**	6-199	65	69
6	Hexagonal		+	**30**	20-256	99	90
3	Rectangular	+	-	**30**	16-122	47	67
4	Rectangular		+	**30**	12-60	26	19

Table 4.74: Average VSS values of first and last two months

Outlet average for all 6 reactors	First Two Months Average(mg/l)/ Removal % (Inlet = 95)	Last Two Months Average(mg/l) / Removal % (Inlet = 65)
	211 / -122%	41 / 37%

Tank 4's oulet VSS graph is selected (Figure 4.47).

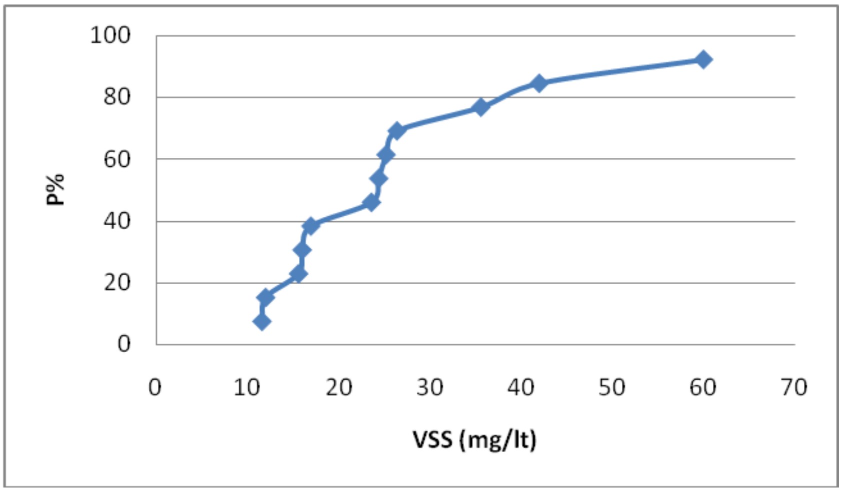

Figure 4.47: Outlet TSS graph of Tank 2

4.3.7 NH_4^+

Inlet NH_4^+ values ranged in 24 – 84 mg/l with an average of 57 mg/l (Figure 4.48).

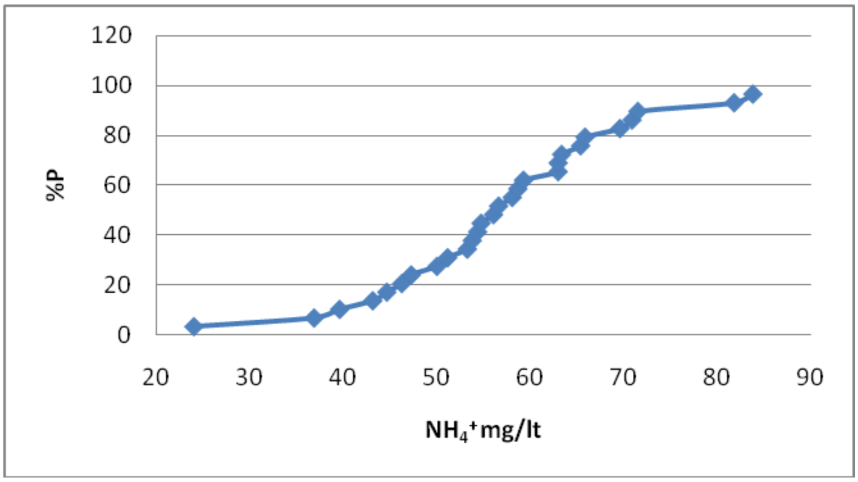

Figure 4.48: Inlet NH_4^+

Average NH_4^+ outlet value was 17 mg/l which is 70% better than inlet value 57 mg/l. All reactors performance was good in terms of NH_4^+ treatment. Even in the first period there was NH_4^+ treatment as seen in Table 4.76, and the performance at last two months was almost 70 %. There were no significant performance differences between tanks.

Table 4.75: NH_4^+ values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	0-56	17	21
5	Hexagonal		-	**30**	2-51	22	16
2	Rectangular		+	**30**	0-65	23	22
6	Hexagonal		+	**30**	0-51	14	16
3	Rectangular	+	-	**30**	1-56	15	18
4	Rectangular		+	**30**	0-56	15	20

Table 4.76: Average NH_4^+ values of first and last two months

Outlet average for all 6 reactors	**First Two Months Average(mg/l)/ Removal % (Inlet = 57)**	**Last Two Months Average(mg/l) / Removal % (Inlet = 63)**
	48 / 16%	20 / 68%

Tank 4 and Tank 3 is selected due to their performance. Since their performance was the same only Tank 3's the outlet graph for NH_4^+ values is given down below (Figure 4.49).

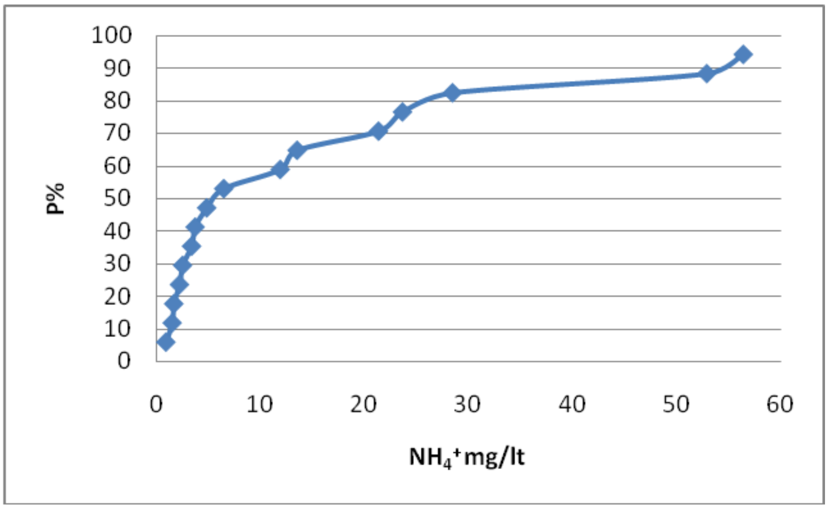

Figure 4.49: Outlet NH_4^+ graph of Tank 3

4.3.8 PO_4^{-3}

Minimum value was 1 mg/lt and maximum value was 30 mg/lt with an average of 12 mg/lt during the study (Figure 4.50).

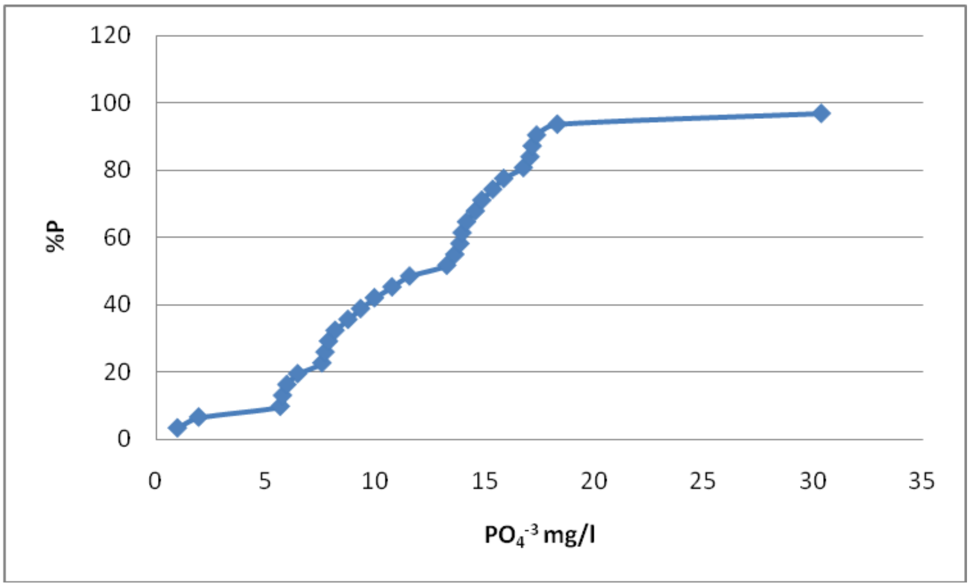

Figure 4.50: Inlet PO_4^{-3}

Average outlet value was 7 mg/l including startup period. The overall performance of the reactors were 42% which is very good. At the same time last two months results were even better with a removal efficiency of 75% (Table 4.78). All reactors performances were almost equal (Table 4.77).

Table 4.77: PO_4^{-3} values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	0-26	7	7
5	Hexagonal		-	**30**	2-38	8	9
2	Rectangular		+	**30**	2-33	8	7
6	Hexagonal		+	**30**	1-16	6	4
3	Rectangular	+	-	**30**	0-50	7	11
4	Rectangular		+	**30**	1-25	7	7

Table 4.78: Average PO_4^{-3} values of first and last two months

Outlet average for all 6 reactors	First Two Months Average(mg/l)/ Removal % (Inlet = 16)	Last Two Months Average(mg/l) / Removal % (Inlet = 12)
	13 / 19%	3 / 75%

Tank 4's outlet graph is down below to represent all reactors (Figure 4.51) .

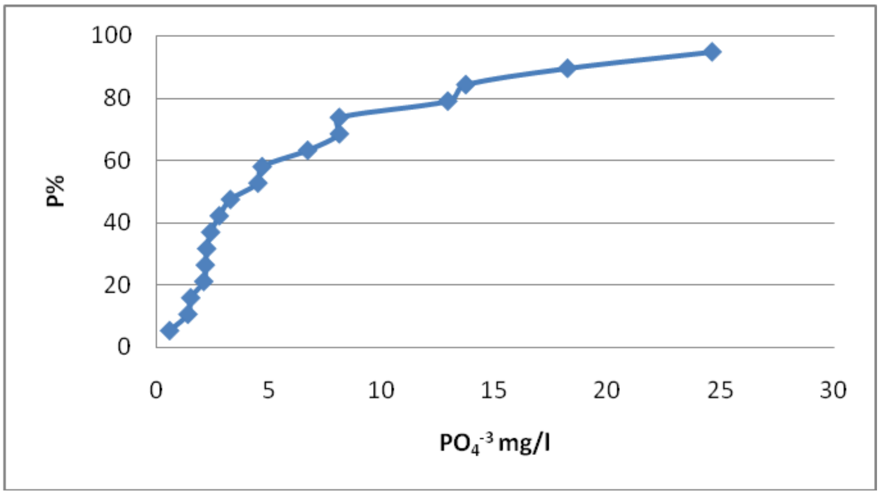

Figure 4.51: Outlet PO_4^{-3} graph of Tank 4

4.3.9 TKN

Minimum value was 43 mg/l and maximum value was 77 mg/l with an average of 62 mg/l during the study (Figure 4.52).

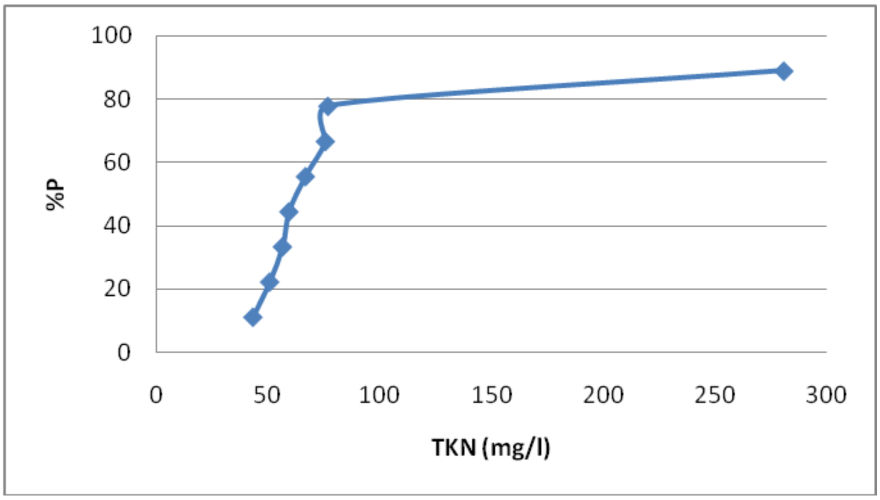

Figure 4.52: Inlet TKN

Even though number of experiments was not exceeded 4, results showed 45% removal efficiency as seen in the Table 4.79. Tanks 3's average value seems lower than others but it should be noted that number of experiments was not enough. Tank 6's TKN value only measured once for instance.

Table 4.79: TKN values of Task 3 reactors

Reactor Number	Reactor Type	Fixed Medium	Aeration	RT (day)	Range (mg/lt)	Average (mg/lt)	Standard Deviation
1	Rectangular	-	-	**30**	26-52	39	18
5	Hexagonal		-	**30**	16-58	32	23
2	Rectangular		+	**30**	12-43	23	18
6	Hexagonal		+	**30**	49	49	-
3	Rectangular	+	-	**30**	7-46	20	23
4	Rectangular		+	**30**	38-42	40	3

4.4 Effectivness of Experimental Designs on TASK 4 (Wetland) Parameters

4.4.1 pH

34 pH readings of inlet have shown in Figure 4.53. During the study, inlet pH values ranged in 7.2 – 8.3 mg/l with an average of 7.8.

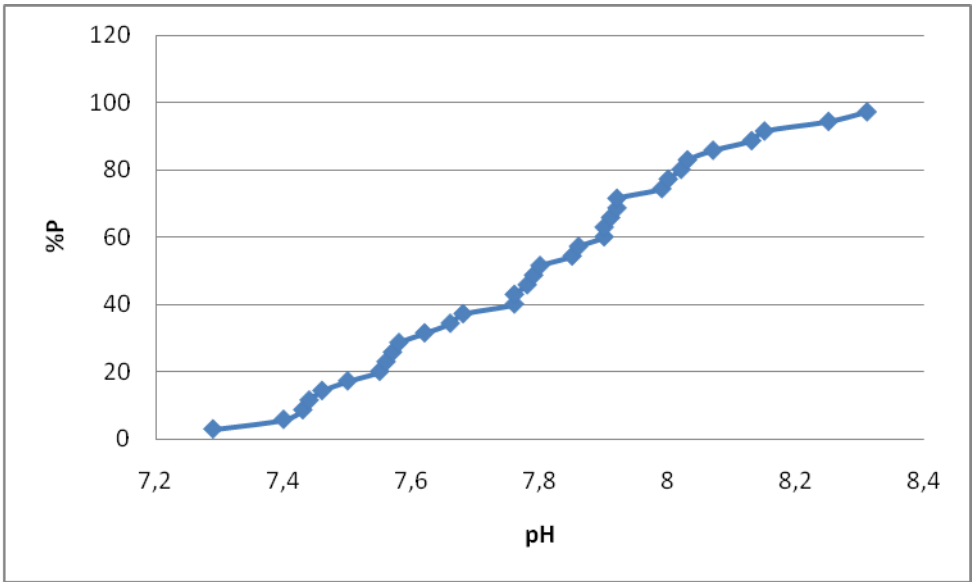

Figure 4.53: Inlet pH

As shown in Figure 3.28 and Figure 3.29 there were two different way filled reactors for Task 4. Average values and specifications are shown in the (Table 4.80 and Table 4.81).

Table 4.80: pH values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
8	Sugar Cane	3	-	7-8	7,3	0,30
7	Sugar Cane		+	7,3-8,1	7,6	0,26
2	Cane	5	-	7,3-7,7	7,5	0,14
4	Reed		-	7,5-8	7,6	0,17
6	Sugar Cane		-	6,9-8	7,4	0,35
1	Cane	5	+	7,2-8	7,5	0,25
3	Reed		+	7,4-7,9	7,6	0,20
9	Reed/Cane		+	7,1-8,1	7,6	0,34
5	Sugar Cane		+	7-8,1	7,6	0,41

Table 4.81: pH values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	7,1-7,9	7,5	0,25
16	Sugar Cane		+	7,1-8,3	7,6	0,38
11	Cane	5	-	7,4-8,1	7,7	0,16
13	Reed		-	6,9-7,7	7,3	0,28
15	Sugar Cane		-	7,1-8,3	7,5	0,39
10	Cane	5	+	7,1-8,1	7,6	0,37
12	Reed		+	6,9-8	7,5	0,39
18	Reed/Cane		+	7,2-7,9	7,6	0,21
14	Sugar Cane		+	7,3-8,2	7,6	0,32

Average pH values for both Type I and Type II systems were same, showed that pH was not connected to material used and retention time. Also as seen in the Table 4.82 in last two months pH was same with inlet value like in the first two months period.

Table 4.82: Average pH values of first and last two months

Tank Specification	First Two Months Average (Inlet = 7,6)	Last Two Months Average (Inlet = 7,3)
Type I	7,5	7,4
Type II	7,6	7,3

As an example, Tank 11 is shown in Figure 4.54 for both type of systems since their pH values were equal and Tank 11's pH closest to the overall average of pH values.

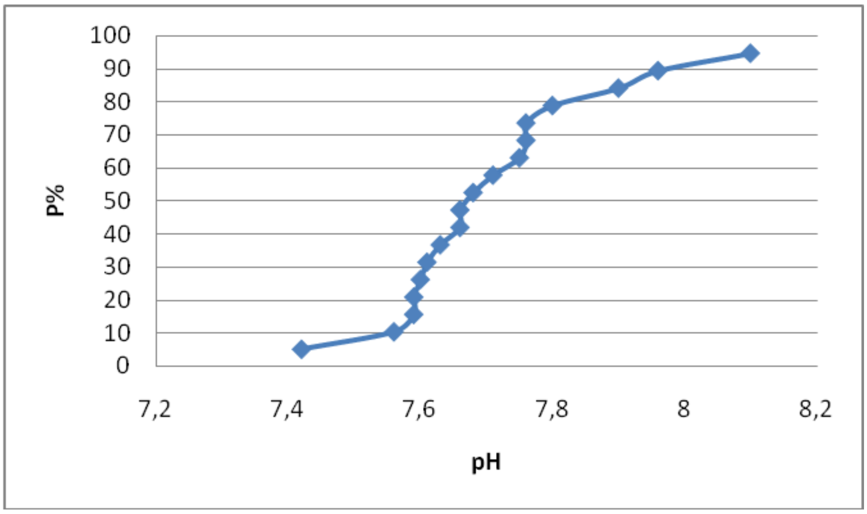

Figure 4.54: Outlet pH graph of Tank 11

4.4.2 Conductivity

33 Conductivity readings of inlet have shown in Figure 4.55. Conductivity values of inlet water were ranged in 1128-1930 mS with an average of 1656 mS.

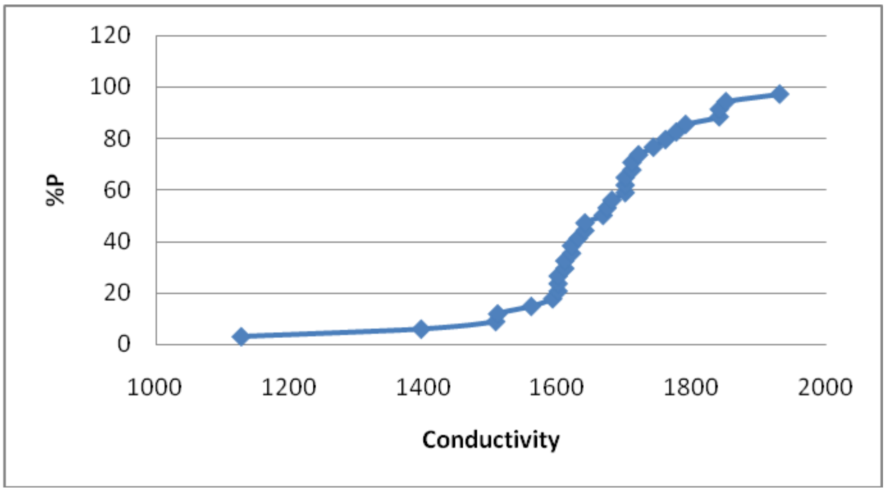

Figure 4.55: Inlet Conductivity

Average outlet conductivity values were obtained as 1760mS and 1637mS for Type I and Type II reactors respectively. Outlet value of Type I reactors was more than Type II reactor even higher than average inlet value which is 1656mS as seen in Table 4.85. It should be noted again that these tables includes values from startup period as well (Table 4.83 and Table 4.84). Hence another comparison was needed to see performance of tanks at last two months period.

Table 4.83: Conductivity values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mS)	Average (mS)	Standard Deviation
8	Sugar Cane	3	-	1112-2150	1734	357
7	Sugar Cane		+	1115-2410	1629	457
2	Cane	5	-	1177-2050	1575	243
4	Reed		-	1192-2620	1961	256
6	Sugar Cane		-	1175-2410	1847	394
1	Cane	5	+	1179-3050	1939	708
3	Reed		+	1262-1660	1461	133
9	Reed/Cane		+	1150-2160	1732	363
5	Sugar Cane		+	1192-2620	1961	587

Table 4.84: Conductivity values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	1102-3180	1753	498
16	Sugar Cane		+	1040-2260	1626	331
11	Cane	5	-	1203-2920	1666	353
13	Reed		-	1186-2230	1627	352
15	Sugar Cane		-	1104-2120	1609	374
10	Cane	5	+	1120-3011	1889	659
12	Reed		+	1130-2420	1484	442
18	Reed/Cane		+	1085-1770	1467	204
14	Sugar Cane		+	1027-2500	1612	496

Table 4.85: Removal rate comparison

Tank Specification	Average Cond. Inlet (mS)	Average Cond. Effluent (mS)	Removal Rate
Type I	1656	1760	-6%
Type II		1637	1%

In the Table 4.86, it is possible to see same amount of removal efficiency difference. And also table shows that Type I reactors performance was good since its outlet value was lower than inlet average as opposite of Table 4.85. As a result there was no significant difference between two different type systems in term of conductivity removing performance.

Table 4.86: Average conductivity values of first and last two months

Tank Specification	First Two Months Average (mS) / Removal % (Inlet = 1631)	Last Two Months Average (mS) / Removal % (Inlet = 1452)
Type I	1530 / 6%	1390 / 4%
Type II	1547 / 5%	1380 / 5%

Tank 11 selected to represent both type of systems since there was no significant difference between systems (Figure 4.56).

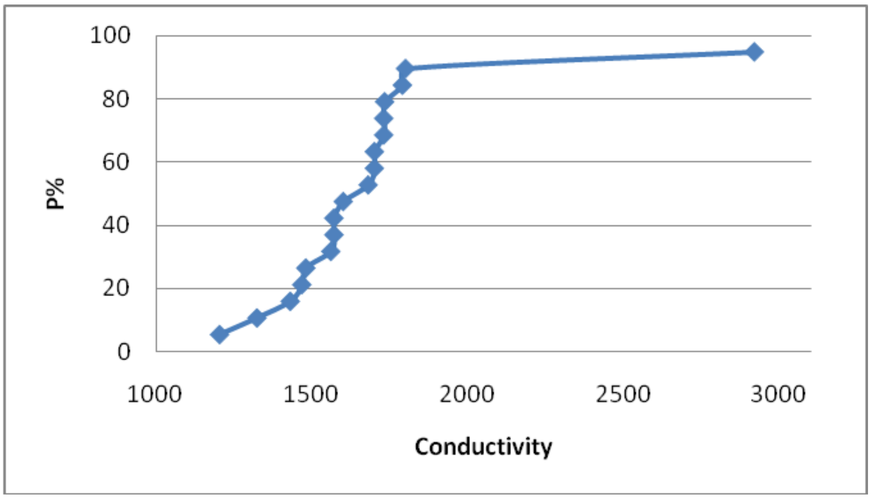

Figure 4.56: Conductivity outlet graphic of Tank 11

4.4.3 COD

35 COD readings of inlet have shown in Figure 4.57. During the experiments COD value of inlet wastewater changed in the range of 81 - 609 mg/lt with an average of 318 mg/lt.

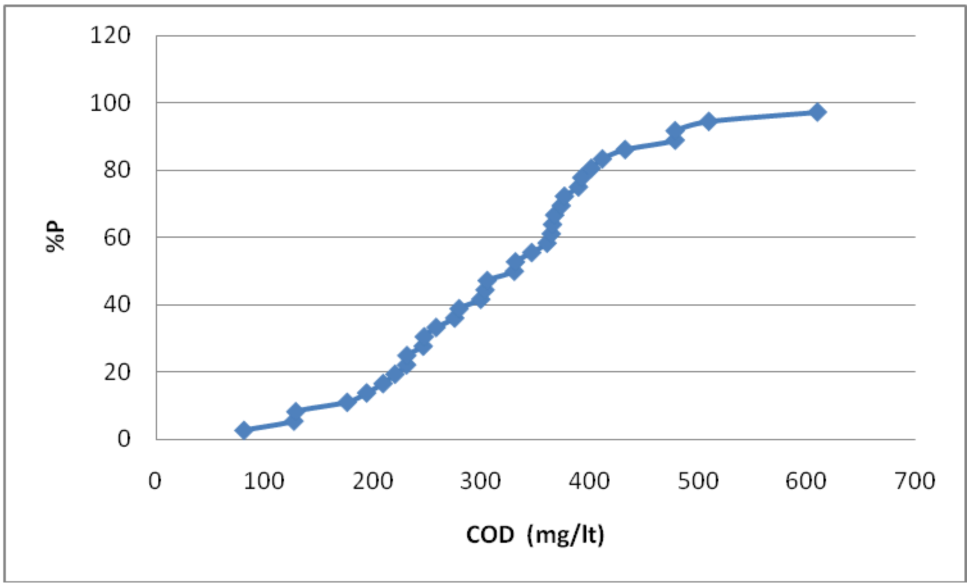

Figure 4.57: Inlet COD

Average values and specifications are shown in the (Table 4.87 and Table 4.88).

Table 4.87: COD values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	95-333	209	88
7	Sugar Cane		+	44-344	194	106
2	Cane	5	-	80-306	177	81
4	Reed		-	97-272	170	61
6	Sugar Cane		-	103-251	189	52
1	Cane	5	+	80-494	266	140
3	Reed		+	50-277	182	89
9	Reed/Cane		+	69-441	174	123
5	Sugar Cane		+	71-248	125	65

Table 4.88: COD values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	73-410	220	107
16	Sugar Cane		+	89-330	181	84
11	Cane	5	-	109-596	223	111
13	Reed		-	45-271	145	69
15	Sugar Cane		-	92-270	168	67
10	Cane	5	+	97-258	156	53
12	Reed		+	49-277	154	93
18	Reed/Cane		+	30-409	187	134
14	Sugar Cane		+	110-451	206	115

Both type of systems performed well and their average values 187 mg/l for Type I and 182 mg/l for Type II reactors considering average inlet value 318 mg/l (Table 4.89). To have more accurate aspect also first and last two months period were inspected.

Table 4.89: Removal rate comparison

Tank Specification	Average Cond. Inlet (mg/lt)	Average Cond. Effluent (mg/lt)	Removal Rate
Type I	318	187	41%
Type II		182	43%

Performances of both systems were almost same again in the last two months period as seen in Table 4.90. However after start up period when system was stabilized the overall system performance greatly increased from 28% to 50% in terms of removing efficiency as expected.

Table 4.90: Average COD values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 337)	Last Two Months Average(mg/l) / Removal % (Inlet = 315)
Type I	243 / 28%	145 / 54%
Type II	241 / 28%	155 / 51%

Best performed tanks Type I Tank 5 and Type II Tank 12's outlet COD graph is down below to represent other tanks (Figure 4.58).

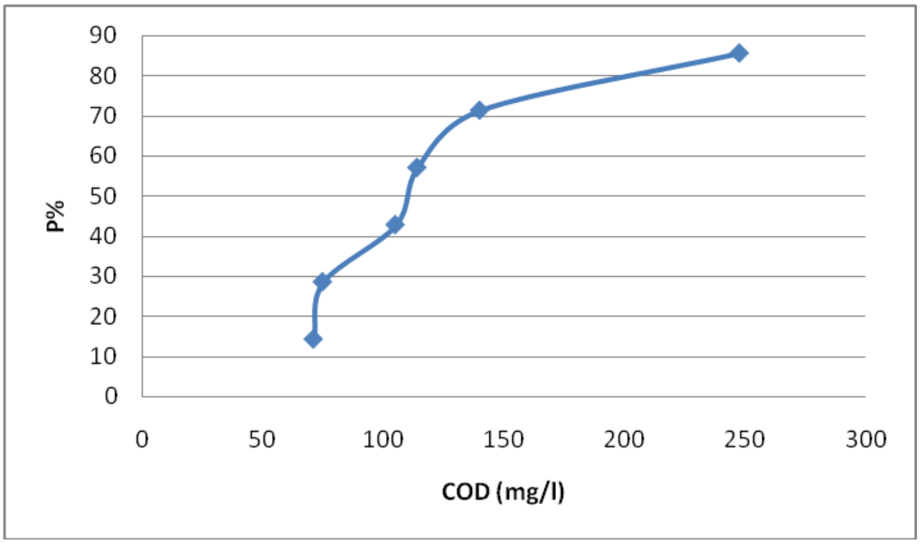

Figure 4.58: Outlet COD graph of Tank 5

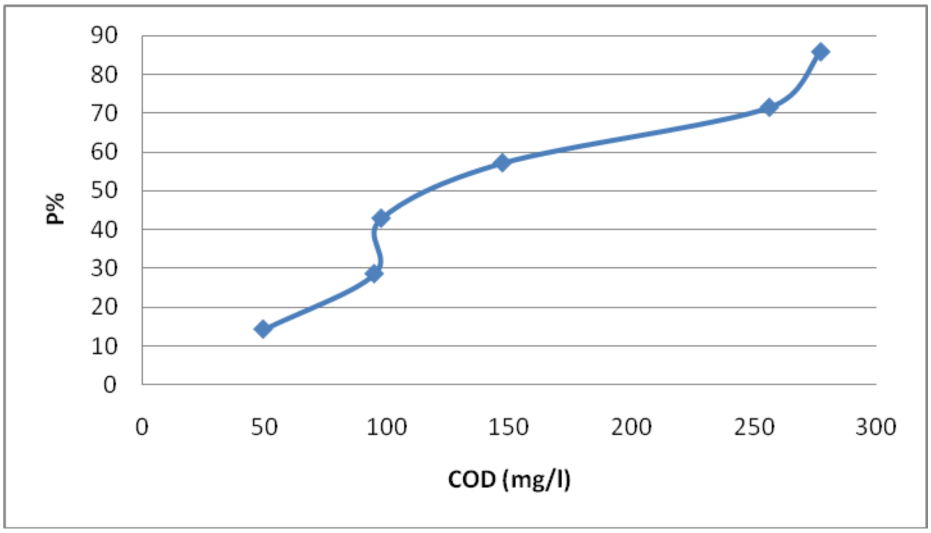

Figure 4.59: Outlet COD graph of Tank 12

4.4.4 BOD_5

Inlet BOD value ranged in 30 – 260 mg/l with an average of 139 mg/l (Figure 4.60).

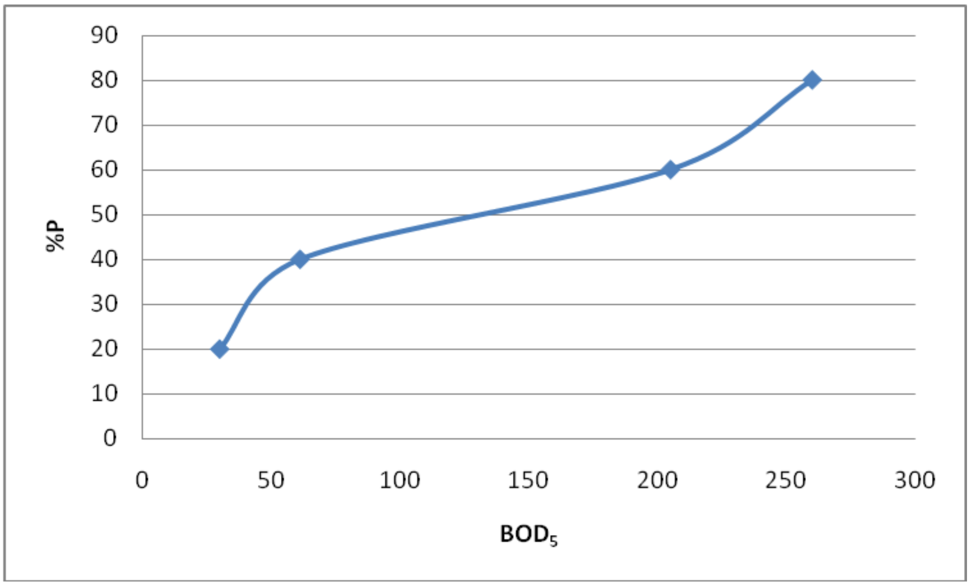

Figure 4.60: Inlet BOD

BOD values will not be considered, since experimental data was not enough (Table 4.91 and Table 4.92).

Table 4.91: BOD values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	-	-	-
7	Sugar Cane		+	235	235	-
2	Cane	5	-	270	270	-
4	Reed		-	190	190	-
6	Sugar Cane		-	-	-	-
1	Cane	5	+	220	220	-
3	Reed		+	-	-	-
9	Reed/Cane		+	-	-	-
5	Sugar Cane		+	-	-	-

Table 4.92: BOD values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	83	83	-
16	Sugar Cane		+	-	-	-
11	Cane	5	-	124	124	-
13	Reed		-	-	-	-
15	Sugar Cane		-	-	-	-
10	Cane	5	+	-	-	-
12	Reed		+	-	-	-
18	Reed/Cane		+	-	-	-
14	Sugar Cane		+	-	-	-

BOD values will not be considered, since experimental data was not enough.

4.4.5 TSS

During the study experiments TSS value ranged in 24-200 mg/l with an average of 91 mg/l (Figure 4.61).

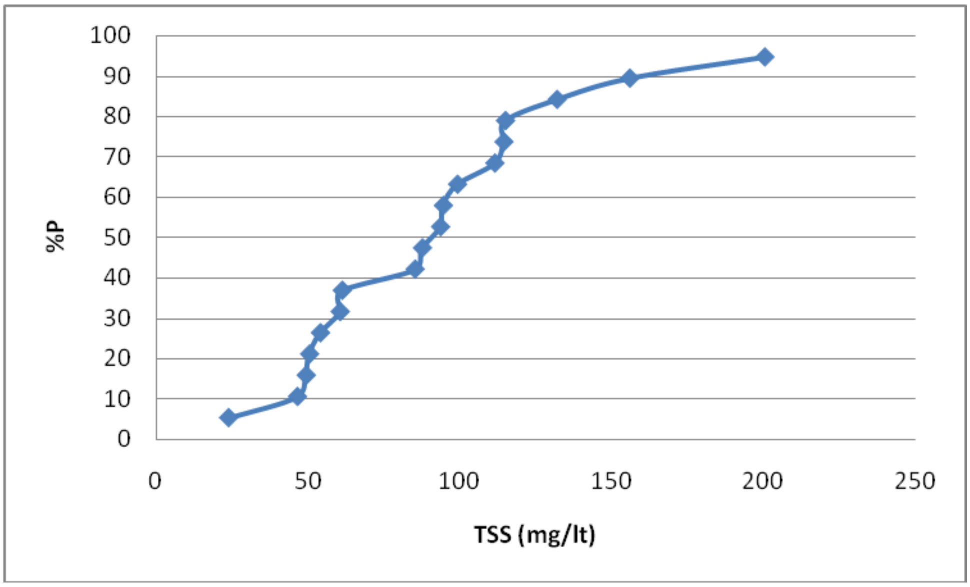

Figure 4.61: Inlet TSS

Number of experiments was not much to compare in detail, but was enough to have an idea about the systems (Table 4.93 and Table 4.94).

Table 4.93: TSS values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	16-95	62	56
7	Sugar Cane		+	15-55	32	28
2	Cane	5	-	9-55	29	24
4	Reed		-	8-179	62	69
6	Sugar Cane		-	25-79	53	22
1	Cane	5	+	17-66	41	26
3	Reed		+	3-85	37	39
9	Reed/Cane		+	10-50	24	18
5	Sugar Cane		+	7-45	25	19

Table 4.94: TSS values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	65-83	73	8
16	Sugar Cane		+	17-92	60	33
11	Cane	5	-	5-174	64	55
13	Reed		-	3-60	31	41
15	Sugar Cane		-	1-65	34	32
10	Cane	5	+	4-41	29	17
12	Reed		+	2-52	21	27
18	Reed/Cane		+	6-85	40	39
14	Sugar Cane		+	18-80	52	32

Average outlet values were 40 mg/l for Type I and 45 mg/l for Type II reactors. Although the values were close to each other, Type I reactors average value was slightly better than Type II reactors Table 4.95. However overall system performance was good, removal efficiencies for both type reactors were more than 50%.

Table 4.95: Removal rate comparison

Tank Specification	Average TSS Inlet (mg/l)	Average Effluent TSS (mg/l)	Removal Rate
Type I	91	40	56%
Type II		45	51%

Table 4.96 shows clear performance difference between first and last two months period. After system reached stabilized conditions average values for both type reactors were almost equal. At the same time again Type I reactors performance was slightly better than the other type.

Table 4.96: Average TSS values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 64)	Last Two Months Average(mg/l) / Removal % (Inlet = 115)
Type I	85 / -32%	53 / 54%
Type II	57 / 11%	55 / 52%

Tanks 6's outlet average value was close to overall outlet average and number of experiments was more than others, hence Tank 6 selected to represent other tanks (Figure 4.62).

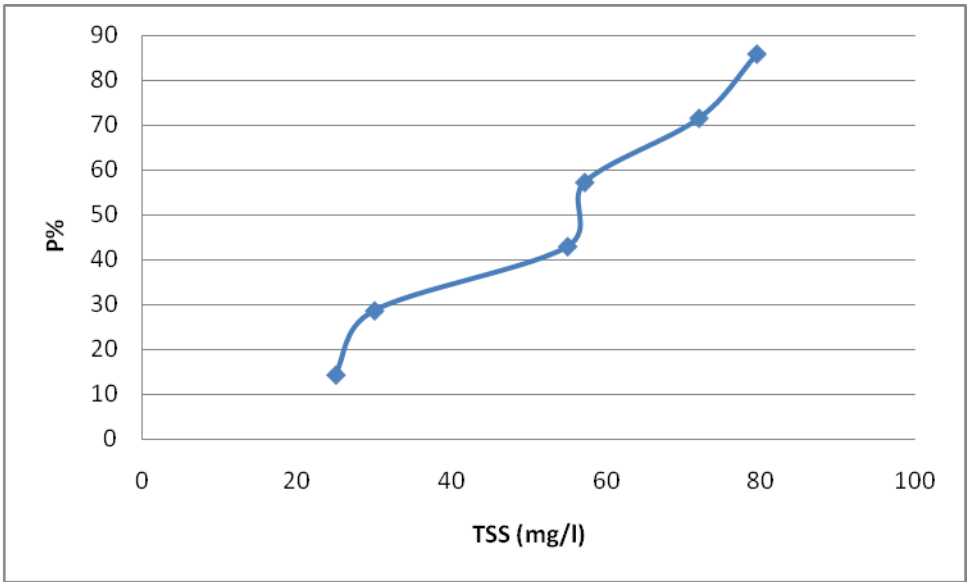

Figure 4.62: Outlet TSS graph of Tank 6

4.4.6 VSS

Inlet VSS value ranged in 50 – 130 mg/l with an average of 82 mg/l (Figure 4.63).

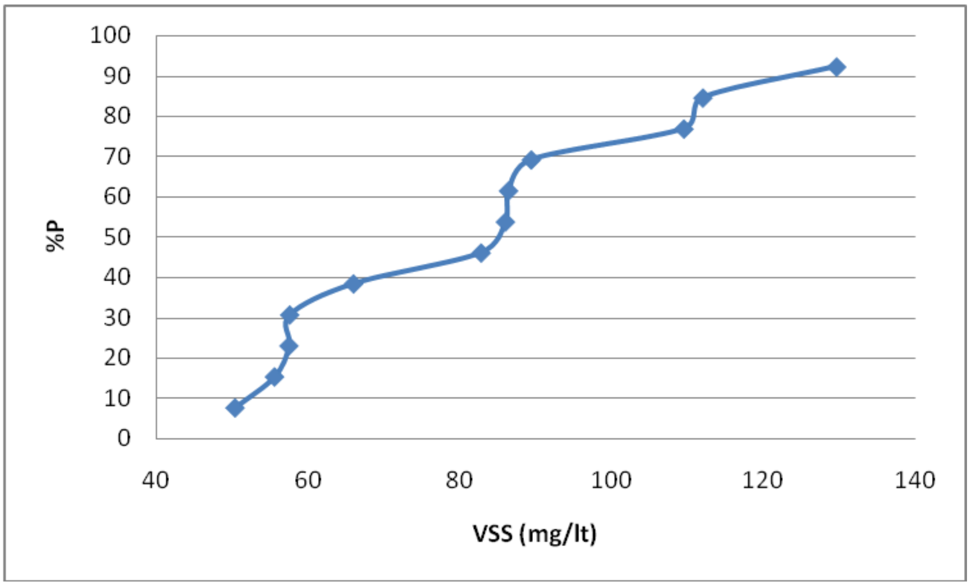

Figure 4.63: Inlet VSS

Average values and specifications are shown in the (Table 4.97 and Table 4.98).

Table 4.97: TSS values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	16-59	38	30
7	Sugar Cane		+	48	48	-
2	Cane	5	-	4-15	9	5
4	Reed		-	24-31	28	5
6	Sugar Cane		-	25-57	44	17
1	Cane	5	+	62	62	-
3	Reed		+	61	61	-
9	Reed/Cane		+	10-45	26	18
5	Sugar Cane		+	7-38	19	17

Table 4.98: TSS values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	0-63	32	45
16	Sugar Cane		+	64-76	70	9
11	Cane	5	-	5-146	56	48
13	Reed		-	-	-	-
15	Sugar Cane		-	-	-	-
10	Cane	5	+	4-38	21	17
12	Reed		+	25	25	-
18	Reed/Cane		+	6	6	-
14	Sugar Cane		+	18	18	-

Inlet average value was 82 mg/l which is 90% of inlet average TSS value 92 mg/l, that shows 90% of the water consist of volatile part. Although number of experiments was very less, it was enough to see that both systems performed well in terms of VSS removal (Table 4.99).

Table 4.99: Removal rate comparison

Tank Specification	Average VSS Inlet (mg/l)	Average Effluent VSS (mg/l)	Removal Rate
Type I	82	37	55%
Type II		30	63%

4.4.7 NH_4^+

Inlet NH_4^+ values ranged in 1 – 92 mg/l with an average of 47 mg/l (Figure 4.64).

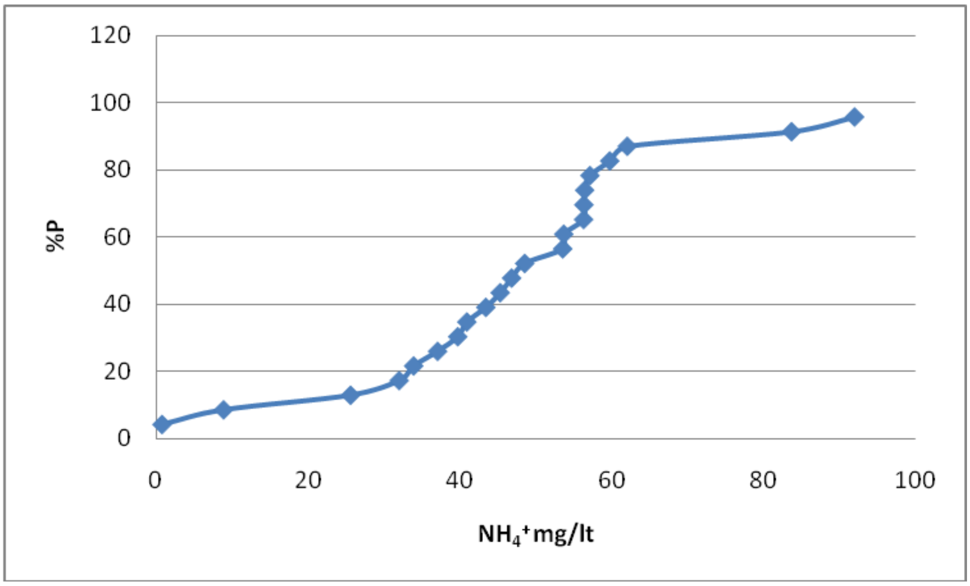

Figure 4.64: Inlet NH_4^+

Average values and specifications are shown in the (Table 4.100 and Table 4.100).

Table 4.100: NH_4^+ values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	18-41	29	12
7	Sugar Cane		+	0-82	28	30
2	Cane	5	-	7-46	34	17
4	Reed		-	16-49	36	13
6	Sugar Cane		-	17-49	29	10
1	Cane	5	+	5-57	25	16
3	Reed		+	1-45	24	19
9	Reed/Cane		+	0-13	4	6
5	Sugar Cane		+	1-24	14	9

Table 4.101: NH_4^+ values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	5-62	37	18
16	Sugar Cane		+	7-47	24	14
11	Cane	5	-	2-48	37	14
13	Reed		-	2-31	12	11
15	Sugar Cane		-	7-39	28	14
10	Cane	5	+	2-29	21	11
12	Reed		+	0-58	26	24
18	Reed/Cane		+	7-50	31	17
14	Sugar Cane		+	2-31	15	12

Both type of reactors performed well and reduced the amount of NH_4^+ significantly. Although the difference was not major, Type I reactors performed better than Type II reactors as seen in the table below (Table 4.102).

Table 4.102: Removal rate comparison

Tank Specification	Average NH_4^+ Inlet (mg/l)	Average Effluent NH_4^+ (mg/l)	Removal Rate
Type I	47	25	47%
Type II		27	43%

Both systems have kept same percentage of difference at all time even in the last two months periods (Table 4.103).

Table 4.103: Average NH_4^+ values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 46)	Last Two Months Average(mg/l) / Removal % (Inlet = 54)
Type I	33 / 28%	27 / 50%
Type II	31 / 33%	29 / 46%

Tank 9 and Tank 5's values ommited due less number of experiments. Tank 3 was the best performed Tank between Type I and Tank 14 was the best between Type II tanks in terms of ammonium removal (Figure 4.65 and Figure 4.66).

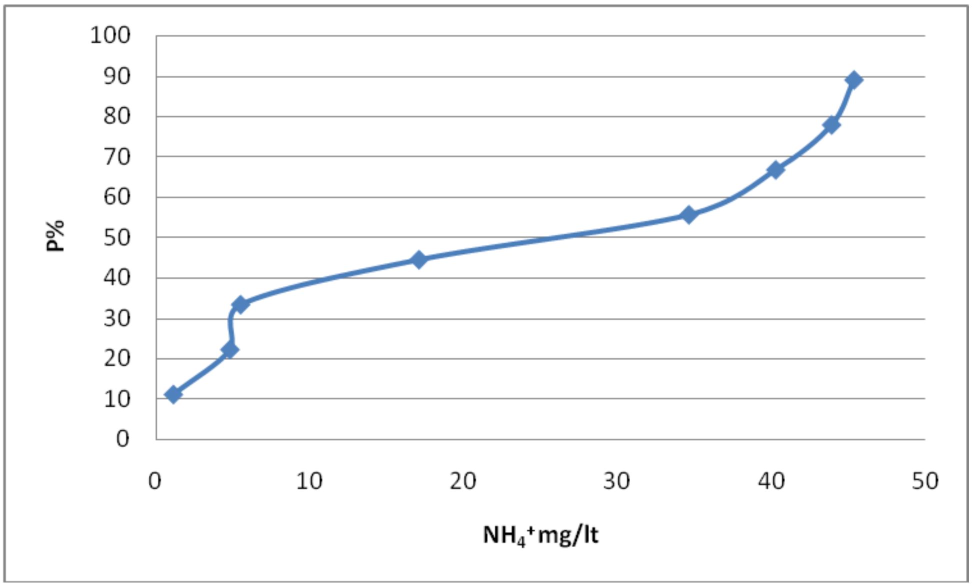

Figure 4.65: Outlet NH_4^+ graph of Tank 3 (Type I)

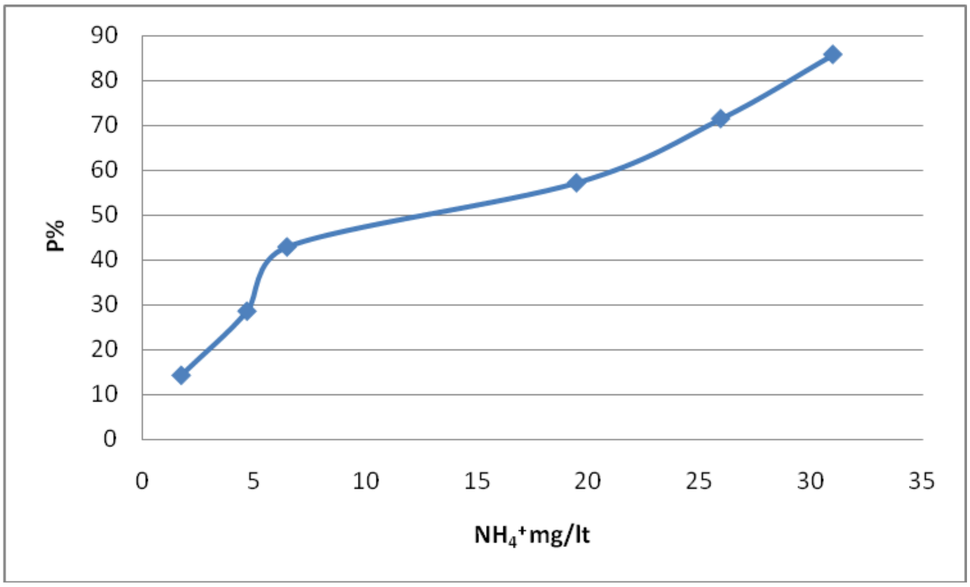

Figure 4.66: Outlet NH_4^+ graph of Tank 14 (Type II)

4.4.8 PO_4^{-3}

Minimum value was 3 mg/lt and maximum value was 66 mg/lt with an average of 15 mg/lt during the study (Figure 4.67).

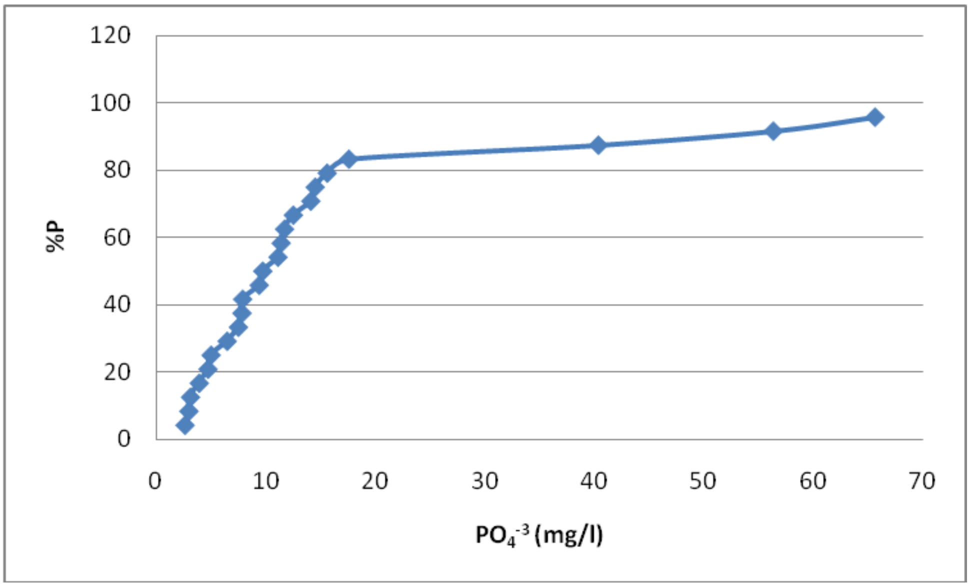

Figure 4.67: Inlet PO_4^{-3}

Average values and specifications are shown in the (Table 4.104 and Table 4.105).

Table 4.104: PO_4^{-3} values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	2-13	9	4
7	Sugar Cane		+	4-18	14	5
2	Cane	5	-	4-13	10	3
4	Reed		-	3-12	9	4
6	Sugar Cane		-	2-20	12	6
1	Cane	5	+	5-47	17	14
3	Reed		+	9-19	14	3
9	Reed/Cane		+	5-16	12	4
5	Sugar Cane		+	1-15	10	5

Table 4.105: PO_4^{-3} values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	5-20	12	6
16	Sugar Cane		+	5-34	16	9
11	Cane	5	-	1-19	12	4
13	Reed		-	13-20	16	3
15	Sugar Cane		-	4-29	18	10
10	Cane	5	+	7-20	12	4
12	Reed		+	9-21	15	4
18	Reed/Cane		+	1-18	11	5
14	Sugar Cane		+	6-31	15	10

As seen in the Table 4.106 Type I reactors performed better than Type II reactors. However the difference was not significant.

Table 4.106: Removal rate comparison

Tank Specification	Average PO_4^{-3} Inlet (mg/l)	Average Effluent PO_4^{-3} (mg/l)	Removal Rate
Type I	15	12	20%
Type II		14	7%

Table 4.107: Average PO_4^{-3} values of first and last two months

Tank Specification	First Two Months Average(mg/l) / Removal % (Inlet = 14)	Last Two Months Average(mg/l) / Removal % (Inlet = 13)
Type I	13 / 7%	13 / 0%
Type II	13 / 7%	14 / -7%

Tank 4 and Tank 18's outlet graphs are down below (Figure 4.68 and Figure 4.69).

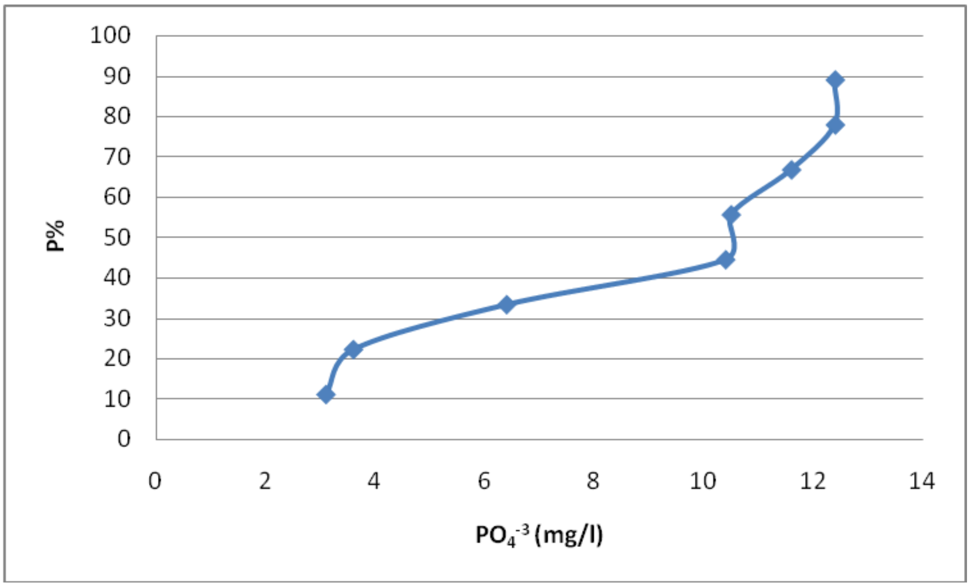

Figure 4.68: Outlet PO_4^{-3} graph of Tank 4

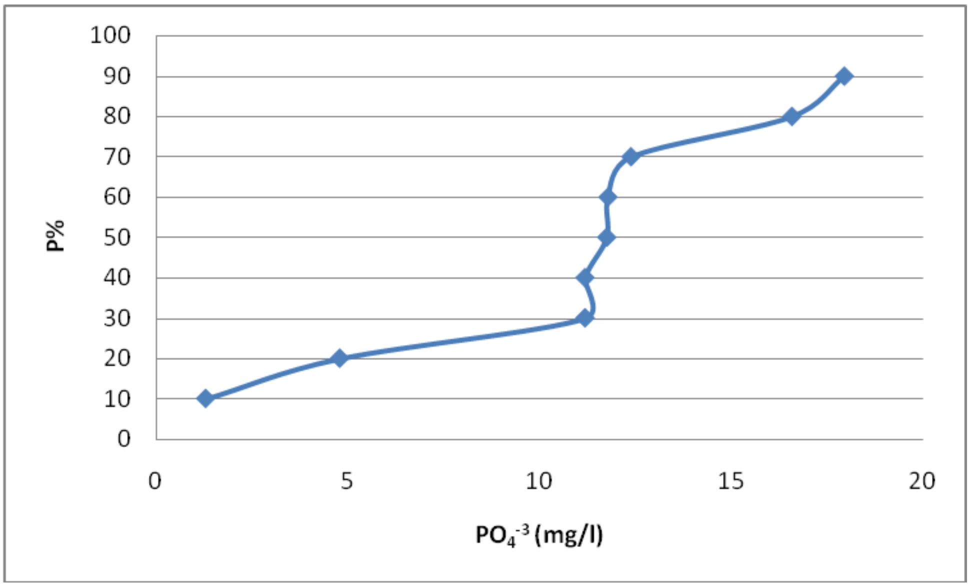

Figure 4.69: Outlet PO_4^{-3} graph of Tank 18

4.4.9 TKN

Minimum value was 49 mg/l and maximum value was 220 mg/l with an average of 88 mg/l during the study (Figure 4.70).

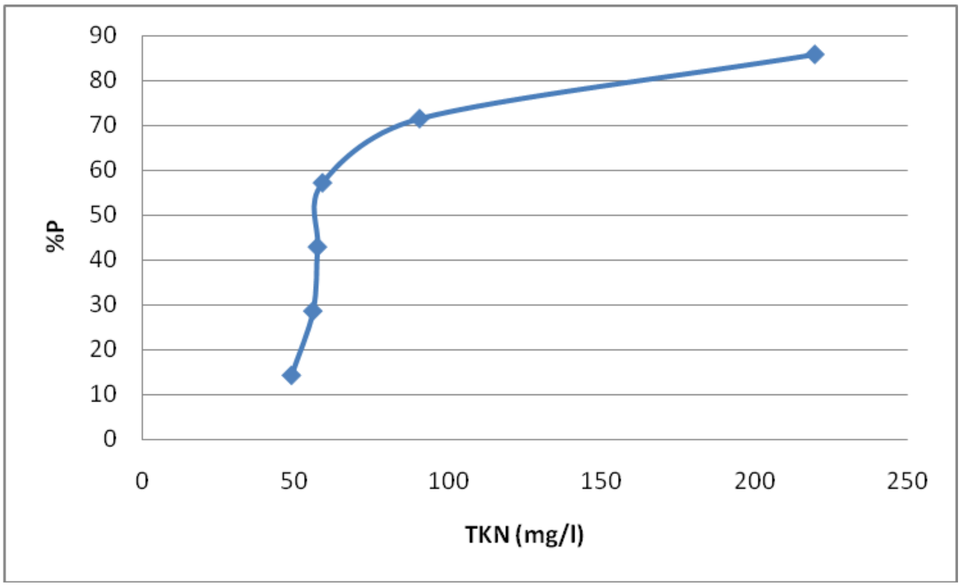

Figure 4.70: Inlet TKN

Average values and specifications are shown in the (Table 4.108 and Table 4.109). Due to availability of laboratory conditions number of experiments was maximum two.

Table 4.108: TKN values of Type I reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range (mg/lt)	Average (mg/lt)	Standard Deviation
8	Sugar Cane	3	-	73	73	-
7	Sugar Cane		+	-	-	-
2	Cane	5	-	35	35	-
4	Reed		-	203	203	-
6	Sugar Cane		-	12-109	60	-
1	Cane	5	+	81	81	-
3	Reed		+	21	21	-
9	Reed/Cane		+	26	26	-
5	Sugar Cane		+	27	27	-

Table 4.109: TKN values of Type II reactors

Reactor Number	Plants Type	RT (day)	Aeration	Range	Average	Standard Deviation
17	Sugar Cane	3	-	-	-	-
16	Sugar Cane		+	37	37	-
11	Cane	5	-	38-42	40	-
13	Reed		-	31	31	-
15	Sugar Cane		-	-	-	-
10	Cane	5	+	-	-	-
12	Reed		+	-	-	-
18	Reed/Cane		+	7	7	-
14	Sugar Cane		+	-	-	-

Even though data was not enough to make a detailed comparison between two types of reactors in terms of TKN removal, both of the system values were less than average inlet value 88 mg/l (**Table 4.110**110).

Table 4.110: Removal rate comparison

Tank Specification	Average TKN Inlet (mg/l)	Average Effluent TKN (mg/l)	Removal Rate
Type I	88	66	25%
Type II		29	67%

5. DISCUSSION

5.1 TASK 1 (Anaerobic lagoon)

As seen in the Table 5.1, covered tanks performed better in terms of COD and TSS removal. Covered Tanks 1, 2, 5 and 7 has given the best results with overall average values 274, 275, 271, 269 mg/l respectively. Although the difference was not major, Tank 7 with a retention time of 4 days performed better.

According to *Wall et al., 2000* at same operating conditions, covered anaerobic reactors has a great advantage over usual anaerobic reactors and has a removal rate of 89%. It should be noted that in our study, even inlet water to our tanks coming from inlet of the real sized anaerobic lagoon, it actually was like outlet of the real anaerobic lagoon due to, lack of control over system (due to unforeseen technical problems wastewater pumped from the lagoon was partially already treated water). Therefore remaining organic matter part in water was mostly inert. In addition to that even though COD removal performance of tanks might be higher than what they have been found, our tanks could remove only already treated water below their actual capacity and efficiency was found at about 30%. Efficiency could have been better than this actual performance, but still should be interpreted within its own results due to differences in characteristics of inlet waters in different systems where it is not comparable.

The main concern of a wastewater treatment facility in operating an anaerobic system is that the various bacterial species function in a balanced and sequential way (Forster 1985). Hence, although other types of microorganisms may be present in the reactors, attention is focused mostly on the bacteria. Energy is required by elevated reactor temperatures to maintain microbial activity at a practical rate.(Generally, the optimum temperature for anaerobic processes is 35°C.) (Carl.E, 1999). Covered tanks reached optimum temperature easily compared to uncovered tanks. This helped to maintain proper conditions for activities of anaerobic bacteria.

As in the COD removal, covered tanks performed better in TSS removal. Tank 2 with a retention time of 8 and a plastic cover, has given the best result which is 59 mg/l overall. Sedimentation was better for covered tanks due to closed system which prevents its contact with environment.

In treatment of NH_4^+, this time uncovered tanks performed better. *Bock et al,1995* found that under low oxygen concentrations of 0.2 mg l^{-1} two ammonia-oxidizers, Nitrosomonas europaea and Nitrosomonas eutropha, were able to simultaneously nitrify and denitrify, producing gaseous nitrogen products from NH_4-N. Since uncovered tanks were in contact with air, water-air layer on top which has oxygen, even it was in low concentration, treatment of NH_4^+ was better with these reactors than covered tanks.

Another reason for better ammonium treatment also could be slightly higher pH values (it was 7,9 overall for uncovered tanks and 7,2 for covered tanks) and open air condition. Figure 5.1 shows the pH and temperature dependant relative shares of NH_4 and NH_3.

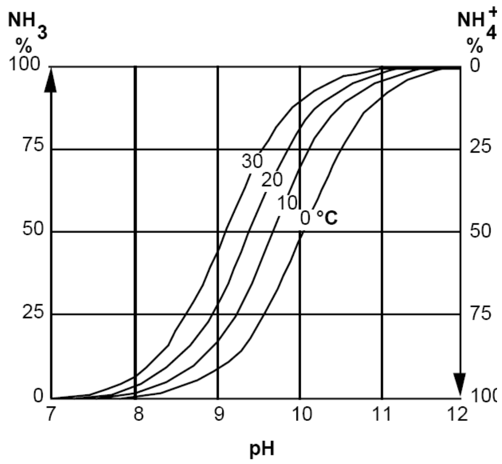

Figure 5.1: NH_4 and NH_3 as a Function of pH and Temperature (Heinss-Strauss, 1998)

At the beginning period NH_4^+ values was not as good as in covered tanks, but when the overall and last two months values were inspected it was clear that uncovered tanks was better in terms of NH_4^+ removal. Tank 12 has given the best result between uncovered tanks with a retention time of 4 days. Average overall outlet value was 35 mg/l.

Both types of the systems almost have same performance in treatment of PO_4^{-3}.

In biological treatment, either in aerobic and anaerobic treatment conditions, liquid phase-phosphate concentrations changes. In anaerobic conditions liquid phase-phosphate concentration rises and organisms release phosphate to ambient (Dursun and Oktaç, 2005). At last two months period the difference was only 1 mg/l while it was 2 mg/l in overall. Uncovered tanks performed slightly better than covered tanks as seen in the Table 5.1.

Table 5.1: Task 1 Summarization of results

Parameters	**First Two Months**			**Last Two Months**		
	Inlet Average (mg/l)	**Outlet Average (mg/l)**		**Inlet Average (mg/l)**	**Outlet Average (mg/l)**	
		Covered / Removal Rate%	**Uncovered / Removal Rate%**		**Covered / Removal Rate%**	**Uncovered / Removal Rate%**
COD	578	374 / 35%	422 / 26%	434	**300 / 31%**	336 / 23%
TSS	131	101 / 23%	104 / 21%	167	**60 / 64%**	82 / 51%
NH_4^+	57	44 / 23%	52 / 9%	37	27 / 27%	**15 / 59%**
PO_4^{-3}	19	17 / 11%	15 / 21%	9	8 / 11%	**7 / 22%**

5.2 TASK 2 (Facultative Lagoon)

Although olive type reactors' performance was not good at the beginning period, they performed better than pine type reactors after they were stabilized. *Robertson et al., 2005* also reported that at the early stages of tests with their wood chip filters, the media leached carbonaceous COD (from tannic acid, etc.) out of the media. However performance difference between two systems was not major.

Tank 1 and Tank 2 has given the best result between olive type reactors with an overall average value of 252 mg/l and 200 mg/l respectively. All of olive type reactors had same retention time which is 6 days. Only the difference was the height of the reactors and the size of the woodchips filled inside. It was clear that, 2,5m height reactors that provide more volume to hold material inside resulted better than

1,5m reactors. Tank 2 filled with woodchips with 5cm diameter which raises the contact area performed better than tanks which filled with 10cm diameter woodchips.

In TSS removal olive type reactors again performed better than pine type reactors. Tank 5 between these reactors had the best performance comparing to others. It had a height of 1,5m, 6 days retention time and it was filled with 5cm diameter sized woodchips. Smaller sized woodchips provided more surface area for suspended solids to be attached on the spaces and pores on woodchips.

Shanableh et al., 1996 stated that there is a relation between COD, NH_4^+ and PO_4^{-3} removal performance of biofilters; increase in denitrification and phosphorus release results increase in COD removal. This relation also can be seen in Table 5.2.

In terms of NH_4^+ treatment, performance difference between two systems was not significant. However pine type reactors' performance was slightly better than olive type reactors (2mg/l). NH_4^+ values were low that show there was nitrification which causes NH_4^+ reduction. Both systems achieved a performance which is over 60%. Tank 9 which has a retention time of 6 days, a height of 2,5m and filled with 5 cm sized woodchips, has given the best result among pine type reactors. Like in COD and TSS parameters optimal retention time was again 6 days.

Olive type reactors again were better than pine type reactors in PO_4^{-3} removal even the difference was slight. This difference has almost never changed since the beginning period as seen in the Table 5.2. Again tanks with 2,5m height which are Tank 1 and 2 in olive type reactors performed better than 1,5m ones. Both tanks' overall outlet average was 8 mg/l compared to 13 and 15mg/l of Tanks 5 and 4 respectively. Retention time was also same for all olive type reactors and 6 days. The size of the woodchips did not make any difference in terms of PO_4^{-3} removal.

Table 5.2: Task 2 Summarization of results

Parameters	**First Two Months**			**Last Two Months**		
	Inlet Average (mg/l)	**Outlet Average (mg/l)**		**Inlet**	**Outlet Average (mg/l)**	
		Olive / Removal Rate%	**Pine / Removal Rate%**		**Olive / Removal Rate%**	**Pine / Removal Rate%**
COD	565	1310 / -131%	568 / -1%	324	**145 / 55%**	161 / 50%
TSS	108	133 / -23%	107 / 1%	112	**62 / 45%**	70 / 37%
NH_4^+	50	48 / 4%	47 / 6%	51	20 / 61%	**18 / 65%**
PO_4^{-3}	17	13 / 23%	14 / 18%	13	**10 / 23%**	11 / 16%

5.3 TASK 3 (Seasonal Reservoir)

Table 5.3 includes average values for all tanks since there were only six tanks with respect to other tasks. As seen in the table there was a clear performance difference between first and last two months period in all parameters.

COD removal rate was 55% for all tanks at last two months period. Between these six reactors, rectangular shaped reactors have given the best results in all parameters. Retention time was 30 days for all reactors. Tank 4 was the best tank in terms of COD removal with a overall average value of 169mg/l considering overall inlet value was 446mg/l. Tank 4 was a rectangular tank which also had a fixed medium and aeration equipment inside. *Ahimou et al.,2006* mentioned that, low oxygen concentrations slowed biofilm growth rate, giving the biofilm more time to consolidate. Biofilm layer on the fixed medium performed better with aeration equipment since additional oxygen optimized the conditions for bacteria.

TSS performance of reactors also greatly improved by the time and achieved a removal performance of 68% (Table 5.3). Rectangular shaped reactors performed better than hexagonal ones in TSS removal like in the COD removal process. Tank 4 again has given the best result that has a fixed medium and aeration equipment. Overall average outlet for this tank was 30mg/l while inlet average was 105mg/l.

In terms of NH_4^+ treatment, Tank 6 which is in hexagonal shape with aeration equipment; has given an overall average outlet of 14mg/l. Tank 4's (Rectangular, with fixed medium and aeration) outlet value was 15mg/l. Aeration which helps nitrification process and ammonia volatilization caused better performance for this reactors.

In PO_4^{-3} removal same as NH_4^+ removal, Tank 6 and Tank 4 performed better than other tanks which are aerated. Hexagonal shaped Tank 6's overall average outlet was 6mg/l while Rectangular shaped Tank 4 with fixed medium, resulted 7mg/l. the difference was slight but Tank 6 without fixed medium performed better.

Table 5.3: Task 3 Summarization of results

Parameters	**First Two Months**		**Last Two Months**	
	Inlet Average (mg/l)	**Outlet average for all 6 reactors (mg/l)**	**Inlet**	**Outlet average for all 6 reactors (mg/l)**
		Outlet / Removal Rate%		**Outlet / Removal Rate%**
COD	510	449 / 12%	303	137 / 55%
TSS	91	145 / -59%	117	18 / 85%
NH_4^+	57	48 / 16%	63	20 / 68%
PO_4^{-3}	16	13 / 19%	12	3 / 75%

5.4 TASK 4 (Wetlands)

COD removal percentage of both type of wetlands was over 50%, Type I wetlands COD removal performance was slightly better than Type II (Table 5.4). The removal of COD in the constructed wetlands can occur via aerobic/anaerobic biological mechanisms, as well as by a variety of physical methods, including adsorption and filtration (Giraldo and Zarate, 2000). The difference between Type I and Type II is the rocks used to fill the tanks (Figure 3.28 and Figure 3.29). Type I reactors were filled with 3 layers of rocks with different sizes changing 0.5cm to 6cm, while Type II reactors were filled with 2 layers of rocks with sizes changing 5cm to 12cm. Sugar Cane seeded Tank 5 which has aeration and a RT of 5 days; has given the best result between Type I reactors with an overall average of 125mg/l considering overall average inlet was 318mg/l.

In TSS removal both systems' performances were over 50% and close to each other (Table 5.4). *Nerella et al., 2000* studied that there was a reduction of 88% in TSS and 83% in VSS by passing the influent through the wetlands in their study. In our study results was not as high as 80% but it should be noted that the inlet water to Task 4 actually is outlet of real sized seasonal reservoir of Sakhnin WWTP. Tank 5 again along with Reed and Cane seeded Tank 9 has given best result between Type I reactors. Results were 25mg/l for Tank 5 and 24mg/l for Tank 9 considering average inlet value was 95mg/l.

NH_4^+ removal of both systems were good, Type I reactors performed better again. Removal percentage of NH_4^+ was 50%. *Reddy and Patrick, 1984* stated that if the pH is below 8.0 the ratio between ammonia and ammonium ions is 1:1 and the losses via volatilization are significant. Average pH values for both type of systems was 7.3, hence this also could have an effect along with nitrification and adsorption. Also *Kadlec and Knight, 1996* pointed out that if the wetland substrate is exposed to oxygen, perhaps by periodic draining, sorbed ammonium may be oxidized to nitrate.

In PO_4^{-3} removal both type of systems performed same since the beginning period, even in last two months there was no difference between both systems. According to *Ying-Feng et al., 2001*, vegetation uptake and deposition in soil and gravel are the two main mechanisms for phosphate removal.

Table 5.4: Task 3 Summarization of results

Parameters	First Two Months			Last Two Months		
	Inlet Average (mg/l)	Outlet Average (mg/l)		Inlet	Outlet Average (mg/l)	
		Type I / Removal Rate%	Type II / Removal Rate%		Type I / Removal Rate%	Type II / Removal Rate%
COD	337	243 / 28%	241 / 28%	315	**145 / 54%**	155 / 51%
TSS	64	85 / -32%	57 / 11%	115	**53 / 54%**	55 / 52%
NH_4^+	46	33 / 28%	31 / 33%	54	**27 / 50%**	29 / 46%
PO_4^{-3}	14	13 / 7%	13 / 7%	13	**13 / 0%**	14 / -7%

5.5 Comparison of Four Tasks

To have an idea about which pilot system performed better than the others, another comparison made as seen in the Table 5.5. Results calculated with average values of all data including startup period. Table 5.5 shows an overall performance comparison table to have a quick overview about all tasks.

In terms of COD removal Task 3 and Task 4's performances were better than other two Tasks with overall performances which are almost the same. While other Tasks performances were almost the same in TSS removal, Task 4 performed far better.

Task 1 was not good as others in ammonium removal, Task 2 and Task 4's performances were very close and again Tasks 3 performed better than other tasks with an overall removal rate of 67%. In terms of PO_4^{-3} removal again best performance was belonged to Task 3 with a rate of 37% while other Tasks performances were almost the same.

Table 5.5: Performance comparison of four TASKs.

Parameters	TASK 1 (Covered Anaerobic Lagoons)	TASK 2 (Biofilters)	TASK 3 (Concrete Tunnels)	TASK 4 (Wetlands)
COD	37%	28%	**44 %**	43%
TSS	31%	35%	33%	**56%**
NH_4^+	20%	50%	**67%**	47%
PO_4^{-3}	21%	21%	**37%**	20%

5.6 Performance Comparison of Existing Full Scale Units and Pilot Scale Units (This Research)

To see the improvement over existing full scale units, their treatment performance was compared to pilot scale units performance. All pilot scale units performed better than existing full scale units. The main reason for poor performance of existing units, they were not under control like pilot scale units.

Figure 5.2 shows the performance difference in percentage for selected main parameters. While existing lagoon performance was poor, since it was left by itself and was not under control, pilot scale system performed a lot better.

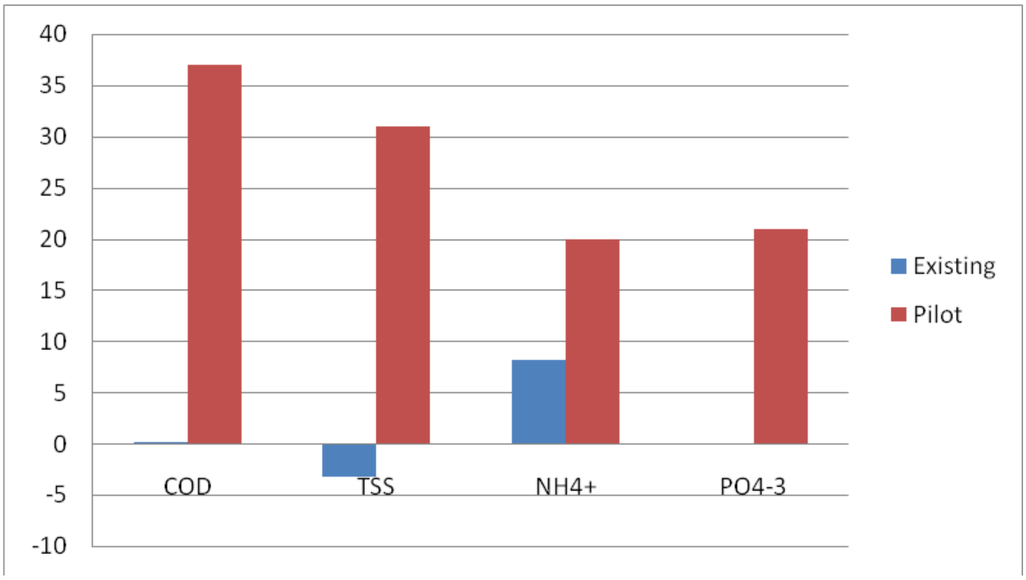

Figure 5.2: Task 1 performance in percentage, over existing anaerobic lagoon

Table 5.6 shows the performance difference in percentage between two system.

Table 5.6: Task 1 performance in percentage, over existing anaerobic lagoon

Parameters	Existing (treatment %)	Pilot (treatment %)
COD	0,2	37
TSS	-3,25	31
NH_4^+	8,2	20
PO_4^{-3}	0	21

As seen in the Figure 5.3 woodchips filled bio reactors performance was better than existing facultative lagoon. Especially in ammonium treatment the improvement was very big.

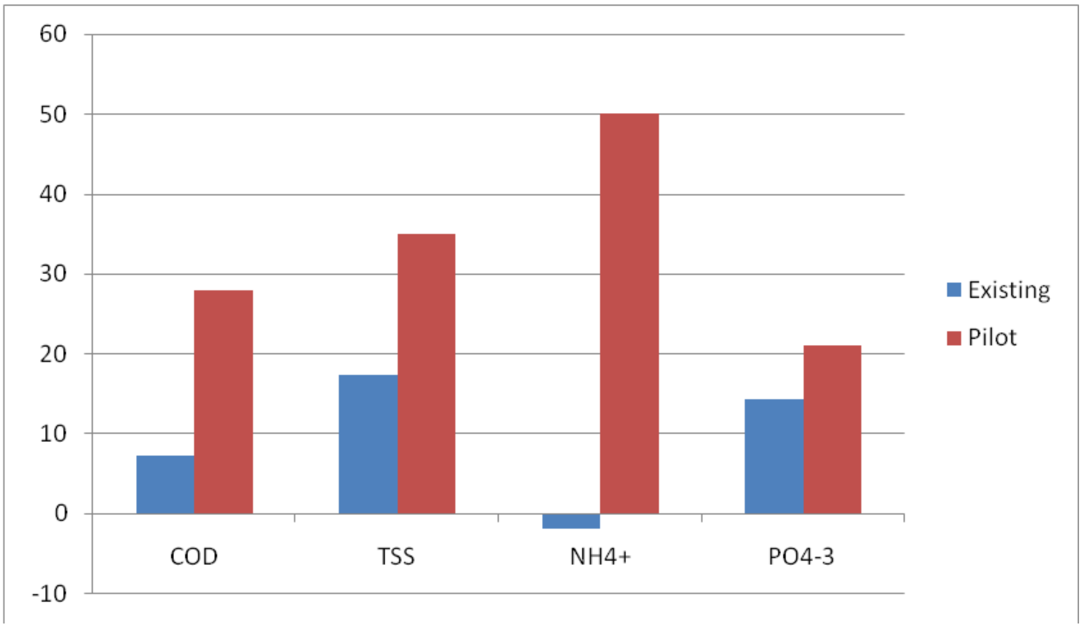

Figure 5.3: Task 2 performance in percentage, over existing facultative lagoon

Table 5.7 shows the performance difference in percentage between two system.

Table 5.7: Task 1 performance in percentage, over existing anaerobic lagoon

Parameters	Existing (treatment %)	Pilot (treatment %)
COD	7	28
TSS	17	35
NH_4^+	-2	50
PO_4^{-3}	14	21

While the performance difference was not much for COD parameter like PO_4^{-3}, concrete tunnels also improved the performance of existing seasonal reservoir for all parameters (Figure 5.4).

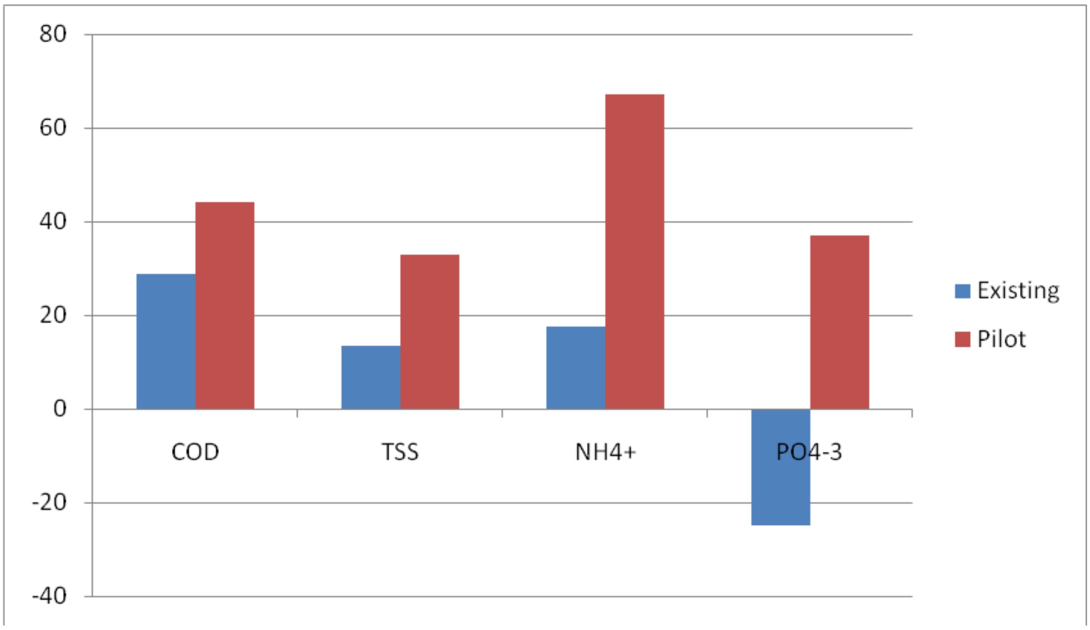

Figure 5.4: Task 3 performance in percentage, over existing seasonal reservoir.

Table 5.8 shows the performance difference in percentage between two system.

Table 5.8: Task 3 performance in percentage, over existing seasonal reservoir.

Parameters	Existing (treatment %)	Pilot (treatment %)
COD	29	44
TSS	13	33
NH_4^+	18	67
PO_4^{-3}	-25	37

Wetlands performance was not compared to existing wetland systems since existing full scale wetlands outlet values was not checked during the study.

Hence Task 4's performance was compared to existing units overall performance. There was a clear improvement for all parameters as seen in the Figure 5.5.

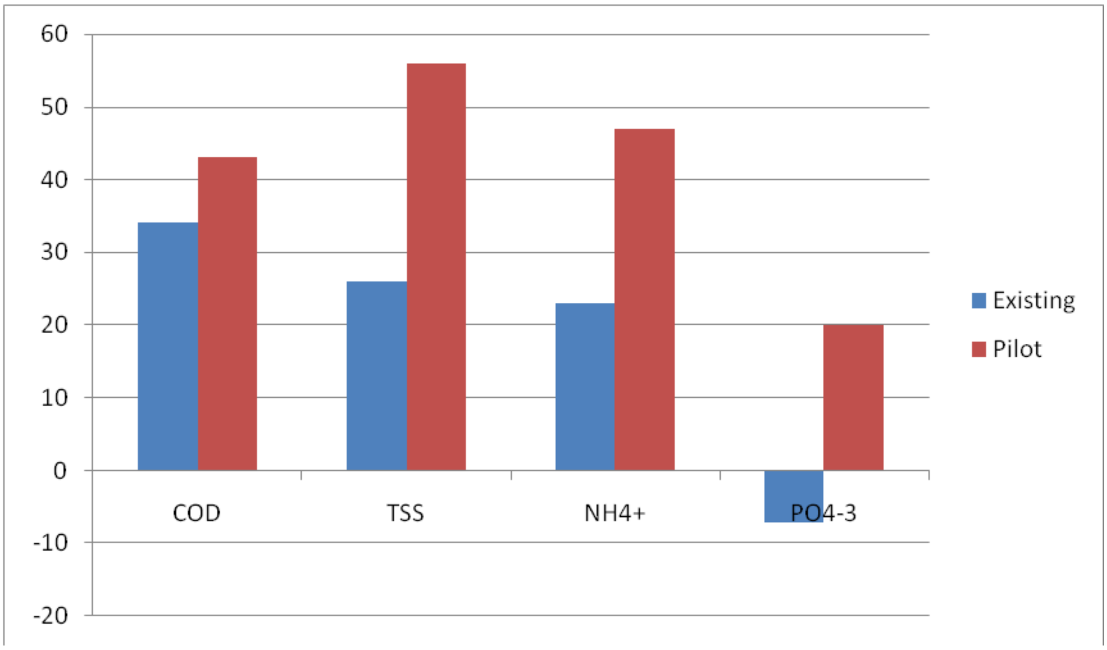

Figure 5.5: Task 4 performance in percentage, over existing seasonal reservoir

Table 5.9 shows the performance difference in percentage between wetlands and seasonal reservoir.

Table 5.9: Task 4 performance in percentage, over existing seasonal reservoir

Parameters	Existing (treatment %)	Pilot (treatment %)
COD	34	43
TSS	26	56
NH_4^+	23	47
PO_4^{-3}	-7	20

6. CONCLUSIONS AND RECOMMANDATIONS

Recently, natural treatment systems are being developed and become common all over the world. Especially for small communities which meet the field requirements of natural systems and have a population ranged between 2000 - 5000, natural treatment systems are more suitable than conventional treatment systems and recommended to use.

In this study, different setups and combinations were experimented to improve the performance of existing natural treatment system in Sakhnin with the materials that are easy to find, locally.

Covering the anaerobic lagoons improved the performance as expected by helping to easily create the anaerobic circumstances and to keep the lagoons from open air conditions effects. These setups will be tested with additional heating element following years to improve performance values, especially during the winter season.

Using woodchips filled bio-filters has many advantages over existing facultative lagoon. First of all they helped to save more space, which is a significant refinement when natural treatment comes to matter. They filled with the woodchips which could be easily found at the region. Filters with longer height (2,5m) performed better than short ones (1,5m).

To improve performance of seasonal reservoir, concrete tunnels and hexagonal shaped tanks constructed and tested with various setup combinations. Rectangular shaped tanks performed better. Especially, tanks with aeration and additional plastic curtains which provided additional surface area for biofilm performed better. Also, these tunnels will be constructed in existing seasonal reservoir for improved performance.

Constructed wetlands tested with various plants and different sized rocks. There were two main different types of wetlands. Type I wetlands which have three different sized rocks as layers performed better. Additional aeration also improved the performance of all wetland types.

Some parameters could not be measured properly due to major inabilities of conditions. These setups will be tested with different add-ons for another 2 years in which improved performance expected.

After all, there may be still some improvements which can be done to increase the performances of systems. Different feedstock those are abundant and economically convenient can be tried to monitor performance and overall gain. For wetland tests, some plants which may be used for trading can be tested for the refinement of post processing. Due to possible heat losses in anaerobic tanks, isolation can be done on tanks to mimic original conditions to create better environment for tests and monitor optimization factors, especially for colder seasons.

Pilot systems performance was better compared to existing full scale system, however it should be noted that experimental system was commissioned while the existing system were in use, hence some specific problems of the full scale treatment plants influenced the performance of the pilot system. If the pilot system was considered as a full system itself, its' performance would be far better.

There are 3500 municipalities all over Turkey and 2500 of them located in regions where the population ranged between 2000 and 10000. Similar systems as in this study can also be applied in Turkey for locations of such population ranges. Seasonal and environmental improvements can be necessary to optimize performances as well as economical considerations.

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RESUME

Gülsan SARAÇOĞLU was born in Istanbul in 1980. He graduated from Ataköy Cumhuriyet High School in 1997. In 1998 he started Sakarya University, Faculty of Civil Engineering, Environmental Engineering Department. After graduation, he had admission to Istanbul Technical University, Environmental Engineering Department. He started Environmental Science Programme in 2003.

APPENDIX

APPENDIX A

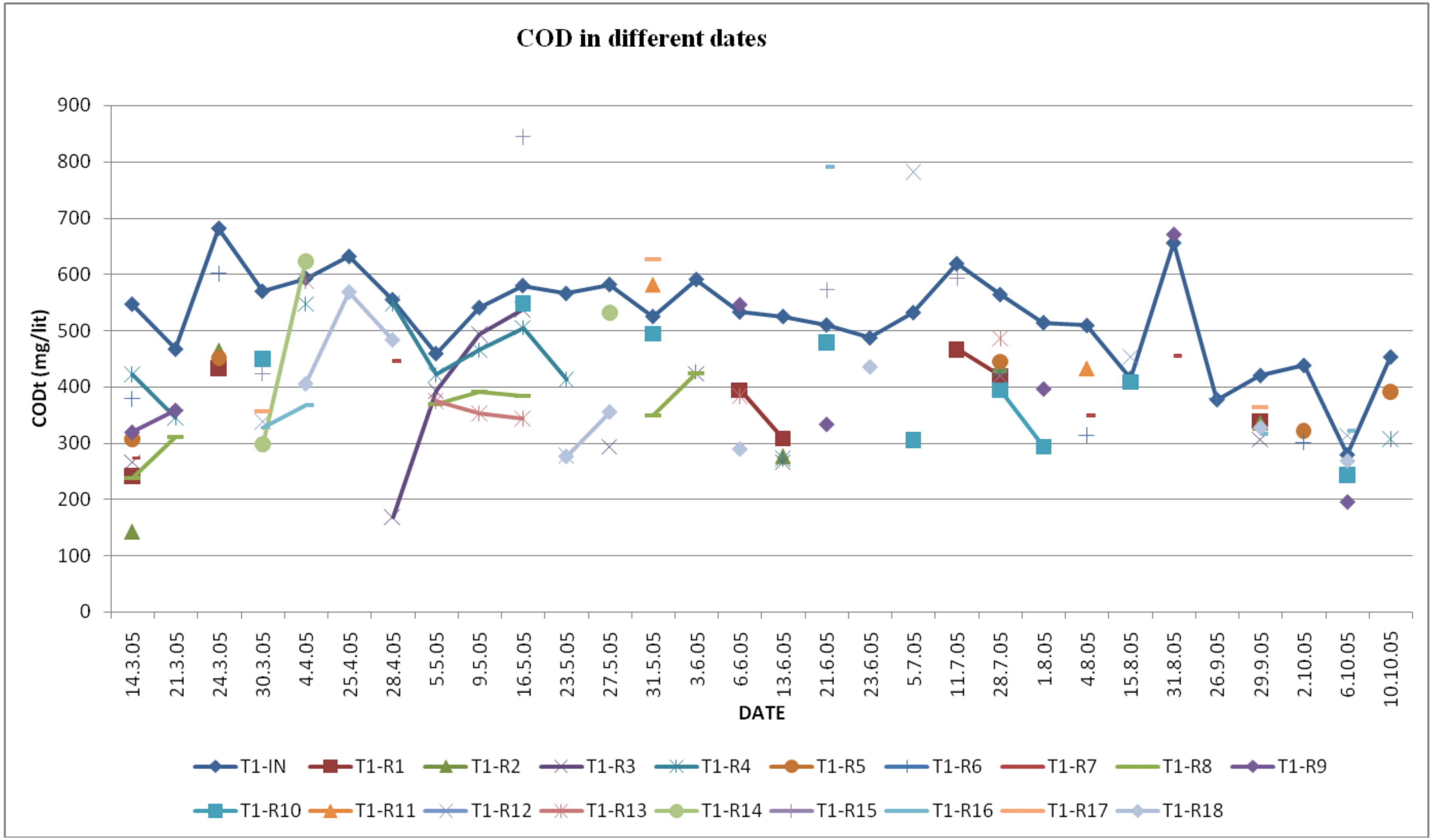

Figure A.1: TASK 1 - COD in different dates

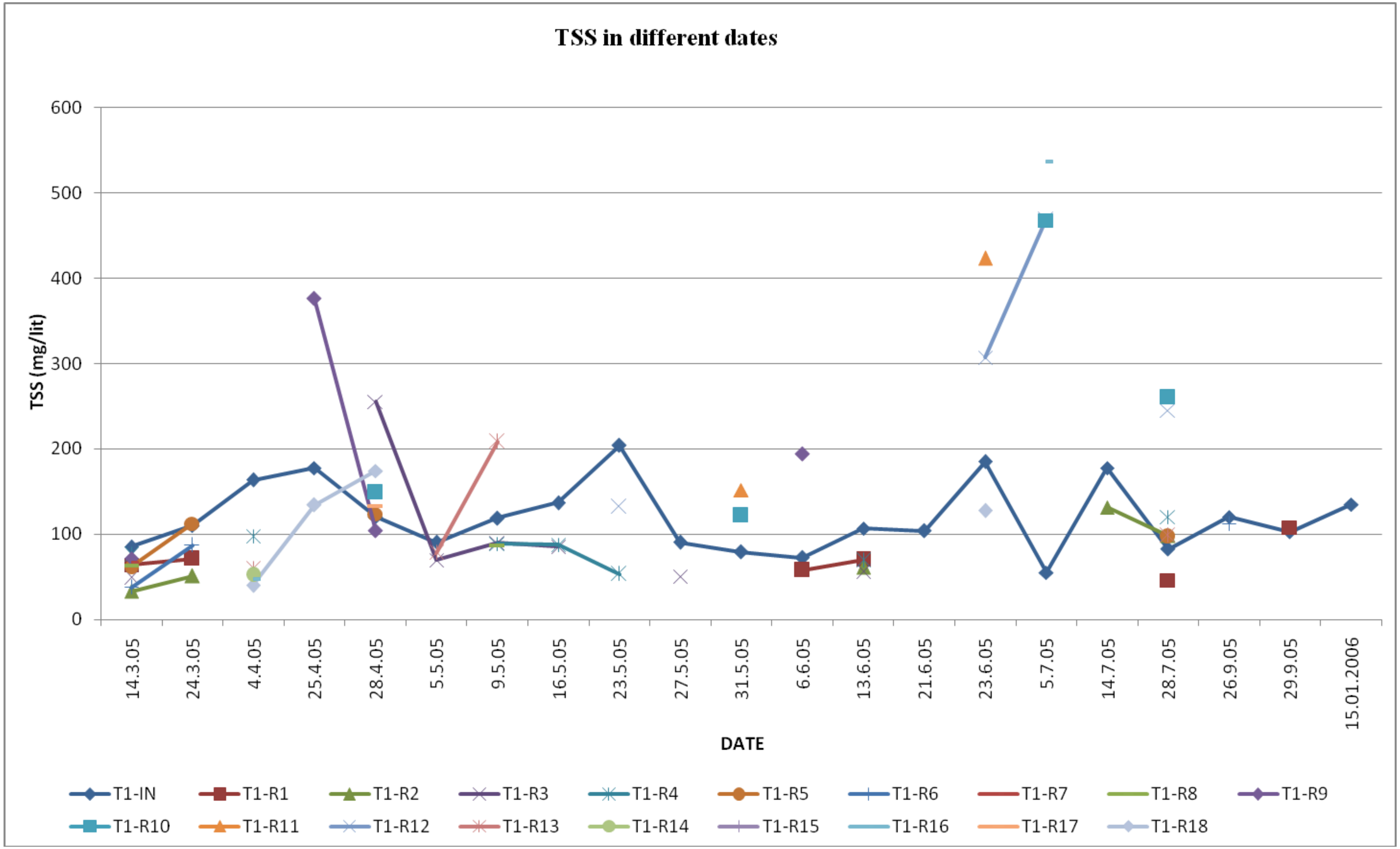

Figure A.2: TASK 1 - TSS (mg/l) in different dates

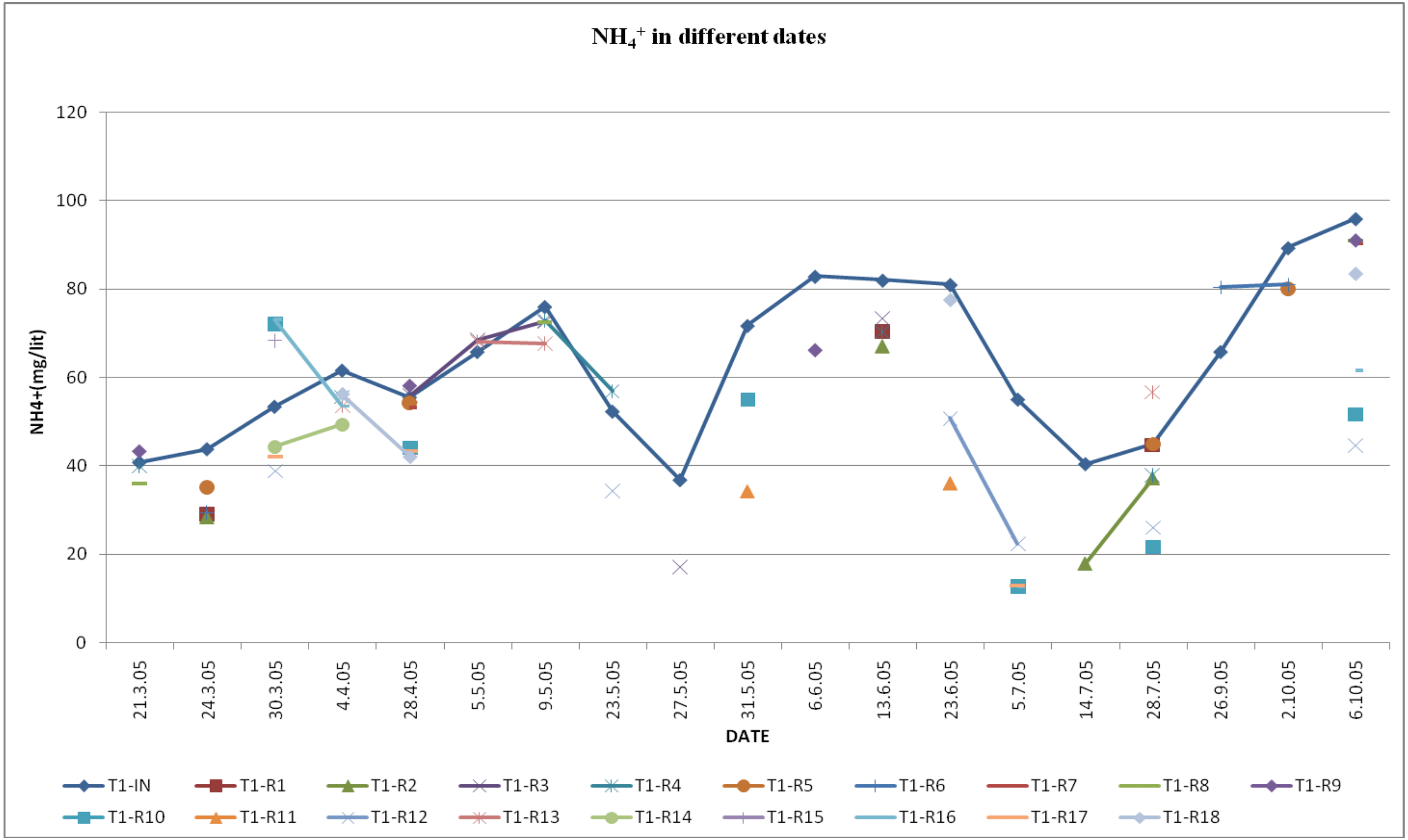

Figure A.3: TASK 1 - NH_4^+ in different dates

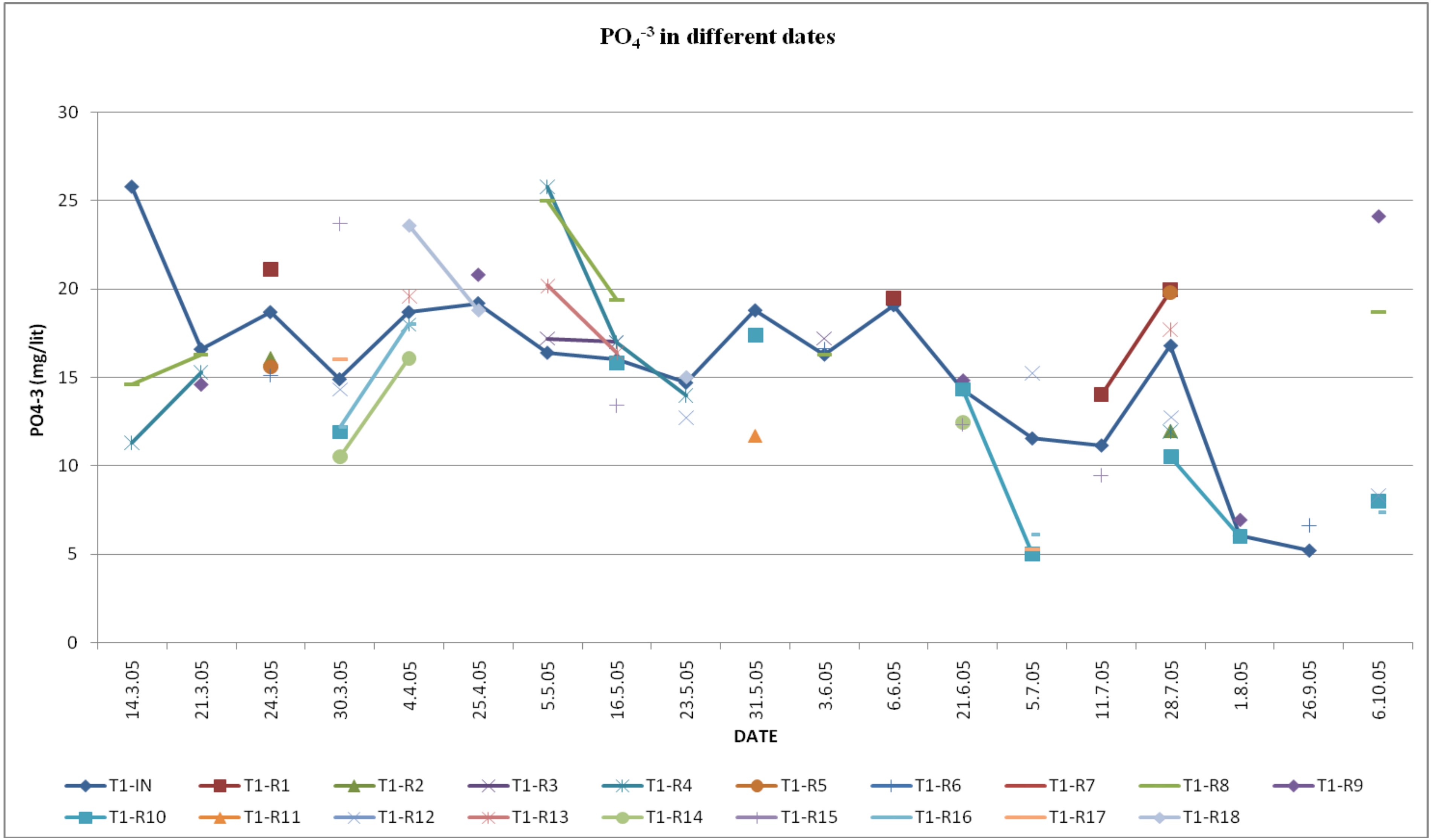

Figure A.4: TASK 1 - PO$_4^{-3}$ in different dates

APPENDIX B

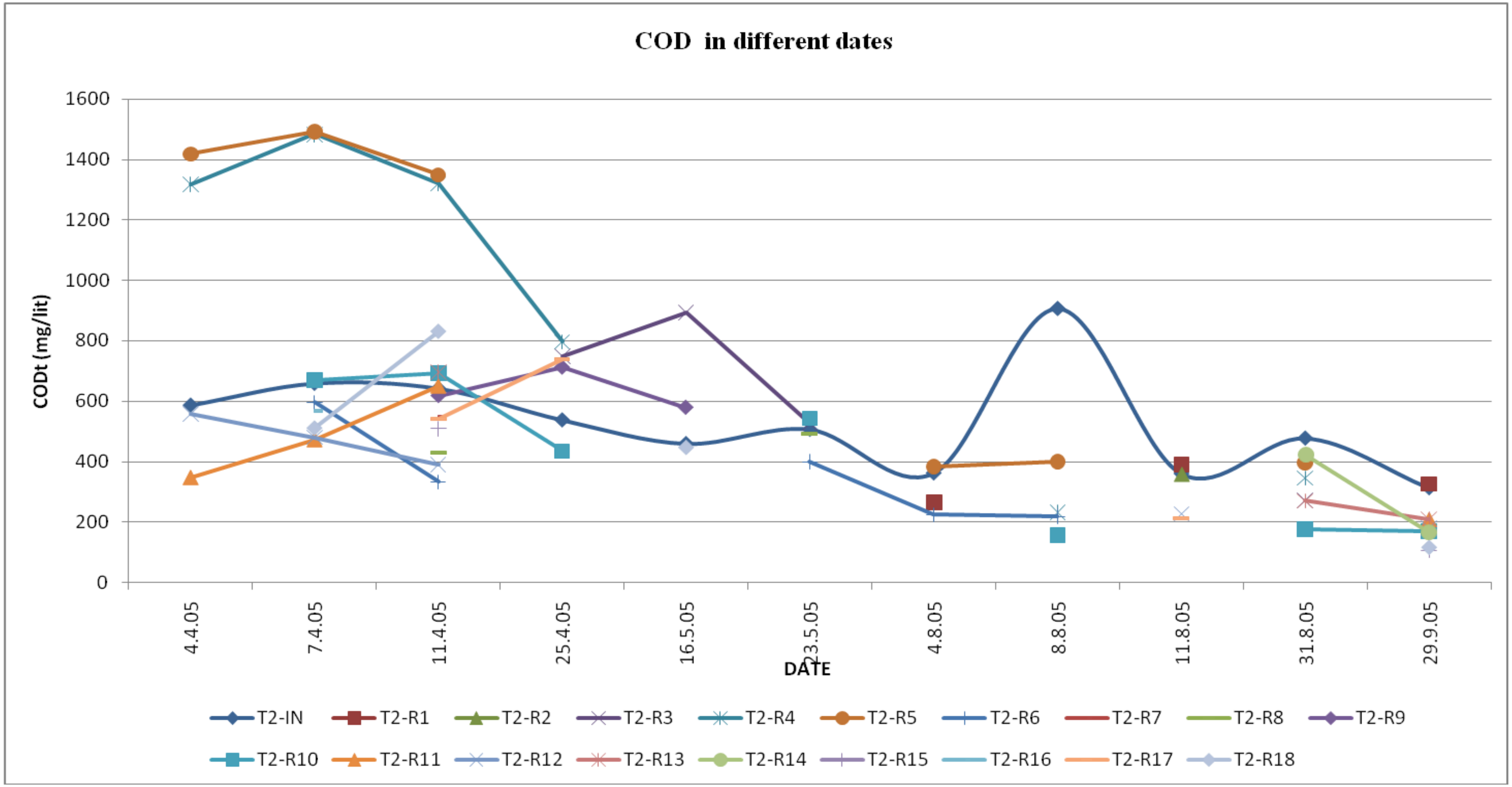

Figure B.1: TASK 2 - COD in different dates

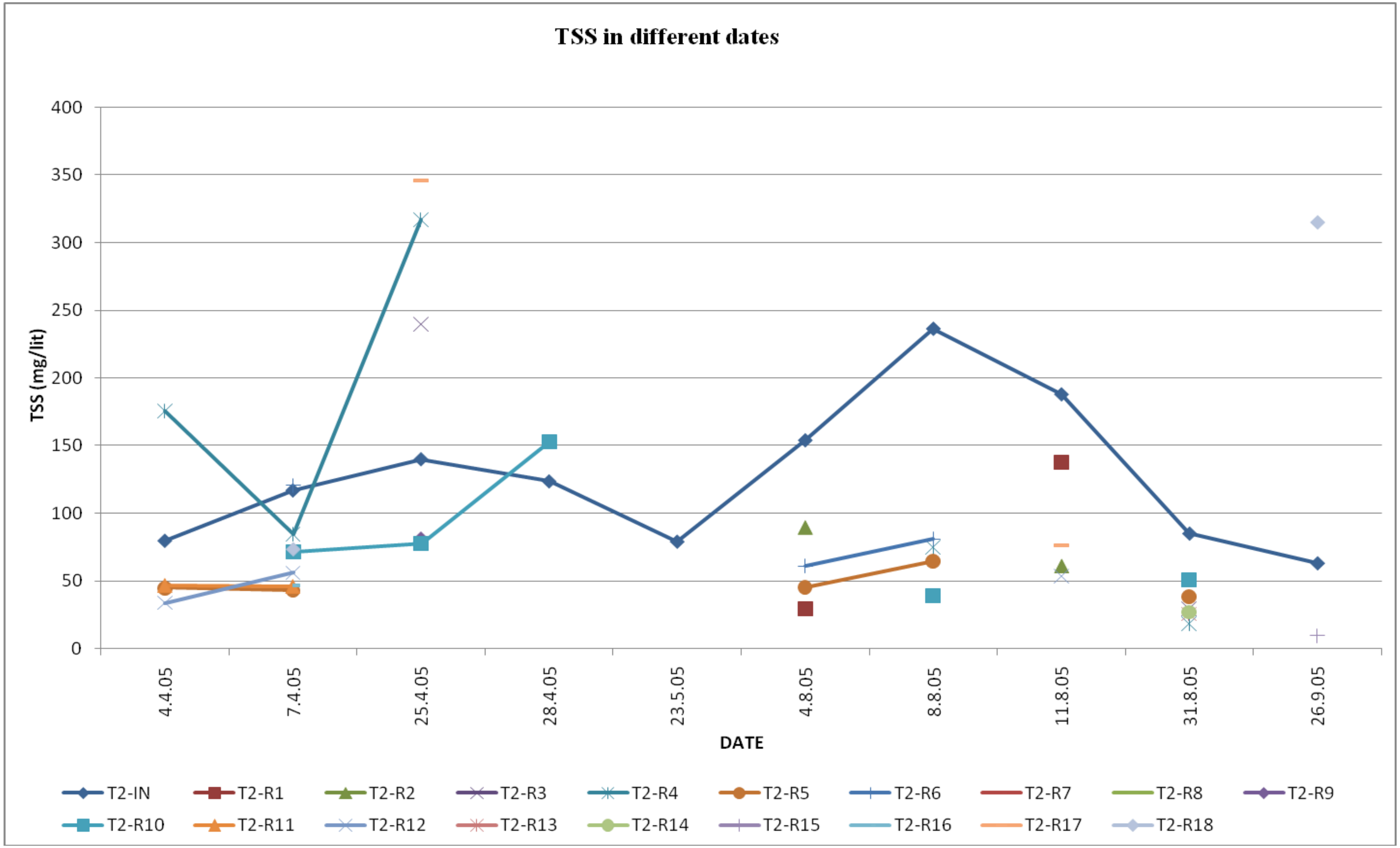

Figure B.2: TASK 2 - TSS in different dates

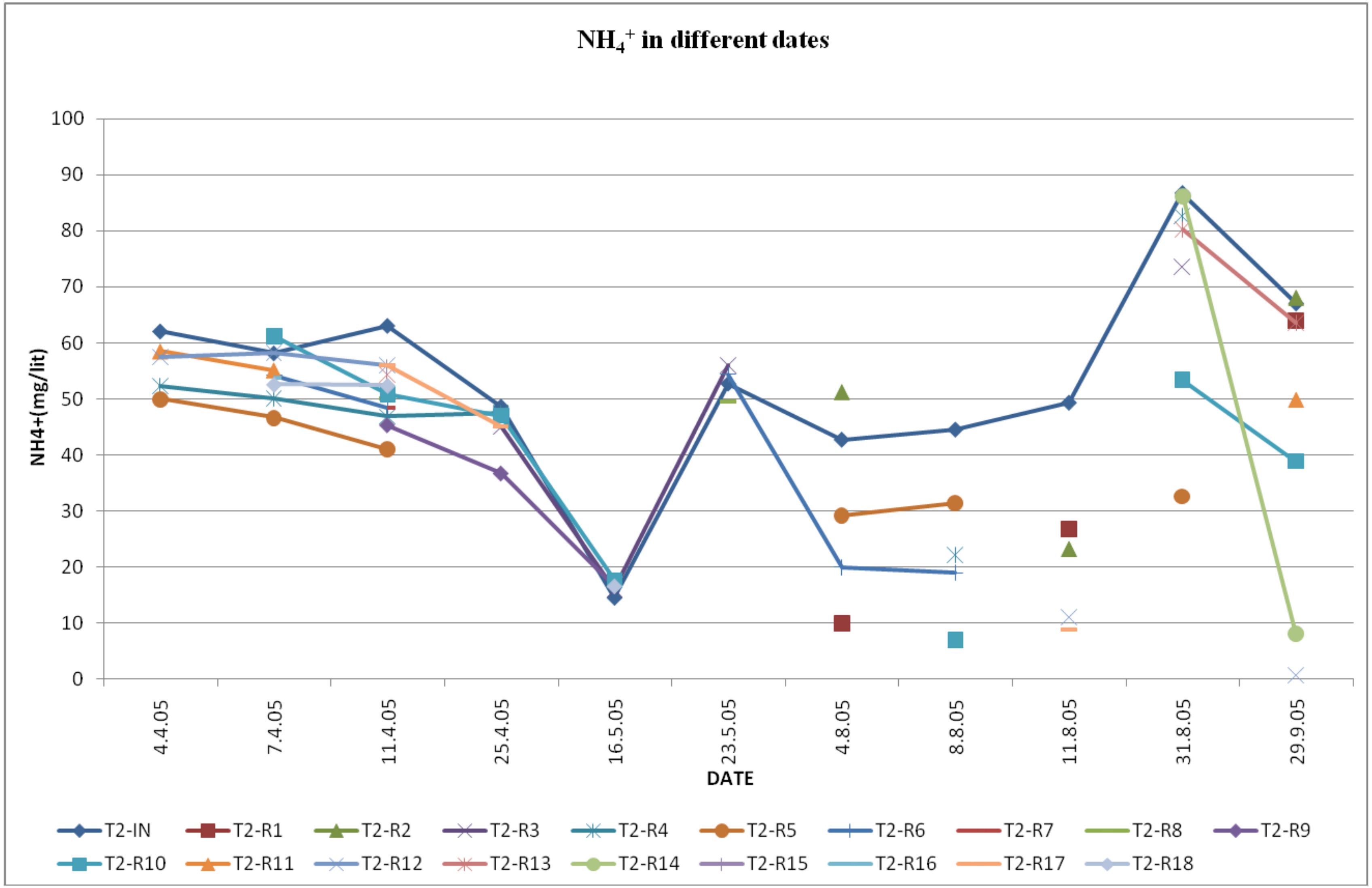

Figure B.3: TASK 2 - NH_4^+ in different dates

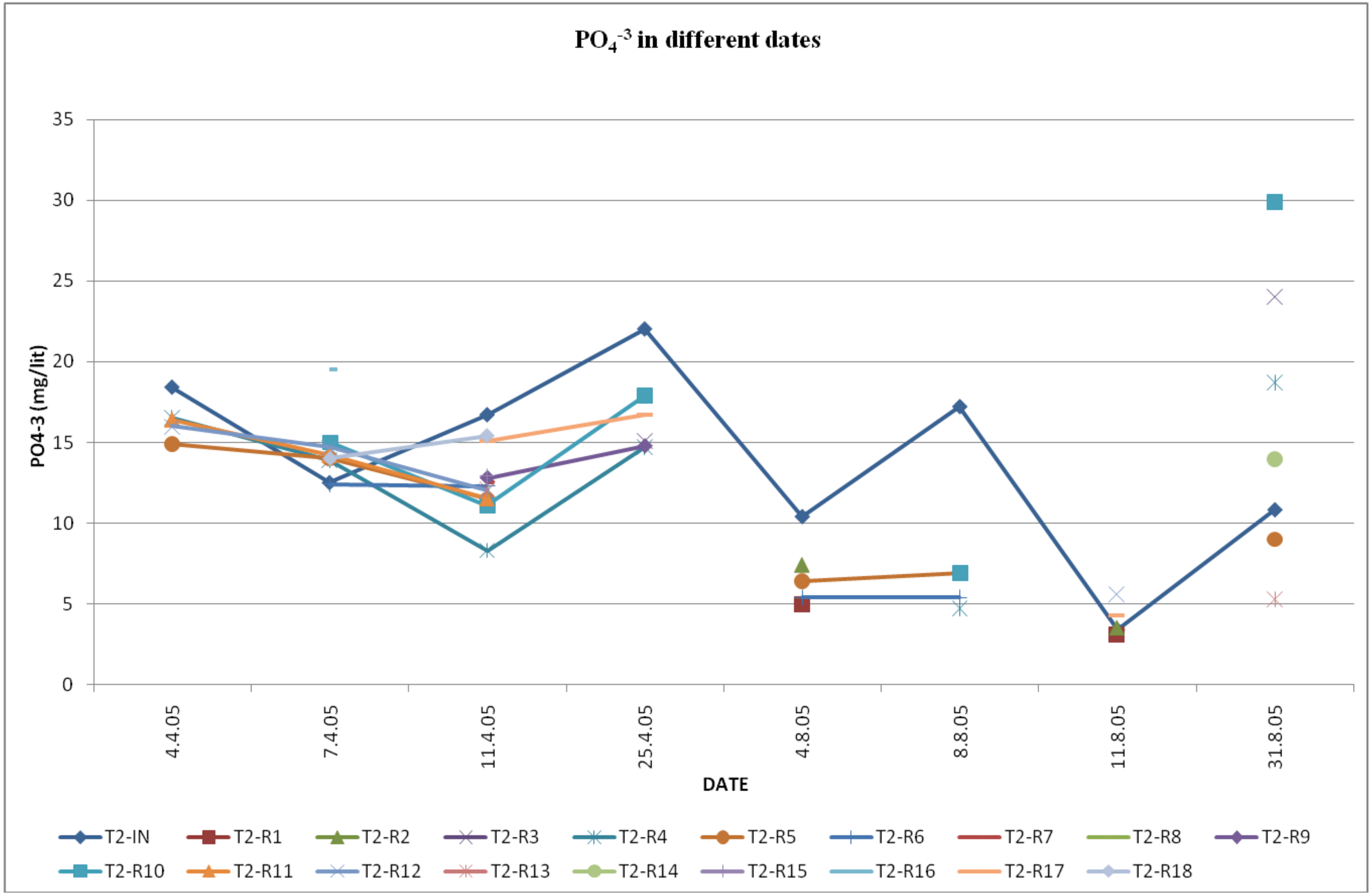

Figure B.4: TASK 2 - PO_4^{-3} in different dates

APPENDIX C

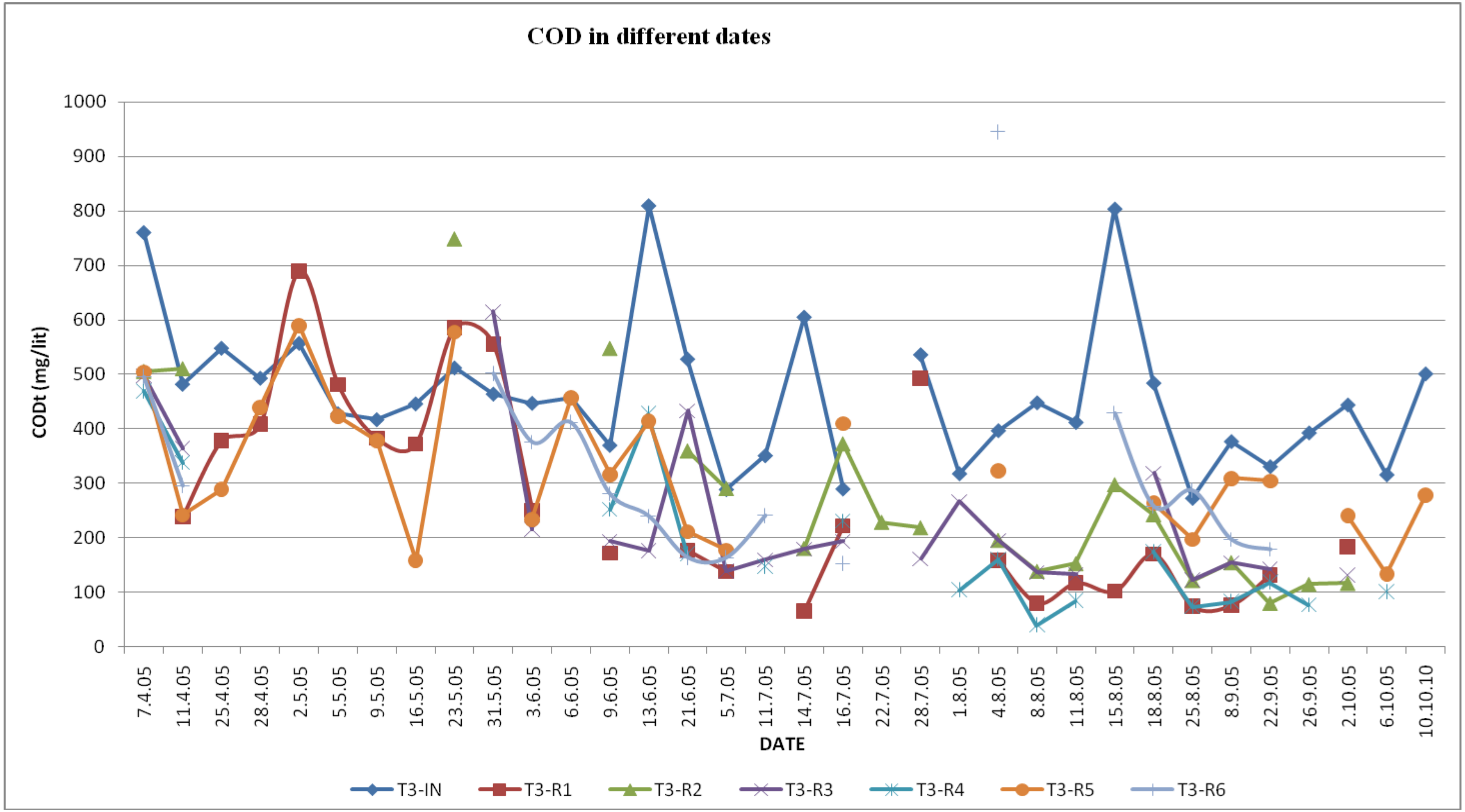

Figure C.1: TASK 3 - COD in different dates

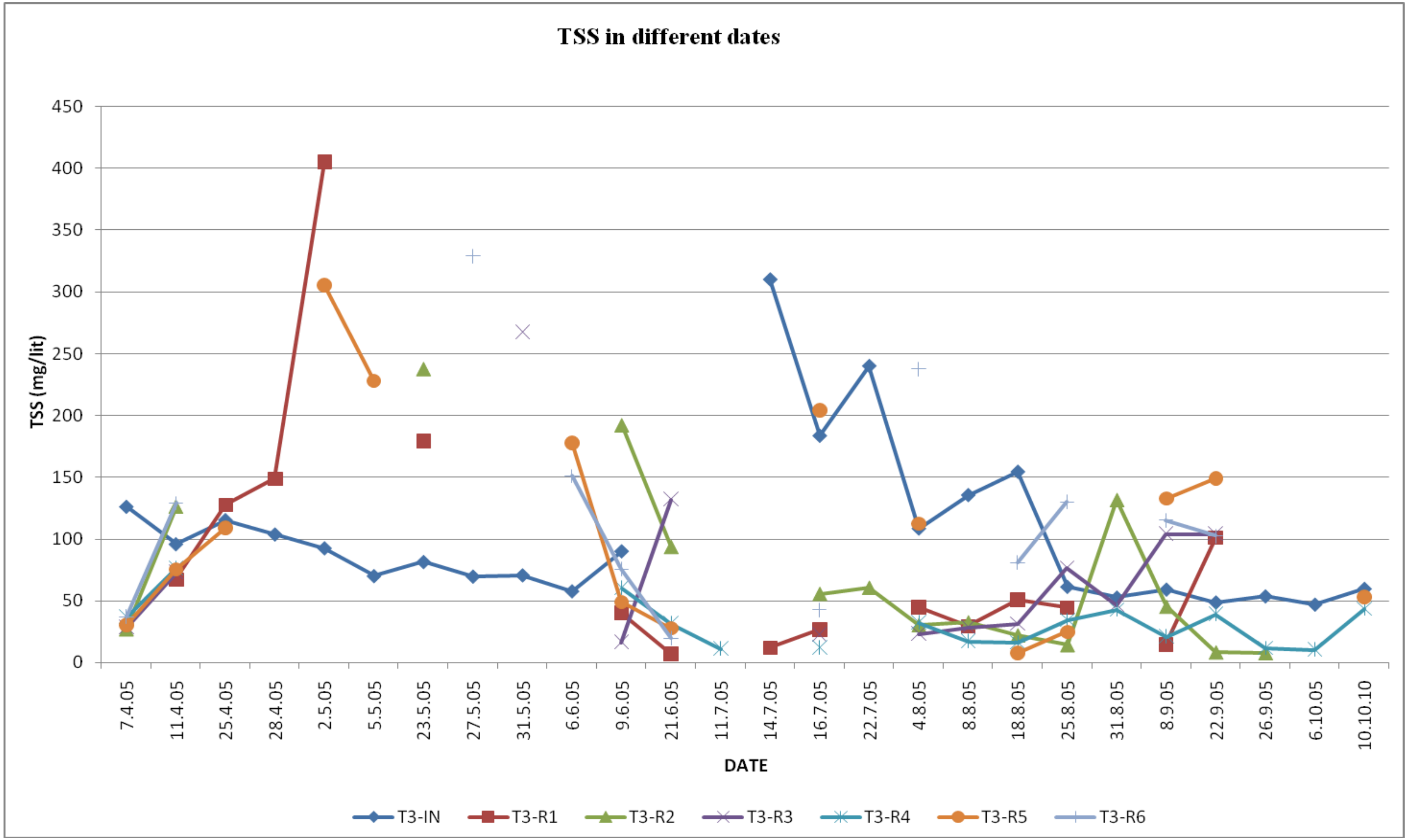

Figure C.2: TASK 3 - TSS in different dates

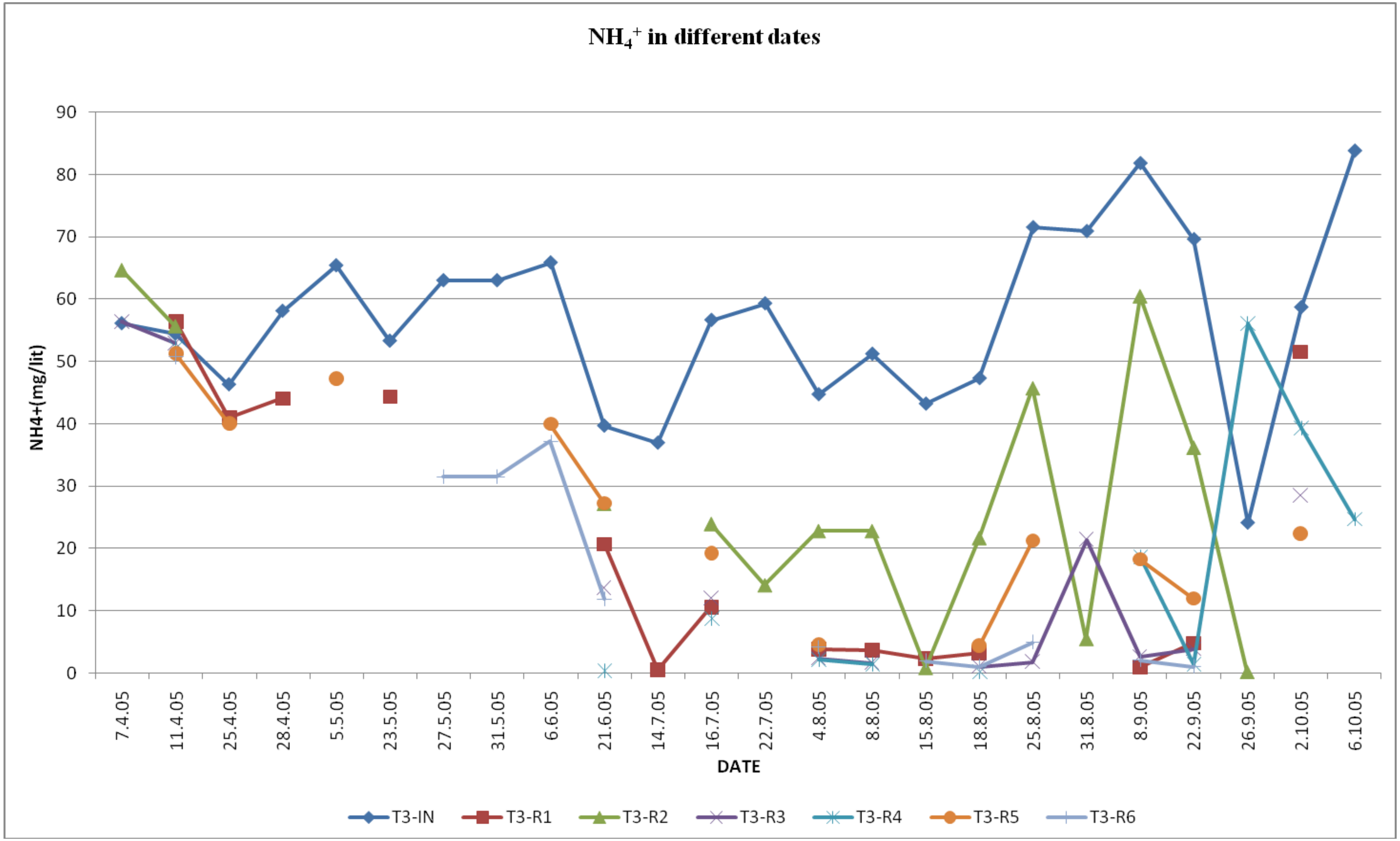

Figure C.3: TASK 3 - NH_4^+ in different dates

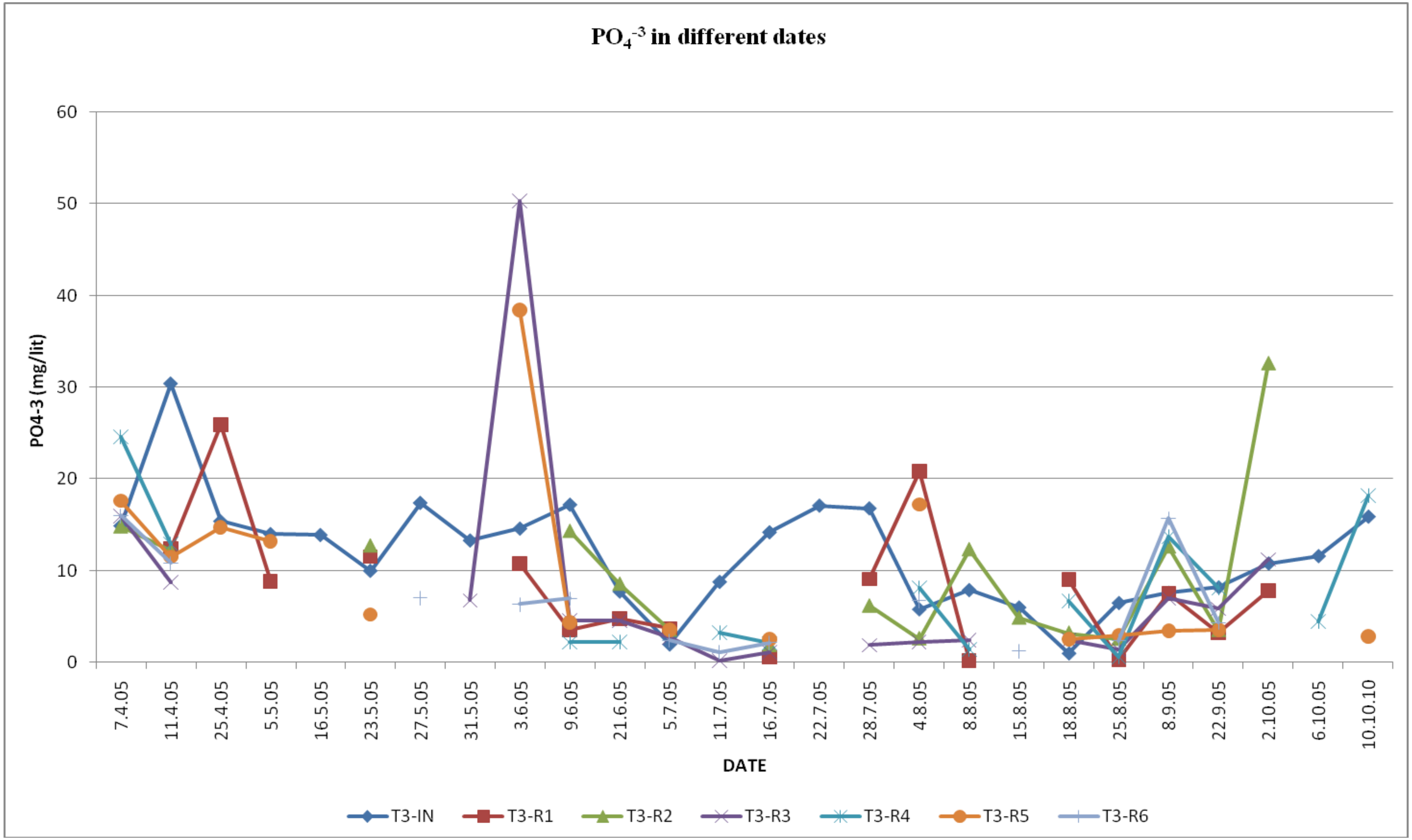

Figure C.4: TASK 3 - PO_4^{-3} in different dates

APPENDIX D

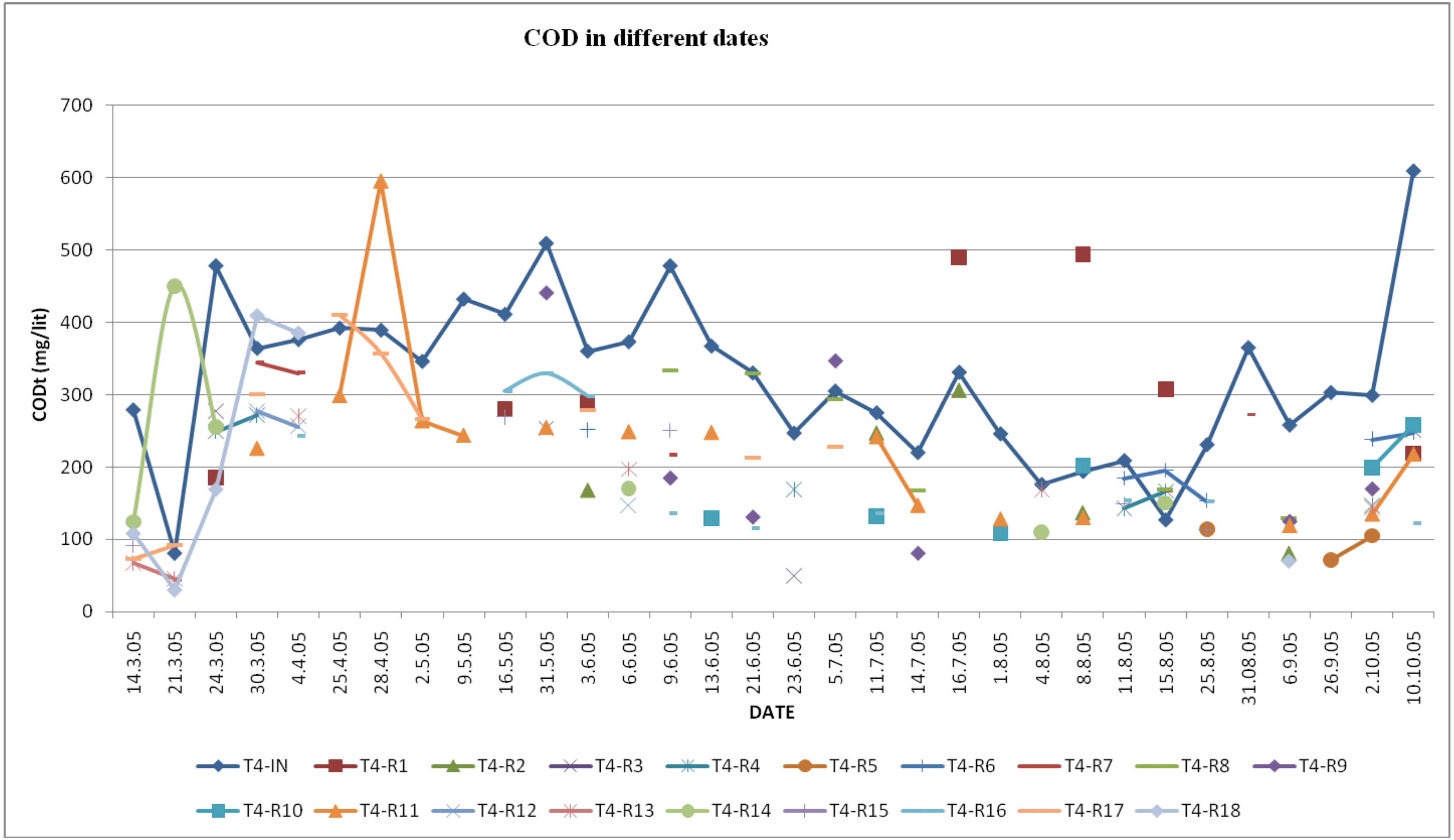

Figure D.1: TASK 4 - COD in different dates

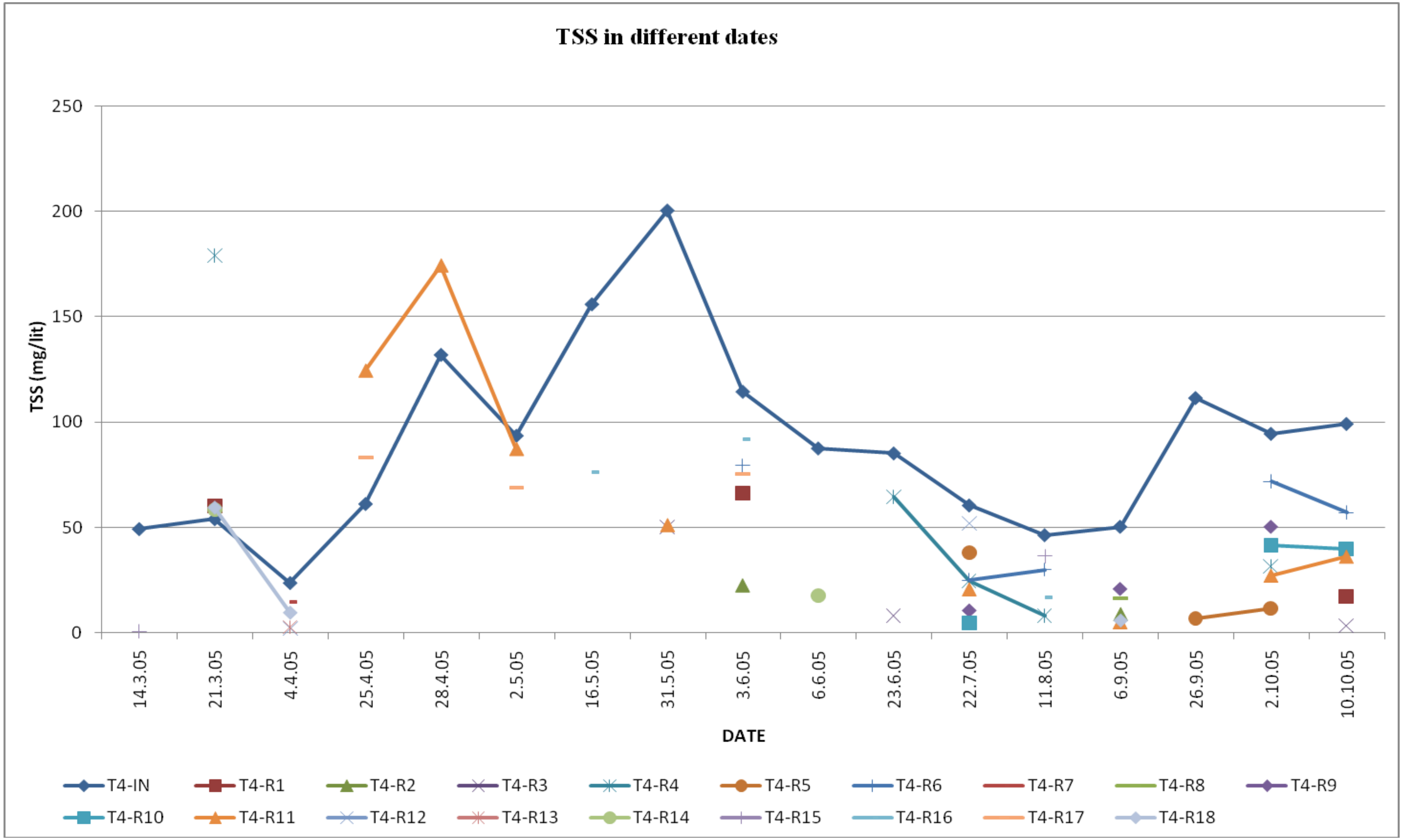

Figure D.2: TASK 4 - TSS in different dates

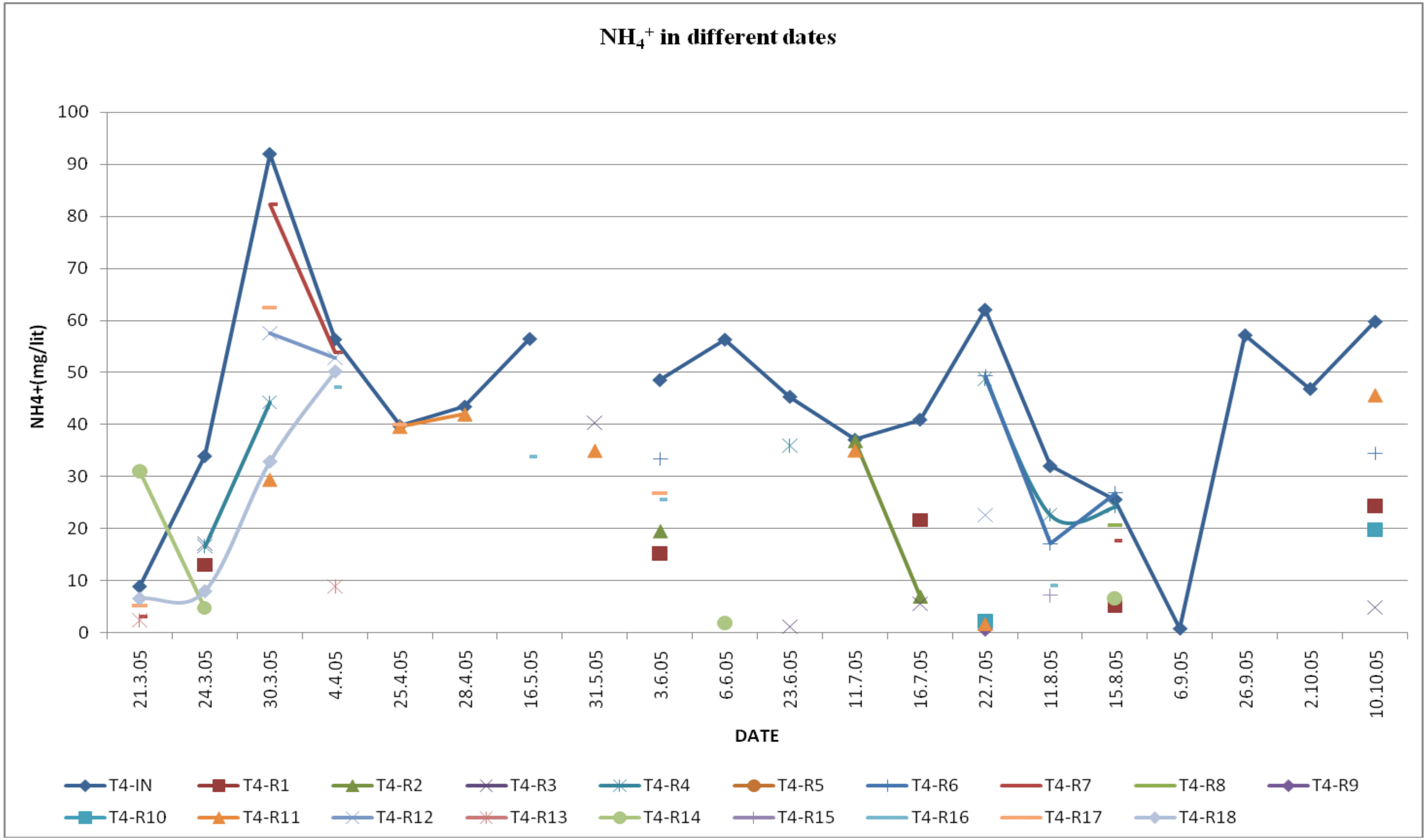

Figure D.3: TASK 4 - NH4+ in different dates

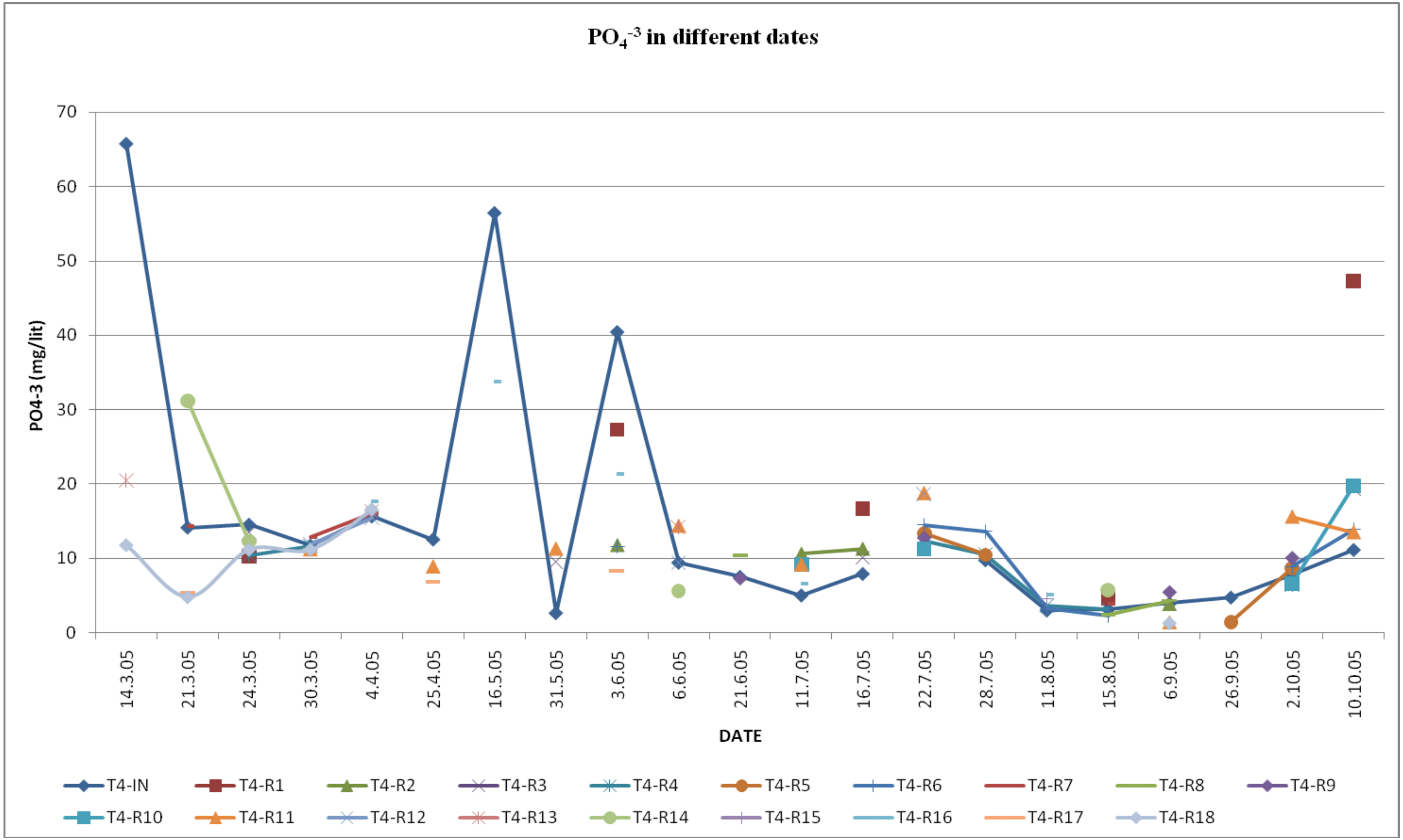

Figure D.4: TASK 4 - PO$_4^{-3}$ in different dates