PERFORMANCE EVALUATION OF OXIDATION BASED SEWAGE TREATMENT PLANTS

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By

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CERTIFICATE

This is to certify that the thesis report entitled "**PERFORMANCE EVALUATION OF OXIDATION BASED SEWAGE TREATMENT PLANTS**" submitted by Sachin Kumar in the partial fulfillment in the award of the degree of **Master of Technology in Environmental Science and Technology,** Thapar University, Patiala, is a record of student's own work carried out by him under my supervision and guidance. The report has not been submitted for the award of any other degree or certificate to this or any other institution.

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ABSTRACT

Oxidation ponds are recognized as the solution to domestic wastewater treatment in developing countries. The use of such natural systems is considered to be very important. This is because it is cheap, easy to construct and they do not require high skilled labor. In the developing countries the objectives for wastewater treatment should put emphasis on pathogen removal since most diseases and deaths in these areas are caused by poor sanitation. The efficiency in the removal of pathogens in oxidation ponds has been found to be very good. Oxidation ponds are mostly designed using empirical formulas derived either from past experience or from performance of already existing STPs elsewhere. Actual performance of the STP can differ from that of design mainly due to differences in sewage characteristics & local conditions. This study is an attempt in this direction. Samples were collected in month of February and March from two types of aeration pond systems one with laminar flow inversion technique and second with jet aerators. There were two ponds in series each type of aeration system. Sampling points were inlet to first pond(P1), two points along the length of first pond at one-third and two-third of length(P2,P3), Inlet to second pond(P4), two points along the length of second pond at one-third and two-third of length(P5,P6) and outlet to second pond(P7) for both type of aeration system. In the pond samples were taken and analyzed at three different depths. Treated effluent of the STP was found complying with the prescribed standards.

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INTRODUCTION

1.1. Background Information

Urbanization has encouraged the migration of people from villages to urban areas in India. With exponential growth in urbanization, a number of environmental problems have emerged. Water, like energy, will probably become the most critical natural resource issue facing most parts of the world. In the Middle East area, once referred to as the *Fertile Crescent*, precipitation is low, droughts are frequent and rivers are few. Countries in the region will experience serious water shortages, which have economic and military drawbacks, making water a national security issue. Many experts warn that disagreements over water have the potential to create political tensions and may result in wars, Thus, water will be a major issue that can catalyze the peace process or inhibit it. It has been said that if you want to save your children from poverty, pay attention to your water. Water scarcity could result from lack of supply or abuse and overuse of available water resources. Pollution from industry, urban wastewater, and agricultural run-off reduces the fitness of fresh water sources.

A steady decline in groundwater levels has been noticed where groundwater is used as the water source. Inappropriate irrigation practices increase the salinity and erosion of the soil, and result in higher sediment loads in watercourses. When industrial wastewater is discharged into surface water, or mixed with municipal wastewater, it creates serious environmental problems and diseases. Although conservation measures include the recycling of wastewater, along with nutrient recovery, uncontrolled wastewater irrigation practices may have major detrimental effects on the health of people who consume the irrigated edible crops, or to farmers who are directly exposed to these processes. In most cases, it is important to assure microbiological and virological safety.

Ambala Military Station is having oxidation pond/aerated lagoon based STPs for treating its sewage. Not satisfied with the performance, the military station has decided to revamp and improve the existing STPs in order to achieve consistent compliance with the applicable effluent discharge standards and reuse the treated effluent for irrigating lawns, plantations and Golf course; in fire hydrants, and in kitchen gardening, vehicle washing, and toilet flushing.

So, this M. Tech dissertation work is concerned with the analysis of sewage of existing STPs to comply with effluent discharge standards.

1.2. Objective of study

Objectives of the present study may be stated as following

1. Characterization and quantification of sewage
2. To carry out performance monitoring of STPs

1.3. Content of the study.

Present thesis work on performance evaluation and monitoring of Sewage Treatment Plant at Ambala Military Station includes altogether five chapters and a reference section.

Chapter - 1 is **"Introduction".** It provides brief background information of the study, explicitly states objectives of the study, brings into light the importance of the work, and provides overview of the contents of the report and limitations of the study are given in the end.

Chapter -2 is **"Literature Review".** This chapter presents the literature review on ponds based treatment system. Their treatment process, constituents units their design methods are discussed in this section.

Chapter -3 is **"Materials and Methods".** This chapter identifies the work elements of the present study and brings forth the approach followed for carrying out the work on identified work elements. Comprehensive detail of the STP in question along with a schematic flow diagram is also covered in this chapter. References to the analytical techniques used in the assessment associated with the study are also provided herein

Chapter –4 is **"Results and Discussion".** Results of the present study are covered in this chapter and discussed.

Chapter -5 is **"Summary and Conclusions".** This is the last chapter. It summarizes the outcomes of the present study and draws conclusions. It goes further to bring forth the limitations of the study and indicates what else can be done in the future studies.

Chapter 2
Literature Review

2.1. Sewage Characterization and Quantification

2.1.1. Types of sewage

Wastewater is domestic, municipal, or industrial liquid waste products. Depending on their origin, wastewaters can be classed as sanitary, commercial, industrial, or surface runoff. Sewage may be classified in following categories.

Sanitary Sewage: The spent water from residences and institutions, carrying body wastes, ablution water, food preparation wastes, laundry wastes, and other waste products of normal living, are classed as domestic or sanitary sewage.

Commercial wastes: Liquid-carried wastes from stores and service establishments serving the immediate community, termed commercial wastes, are included in the sanitary or domestic sewage category if their characteristics are similar to household flows.

Surface runoff: It is also known as storm flow or overland flow, is that portion of precipitation that runs rapidly over the ground surface to a defined channel. Precipitation absorbs gases and particulates from the atmosphere, dissolves and leaches materials from vegetation and soil, suspends matter from the land, washes spills and debris from urban streets and highways, and carries all these pollutants as wastes in its flow to a collection point. Discharges are classified as point-source when they emanate from a pipe outfall or non-point-source when they are diffused and come from agriculture or un-channeled urban land drainage runoff.

2.1.2. Typical characteristics of sewage

Sewage is characterized in the term of its physical, chemical, and biological composition. Many of these three properties are interrelated with each other. These can be described briefly as following.

Physical characteristics

The physical characteristics of wastewater include those items that can be detected using the physical senses. They are temperature, color, odor, and solids.

Temperature: The temperature of wastewater varies greatly, depending upon the type of operations being conducted at your installation. Wide variation in the wastewater temperature indicates heated or cooled discharges, often of substantial volume. They have any one of a number of sources. For example, decreased temperatures after a snowmelt or rainfall may indicate serious infiltration. Changes in wastewater temperatures affect the settling rates, dissolved oxygen levels, and biological action. The temperature of wastewater becomes extremely important in certain wastewater unit operations such as sedimentation tanks and recirculating filters.

Color: Historically colour was used to describe the condition of wastewater, which in turn refers to the age of water. Fresh wastewater is usually a light brownish gray color. The color of water changes from gray to dark gray and ultimately to black. When the colour of wastewater is black, the wastewater is described as septic. In most cases gray, dark gray and black color of wastewater is due to formation of metallic sulfides, which form as sulfide produced under anaerobic conditions reacts with the metals in the wastewater.

Solid: Wastewater is normally 99.9 percent water and 0.1 percent solids. If a wastewater sample is evaporated, the solids remaining are called total solids. The amount of solids in the drinking water system has a significant effect on the total solids concentration in the raw sewage. Industrial and domestic discharges also add solids to the plant influent. There are many different ways to classify solids. The most common types are dissolved, suspended, settle able, floatable, colloidal, organic, and inorganic solids. Part of the total solids is dissolved in wastewater. Much like sugar dissolves in coffee, many solids dissolve in water. Dissolved solids pass through a fine mesh filter. Normal wastewater processes using settling or flotation are designed to remove solids but cannot remove dissolved solids. Biological treatment units such as trickling filters and activated sludge plants convert some of these dissolved solids into settle able solids that are then removed by sedimentation tanks. Those solids that are not dissolved in wastewater are called suspended solids. When suspended solids float, they are called floatable solids or scum. Those suspended solids that settle are called settle able solids, grit, or sludge. Very small suspended solids that neither float nor settle are called colloidal particles. Colloidal particles are often removed in the biological treatment units. They may also be removed by chemical treatment followed by sedimentation. All the solids discussed above may be either organic or inorganic. Organic solids always contain carbon and hydrogen and when ignited to high temperatures (500°C to 600°C) burn to form carbon dioxide, water, and sometimes various other compounds. The burning or volatilization of organic solids has led to the term volatile solids. All solids that burn or evaporate at 500°C to 600°C are called volatile solids. These solids serve as a food source for bacteria and other living forms in a wastewater treatment plant. Most organic solids in municipal waste originate from living plants or animals.

Turbidity: It is a measure of light transmitting properties of water, is another test to indicate the quality of waste discharge and natural water with respect to colloidal and residual suspended matter. The measurement of turbidity is based on the comparison of the intensity of light scattered by a reference suspension under the same conditions

Conductivity: The electrical conductivity of water is a measure of the ability of a solution to conduct an electrical current. Because the electrical current is transported by the ions in solution, the conductivity increases as the concentration of ions increases. The measured EC value is used as surrogate measure of total dissolved solids concentration. Now a day's salinity of treated wastewater to be used for irrigation is estimated by measuring its electrical conductivities.

Chemical Characteristics

pH : The term pH is used to describe the acid or base properties of water solutions. A scale from0 to 14 has been established where pH value of 7 is neutral. A pH value less than 7 is acidic. A pH value above 7 is alkaline or basic. A pH value less than 7 in the wastewater plant influent may indicate septic conditions of wastewater. The pH values less than 5 and more than 10 usually indicate that industrial wastes exist and are not compatible with biological wastewater operations. Pretreatment of these wastes at the source is usually required since extreme pH values may damage biological treatment units.

Dissolved Oxygen: Dissolved oxygen in wastewater has a great effect on treatment of wastewater. Wastewater that has DO is called aerobic or fresh. Aerobic raw sewage is usually gray in color and has a musty odor. Wastewater that has no DO is called anaerobic or septic. Anaerobic raw sewage is usually black and has an offensive hydrogen sulfide or rotten egg odor.

Oxygen Demand: Oxygen demand is the amount of oxygen used by bacteria and other wastewater organisms as they feed upon the organic solids in the wastewater. Chemical tests such as the BOD (biochemical oxygen demand), the COD (chemical oxygen demand), the ODI(instantaneous oxygen demand or oxygen demand index), and the TOC (total organic carbon) measure the "strength" of sewage. It is important that organic wastes be removed to protect the receiving body of water into which the wastewater plant is discharging.

Alkalinity: Alkalinity in wastewater results from the presence of the hydroxides, carbonates and bicarbonates of elements such as calcium, sodium, potassium, and ammonia. Borate, silicates, phosphates and similar compounds can also contribute to the alkalinity (APHA). The concentration of alkalinity in wastewater is important where chemical and biological treatment is to be used, in biological nutrient removal, and where ammonia is to be removed by air stripping.

Nutrients: Nutrients are life-supporting nitrogen and phosphorus. They stimulate excessive growths of algae and other aquatic plant life. They are always present in domestic wastewaters and are not removed during conventional primary and secondary treatment.

Removal is accomplished by processes in addition to normal wastewater treatment or tertiary treatment, when specific reuse requirements require it.

Chlorides: Chloride is a constituent of concern in wastewaters it can impact the final reuse application of treated wastewater. Chlorides in natural water results from the leaching of chloride containing rocks and soils with which the water comes in contact. Conventional methods of waste treatment do not remove chloride to any significant extent, higher than usual chloride concentration can be taken as an indication that a body of water is being used for waste disposal.

Toxic Chemicals: Most military and industrial installations use various types of toxic chemicals, the discharges of which can be harmful to wastewater treatment processes. These toxic chemicals should be pretreated or removed before the waste-water enters the collecting system.

Biological characteristics

There biological organisms present in wastewater are bacteria, virus and parasites. These are mainly disease causing agent.

Bacteria: Sewage consists of vast quantities of bacteria, most of which are harmless to man. However, pathogenic (disease-causing) organisms such as typhoid, dysentery, and other intestinal disorders may be present in wastewater. Tests for total coliform and fecal coliform nonpathogenic bacteria are used to indicate the presence of pathogenic bacteria. Because it is easier to test for coliforms, fecal coliform testing has been accepted as the best indicator of fecal contamination. Fecal coliform counts of 100 million per 100 milliliters may be found in raw domestic sewage. Detectable health effects have been found at levels of 2,300 to 2,400 total coliforms per 100 milliliters in recreational waters. Disinfection, usually chlorination, is generally used to reduce these pathogens. Breakdown or malfunctions of chlorination equipment will probably result in excessive discharge of pathogenic organisms and can seriously affect public health. Bacteria can also be classified according to their dissolved oxygen requirement. Aerobic bacteria are bacteria that require dissolved oxygen to live. Anaerobic bacteria cannot live if dissolved oxygen is present. Facultative bacteria can live with or without dissolved oxygen.

Viruses: Wastewater often contains viruses that may produce diseases. Outbreaks of infectious hepatitis have been traced through water systems because of wastewater entering the supply, sedimentation, filtration and disinfection, if used efficiently, usually provide acceptable virus removal.

Parasites: There are also many species of parasites carried by wastewater. The life cycle of each is peculiar to the given parasite. Some are dangerous to man and livestock, particularly during certain stages of the life cycle. Amoebic dysentery is a common disease caused by amoebic parasites. Chlorination, chemical precipitation, sedimentation, or sand filtration is used to ensure protection against parasites.

2.1.3. Sewage sampling

Samples of sewage are taken to find out how well a treatment plant is working and what operating changes may need to be made, Some samples show how much the plant is reducing pollutants like BOD, solids, and so forth. Raw sewage entering the plant must be tested as well as the effluent from the plant and the receiving stream above and below the discharge point to determine how well the plant is removing pollutants. Since wastewater flows often change a great deal, daily sampling is suggested.

Representative sampling: A sample should be taken in a way that will represent the wastewater being treated. No matter how good the lab analysis is, if the sample was not correctly collected, the lab data will not be correct. With the large changes in composition and flow rate, getting a representative sample can be very hard. Careful thought, planning, and training must be used to develop and carry out a good sampling program. Samples may be taken by hand or automatically. Taking samples by hand may be as simple as tying an open bottle to a pole that can be lowered into the wastewater. The automatic samplers may be made by the operator or bought.

Grab sampling: A grab sample is a single sample of wastewater taken over a short span of time, usually less than 15 minutes. This type of sample yields data about the wastewater at one time and place. The grab sample should be used where the wastewater does not change suddenly or change a great deal. For example, grab samples may be used to determine pH and temperature. Grab samples are also used when a batch dump or sludge discharge is seen.

Composite sampling: A composite sample yields data about the wastewater over a longer span of time. A series of grab samples may be taken over a certain amount of time and combined to form a composite sample. These samples should show the time and frequency of the sample; for example, an 8-hour composite of 30-minute grab samples. The composite sample is used to find BOD, COD, suspended solids, and nutrients.

Flow proportional samples: The composite may be flow proportional. For this type of sample, the volume of the sample changes in proportion to the flow. The flow proportional composite sample is most often run for 24 hours with a 2-hour interval between each collection. To collect this kind of sample, the volume needed for the tests and the average daily flow for the plant must be known. The following formula may be used to find the volume of sample to be taken at each interval.

$$\text{Litres required} = \frac{\text{Flow at sampling time} \times \text{Total sample size}}{\text{Average flow} \times \text{No. of samples}}$$

2.1.4. Flow measurement techniques

This section familiarizes the field investigator with the most commonly used methods for wastewater flow measurements and the primary devices that will be encountered during field studies. The following methods are included only to enable the field investigator to make accurate flow estimates when necessary.

Volumetric

Volumetric flow measurement techniques are among the simplest and most accurate methods for measuring flow. These techniques basically involve the measurement of volume and/or the measurement of time required to fill a container of known size.

Vessel Volume: Vessel volume is used to obtain flow data particularly applicable to batch wastewater discharges. An accurate measurement of the vessel volume and the frequency at which it empties is all that is required.

Sump Pump: This measurement is made by observing the sump levels when the pumps cut on and off and calculating the volume contained between the two levels. This volume, along with the number of pump cycles, will give a good estimate of the daily wastewater flow. The inspector must also account for the quantity of wastewater that flows into the sump during the pumping cycle.

Bucket and Stop Watch: The bucket and stop watch technique is particularly useful for the measurement of small wastewater flows. It is simple to use. The only equipment required to make this measurement is a calibrated container (bucket, drum, tank, etc.) and a stop watch. A minimum of 10 seconds to fill the container is recommended. Three consecutive measurements should be made, and the results should be averaged.

Dilution Methods: Dilution methods for water and wastewater flow measurements are based on the color, conductivity, fluorescence, or other quantifiable property of an injected tracer. The dilution methods require specialized equipment, special attention to detail by the investigator, and are time consuming.

Open Channel Flow Measurements

Measurement of wastewater flow in open channels is the most frequently encountered situation during field investigations. An open channel is defined as any open conduit, such as a channel or flume, or any closed conduit, such as a pipe, which is not flowing full. The most commonly encountered methods in measuring open channel wastewater flows are described in this section.

Weirs: A weir is defined as an overflow structure built according to specific design standards across an open channel to measure the flow of water. Equations can be derived for weirs of specific geometry which relate static head to water flow. Weirs are classified into two general categories, broad crested and sharp crested. Broad crested weirs can only be used to calculate instantaneous flows. Sharp crested weirs are constructed in a wide variety of shapes and the most commonly encountered are V-notch, rectangular, and Cipolletti weirs. They are considered standard primary flow devices. All weirs should be inspected to determine and provide a uniform influent flow distribution, and that the weir is placed squarely across the channel perpendicular to the direction of flow. Useful tools for checking weir construction and installation include a carpenter's level, a framing square, a measuring tape, a staff gage, or surveyor’s level and rod.

Flumes: There are several types of flumes (e.g., Palmer-Bowlus, Cutthroat, and Trapezoidal) but the most widely used is the Parshall flume. The Parshall flume is considered a standard primary flow device when constructed and installed. All flumes should be inspected to determine if entrance conditions provide a uniform influent flow distribution, the floor of the flume at the throat section is level, and the throat section walls are vertical. Useful tools for checking the construction and installation of Parshall and other flumes include a carpenter's level, a framing square, and a measuring tape. The flume should be closely examined to determine if it is discharging freely. If there is any question about free discharge, the downstream head should be measured and compared to the head at the proper location in the converging section. A staff gage is useful for making head measurements.

Open Flow Nozzles: Open flow nozzles such as parabolic or Kennison nozzles are factory calibrated and are ordinarily supplied as part of a flow measurement system. Calibration and installation information for each nozzle should be supplied by or obtained from the manufacturer. The accuracy of these devices is reported to be often better than + 5 percent of the indicated flow. A volumetric flow measurement may be used to check accuracy of this device if flow volumes are not excessive.

Velocity-Area Method: The basic principal of this method is that the flow in a channel (cubic meter/second) is equal to the average velocity (meter/second) times the cross sectional area (square meter) of the channel. There are two methods for determining flow using the area velocity method. The first method uses an area-velocity flow meter in which the probe senses velocity and water depth and converts these readings to a flow rate. In the second method, the velocity of the water or wastewater is determined with an Current Flow meter (which can also calculate cross-sectional area changes) or a current meter. The area of the channel is either measured or calculated using an approximation technique.

Closed Conduit Flow Measurements

The accuracy of closed conduit flow measuring devices may be difficult to verify. However, the accuracy can be checked by making an independent flow measurement. Two of the available procedures are the Instrument Direct F-100-902 Doppler Ultrasonic flow meter and a dilution technique.

Venturi Meter: The Venturi meter employs a conversion of static head to velocity head whereby a differential is created that is proportional to flow (Roger C. Baker). The typical accuracy of a Venturi meter is at 1 to 2 percent.

Orifice Meter: The orifice meter is a pressure differential device that measures flow by the difference in static head. They can be accurate, e.g., within 0.5 percent, although their usable range is limited.

Flow Nozzle: The basic principle of operation is the same as that of the Venturi meter. The flow nozzle has an entrance section and a throat, but lacks the diverging section of the Venturi meter. Flow nozzle accuracies can approach those of Venturi meters.

Electromagnetic Flow Meter: The electromagnetic flow meter operates according to Faraday's Law of Induction where the conductor is the liquid stream, and the field is produced by a set of electromagnetic coils. The accuracy of the device is within +1 percent of full scale.

2.2. Oxidation Ponds

2.2.1. Introduction

These are the ponds (0.5 to 1.2 m) where aerobic conditions are predominant. Based on the depth oxidation ponds are categorized as shallow ponds and aerated ponds. In the shallow ponds oxygen is supplied by natural surface reaeration and by algal photosynthesis. In the aerated ponds mechanical or diffuser aerators or some other methods of aeration supplement the natural reaeration and algal photosynthesis. These ponds contain aerobic bacteria and algae in suspension which cause the degradation of organic matter and reaeration of ponds.

2.2.2. Shallow ponds

These ponds obtain oxygen from algal photosynthesis and by natural surface reaeration. BOD removal efficiency in these ponds under normal condition is 80 to 95% and may be lesser during winter months. (Pano and Middlebrooks, 1982;). Shallower depths (<1.0 m) encourage growth of rooted aquatic plants and greater depth may interfere with the mixing and oxygenation from the surface. Very shallow depth of aerobic pond (depth 0.15 to 0.45 m) is used for the treatment of irrigation return water or any other industrial sewage where the aim is the removal of nitrogen by algal growth. However, for the treatment of domestic sewage the depth is kept between 1 to 1.2 m. Length to width ratio usually depends on the geometry of the land available but efforts are to maintain the aspect ratio > 3:1 in order to prevent short circuiting.

The inlet and outlet structures are so located that the entire pond volume is effectively utilized. Contents of the pond may be stirred occasionally to prevent anaerobic conditions in the bottom settled sludge.

Facultative Ponds

Facultative ponds (1-2 m deep) are of two types: Primary facultative ponds that receive raw wastewater, and secondary facultative ponds that receive particle-free wastewater (usually from anaerobic ponds, septic tanks, primary facultative ponds, and shallow sewerage systems). The process of oxidation of organic matter by aerobic bacteria is usually dominant in primary facultative ponds or secondary facultative ponds. In facultative ponds the wastewater is profiled to include a top aerobic zone, a middle facultative zone and a bottom anaerobic zone. That is, wastewater treatment in facultative ponds takes the advantage of both anaerobic and aerobic processes. For ensuring aerobic conditions and higher algal activity, organic loading of the facultative ponds should be within limits.

While primary facultative ponds are designed and used for the removal of both suspended solids and BOD/COD, secondary facultative ponds are designed and used for the removal of only BOD/COD. However, even nutrient and pathogen removal occurs coincidentally in the facultative ponds (Knorr and Torella, 1995).

These ponds are shallow (1-2 m deep) and have relatively higher aspect ratios (up to 10:1). Increasing aspect ratios were reported to influence and upgrade the pond's performance. But, increasing depths were found not affecting the pond's performance (Pearson, Mara, and Arridge1995). Higher aspect ratios must be reducing the short-circuiting problems. Use of higher aspect ratios is possible especially in secondary facultative ponds, where sludge accumulation around the inlet is not a serious problem (Mara and Pearson 1998). Surface area exposed to air and solar radiation is apparently very important in the facultative ponds.

Maturation ponds

These ponds used in series with facultative ponds are usually 1 -1.5m deep and are geometrically designed to have a high length-to-width ratio (up to 10:1) to simulate a hydraulic plug flow regime (Mara et al., 1992). The primary function of maturation pond is to remove excreted pathogens to enable the practice of unrestricted crop irrigation (WHO, 1989). Maturation ponds achieve only a small removal of BOD, but their contribution to nutrient (nitrogen and phosphorous) removal is significant. The size and number of maturation ponds is governed mainly by the required bacteriological quality of the final effluent. Shallower maturations pond were more efficient in removing the faecal coliform than the deeper maturation ponds (Pearson, Mara, Arridge, 1995). The treatment is achieved through natural disinfection mechanisms (Mara et al., 1995).

Maturation ponds usually show less vertical biological and physicochemical stratification and are well oxygenated throughout the day. Algal diversity increases from pond to pond along the series (Mara, 1989). Chlorella predominance attribute to its resistance to the ammonia toxicity (Pearson et al. 1987b). The high pH which is often found in maturation ponds increase the toxicity of ammonia.

The high amount of algal biomass in the effluent represents a high-suspended matter concentration, which may exceed the final effluent quality guidelines. Typically the oxygen demand exerted by these suspended algal material is around 0.5-0.6 mg BOD5/mg algal TSS (Arceivala, 1986). BOD removal in maturation ponds is much slower than in facultative ponds, since the most easily degradable substances consumed already. In addition, experimental results showed no correlation between BOD removals in maturation ponds with temperature or retention time (Mara et al., 1992). For design purposes, it recommended to assume 25% BOD removal (based on BOD influent-total and BOD effluent-soluble) in maturation ponds (Mara and Pearson, 1992).

2.2.3. Aerated ponds

These ponds may be deeper (2-5m). In these mechanical aeration supplements the oxygen supply. The aeration also provides the physical mixing necessary to distribute dissolved oxygen, to suspend the organic material and to bring the organisms into contact with the organic material. Aerated ponds are of two types: Partially mixed and aerobic flow through completely mixed ponds.

Partially mixed ponds Here the energy input for aeration is only sufficient to transfer the amount of oxygen required, but is not sufficient to maintain the solids in suspension. Because of this reason, a portion of the incoming solids together with some of the biological solids produced will settle to the bottom. The settled solids undergo anaerobic decomposition. Effluent from these ponds contains higher insoluble BOD partially mixed ponds are shown in

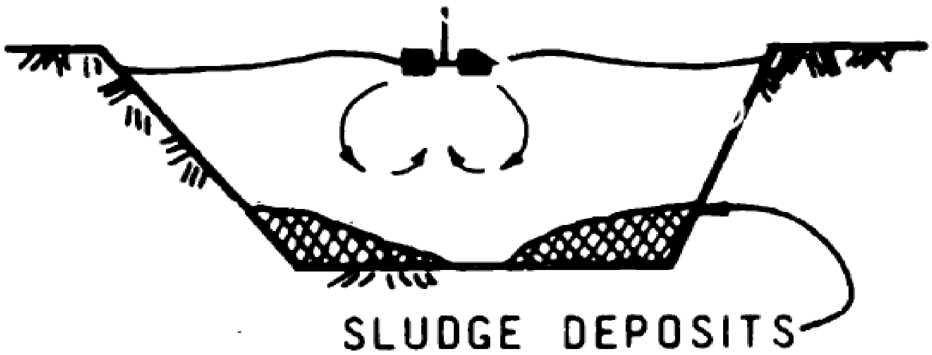


Fig :2.1. Partially mixed ponds
Source: Andersen, Lorri, biological treatment process control guide 1984.

Continuous laminar flow inversion and oxygenation

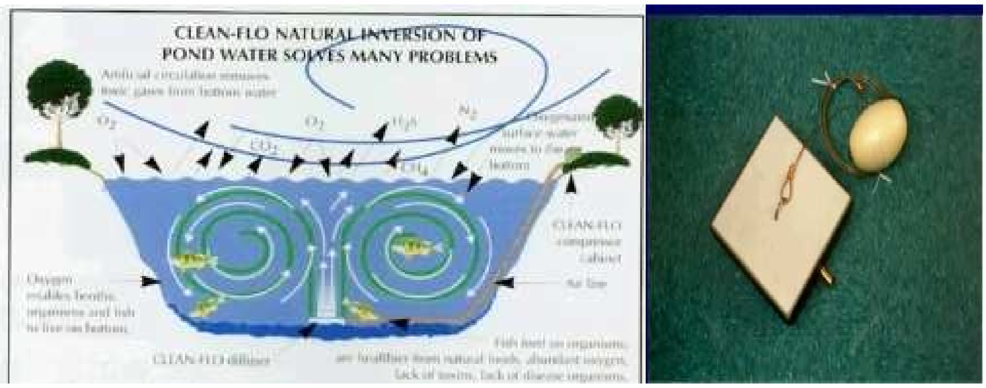


Fig 2.2(a) Clean flow inversion/oxygenation (b) microporous diffuser

Source: from the official site of clean-flo.com

CLEAN-FLO develops a water treatment process for river improvement to bring river water at the bottom of the river is brought up to the surface so that the entire river is oxygenated by the reaeration from the atmosphere surface. Bottom toxic gas neutralizes and foul odour disappears. In this technique the bottom water is circulated to the top surface with the help of micro-porous ceramic diffuser which are placed at the bottom of the pond.

CLEAN-FLO calls the most important part of CLEAN-FLO's unique process "Continuous Laminar Flow Inversion and Oxygenation." This process enables the other parts of the process to perform their functions. CLEAN-FLO International (at that time CLEAN-FLO Laboratories, Inc.) invented and perfected this energy efficient process. Unlike ordinary diffused air systems, surface aeration, paddlewheels, hypolimnetic aerators, or propeller-aspirator aerators, CLEAN-FLO oxygenates an entire body of water from top to bottom. CLEAN-FLO oxygenates and rids the entire bottom of gases including ammonia and carbon dioxide which aquatic weeds and algae need for food.

A report by Dr. Dean Martin and Ph.D. candidate Joseph Carr at the University of South Florida showed that CLEAN-FLO natural inversion transfers 4.4 lbs. of oxygen per horsepower-hour in lakes and canals. . The same report showed other diffused air systems to transfer 1.8 to 2.2 lbs. per horsepower-hour. Oxygen transfer for the CLEAN-FLO system is 7 kg O2 per diffuser per day at 3.7 CFM in waste treatment lagoons.

Ordinary diffused air systems move about 24,000 gallons of water from the bottom of lakes to the surface per horsepower-hour. Dr. Robert D. Goodwin of Lake Improvement Consultants, Walled Lake, Michigan and Dr. Ronald White of Environmental Quality Laboratory, Inc., Port Charlotte, Florida independently measured the CLEAN-FLO system to move 480,000 gallons per horsepower-hour. Of course, propeller-aspirator types, surface aerators, hypolimnetic aerators, or paddlewheels are not designed to move bottom water to the surface. If used for that purpose, they would create considerable turbulence and nutrient recycling from the bottom muck.

CLEAN-FLO Continuous Laminar Flow Inversion and Oxygenation is a process and equipment engineered and developed to duplicate natural spring and fall turnover of lakes. In tropical countries, natural inversion occurs at random with typhoons, cyclones and torrential rains. Natural inversion oxygenates the bottom water and rids the water of toxic gases that accumulate in stagnant water.

This process increases natural twice-a-year inversion of lakes to several times a day or several times a week or month. The amount of inversion depends on CLEAN-FLO's engineering design to counteract incoming pollutants and pollutants in that particular body of water. CLEAN-FLO has thirty years of experience restoring lakes, ponds, reservoirs, rivers and wastewater built into our computers to tell us how many inversions are necessary for each body of water. It takes many factors into account, such as incoming water quality and flow rate, retention time of the water body, sediment, weeds, algae, fish-kills, and much more.

The Continuous Laminar Flow Inversion and Oxygenation process carries oxygenated, toxic gas-free surface water down to the bottom of the water body. At the bottom, oxygenated water binds phosphorus and nitrogen to the sediments and kills anaerobic, often pathogenic (disease-producing), bacteria that produce acids and toxic gases. The CLEAN-FLO process usually reduces phosphorus and nitrogen in the water body three to twenty times as much as point-source watershed treatment can accomplish. This technique enables beneficial microorganisms to feed on bottom organic sediment. It enables aquatic insects to feed on the microorganisms, and fish to inhabit the bottom waters and feed on the insects.

This process inverts and oxygenates 10,900 M^3 (2,880,000 gallons) per day of water per diffuser at 6-feet depth using laminar flow principle. Laminar flow prevents mixing of anoxic, high toxic gas-content bottom water with the main water body before the water reaches the surface. At the surface, the uprising water spreads out toward the shore in a 0.04 cm (0.1-inch) deep layer. This oxygenates the water and rids the water of toxic gases. Laminar flow causes Venturi Effect to add water from the water body to the water column as it rises, causing increased water flow in deeper water.

Oxygen transfer rates under different system and ponds is variable as following

- In fish ponds, 1 meter deep: 0.72 kg/kW-hr,3 feet deep: 1.2 lbs./hp-hr
- In lakes, 2-4.5 meters deep: 2.68 kg/kW-hr ,6-15 feet deep: 4.4 lbs./hp-hr
- In waste treatment lagoons: varies according to wastewater loading rates

Aerobic flow through completely mixed ponds: these ponds are modification to the activated sludge process without sludge recycling. Turbulence level in these is sufficient to maintain the solids in suspension and provide the oxygen throughout the liquid volume. Aeration is provided by means of mechanical aerator or by submerged diffuser aerator. Detention time is usually less than 3 days and power levels are 10 times higher than that of the partially mixed ponds. Aerobic flow through completely ponds is shown in fig-2.3.

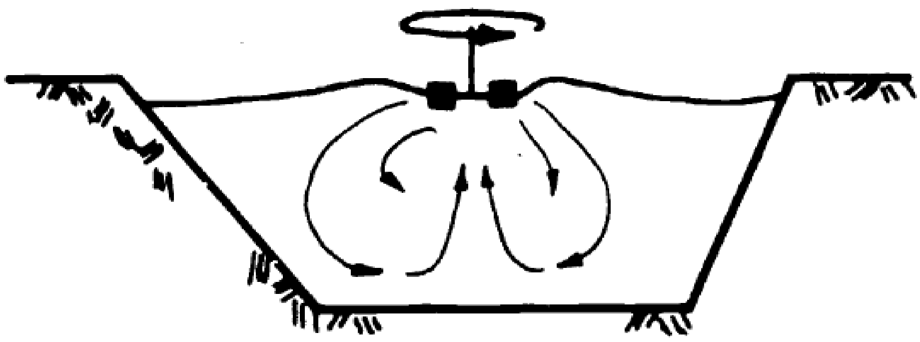


Fig--: 2.3. Aerobic flow through complete mixing
Source: Andersen, Lorri, biological treatment process control 1984.

Aerated Lagoons: Oxidation lagoons are holding basins used for secondary wastewater treatment where decomposition of organic matter is taking place naturally. An aerated lagoon is a relatively shallow body of wastewater contained in an earthen manmade basin into which wastewater flows and after certain retention time a well-treated effluent flows out. Retention time is the time taken by the effluent to flow from the inlet to the outlet of the basin. Air is supplied by mechanical aerators.

Oxidation ditches: The oxidation ditch (OD) is a sort of equipment used for a long-term aeration. It consists of a long channel of an elliptical or circular shape equipped with aeration equipment called a rotor for generating a water flow and stirring water in the channel to supply oxygen. Thought it requires a relatively large area, it has a simple structure and can be easily operated as well as being able to remove nitrogen easily. Thus, it has recently been widely used in relatively small wastewater treating plants.

Advantages of the oxidation ditch are as follows.

- It can be easily maintained
- It is hard to be effected by load fluctuations and forms only a little sludge.
- It requires relatively little energy as the rotor is operated efficiency.

2.3 How treatment occurs

Bacterial cells oxidize the organic matter of the waste water. During the process of oxidation, ammonia, carbon dioxide and other substances are liberated which are used by the algal population for their growth to produce more algal cells. The algal population in the presence of sunlight liberates free oxygen which is again used by the aerobic bacterial population to decompose the organic matter. The action taking place in these ponds is known as bacterial-algal symbiosis. The algal-bacterial interaction (symbiosis) taking place in an oxidation pond is schematically shown in the Figure-(2.4)

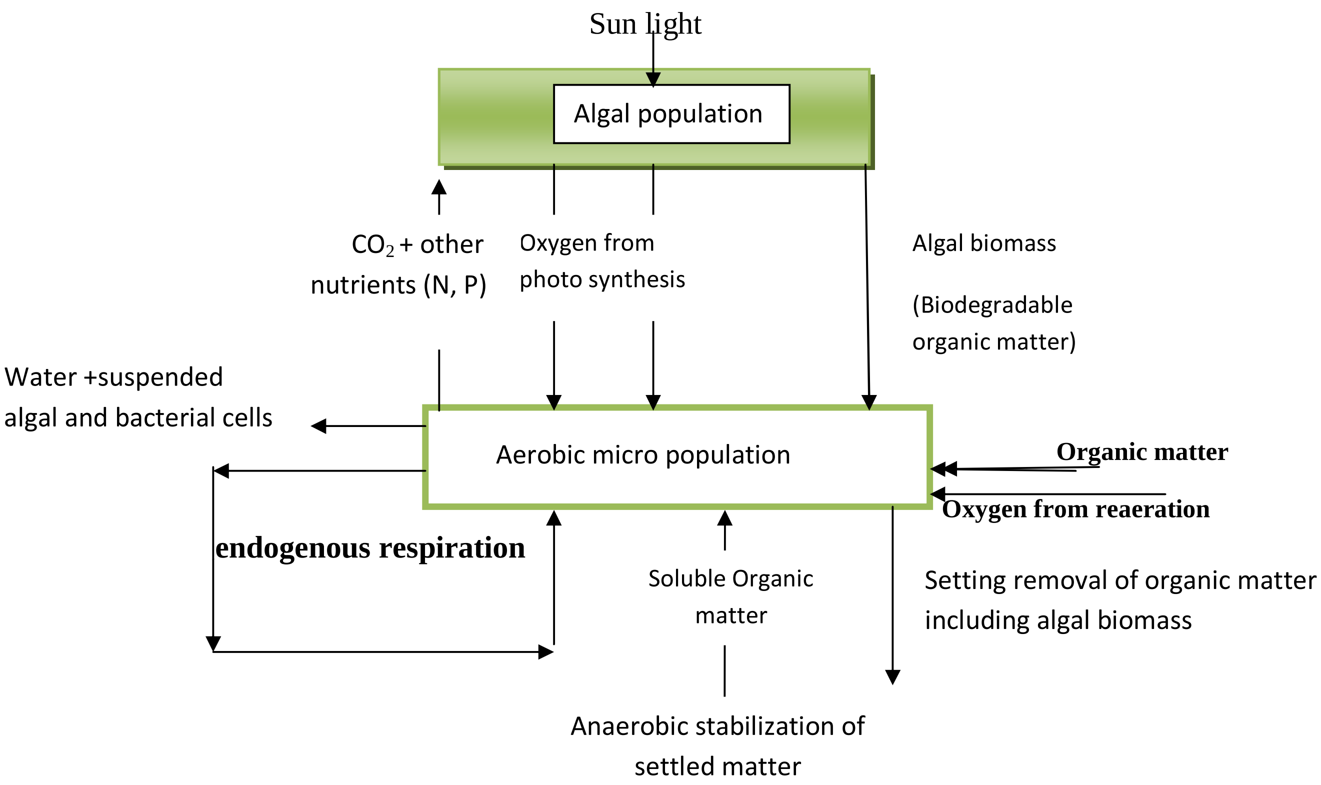


Fig --:2.4. Degradation of organic matter in oxidation ponds

Algal photosynthesis that contribute oxygen can be shown by the following equation

$$106CO_2 + 90H_2O + 16NO_3^- + PO_4^{3-} + Light \rightarrow C_{106}H_{108}O_{45}N_{16}P + \frac{309}{2}O_2$$

2.4. Designing of ponds

2.4.1. Designing of Oxidation Ponds

At a constant basin detention time, the equilibrium biological solids concentration and the overall rate of organic removal can be expected to increase as the influent organic concentration increases. For a soluble industrial waste water the equilibrium biological solid concentration (X_v)

$$X_v = \frac{aS_r}{1+bt} \qquad (2.3)$$

When non- degradable volatile suspended solids are present in the wastewater then the above equation becomes

$$X_v = \frac{aS_r}{1+bt} + X_i \qquad (2.4)$$

Where

Xi is influent volatile suspended solids concentration not degraded in the lagoon

Sr is organic removal rate (kg/d)

a is biomass yield coefficient and its value 0.5

b is endogenous rate coefficient

t is the retention time(days)

Amount of oxygen produced by photosynthesis can be estimated by

$$O_2 = CfS$$

Where

O_2 is oxygen production, Kg/(m^2.d)

C is 2.8 X 10^{-5} if O_2 is in Kg/ m^2d

S is light intensity, Cal/cm^2d

f is light conversion efficiency

Oxygen requirement for these ponds can be calculated on the basis of the biological solid concentration X_v

$$R_r = a'S_r + b' X_v \qquad (2.5)$$

Where

a' is the fraction of the organics removed that is oxidized to end products for energy and the coefficient its value is 0.52

b' the oxygen required for endogenous respiration

a and b are coefficients dependent on temperature and needs temperature correction according to following equation

The equation for winter water temperature in aerobic pond

$$\frac{t}{D} = \frac{T_i - T_w}{f(T_w - T_a)} \qquad (2.6)$$

Where

t is basin detention time (days)

D is basin depth (m)

T_i is influent wastewater temperature (C^0)

T_a is mean air temperature (C^0)

T_w is basin temperature (C^0)

f is proportionality factor

Argamon and Adams model (for the aerobic pond temperature)

In this model the overall heat balance in the basin including heat gained from solar radiation, mechanical energy input, biochemical reaction, and heat lost by long wave radiation, evaporation from the basin surface, conduction from the basin surface, evaporation and conduction from the aerator spray, and conduction through the basin walls is considered for assessing the basin temperature. The model is represented below

$$T_w = T_a + \left[\frac{q}{A}(T_i - T_a) + 10^{-6}(1 - 0.0071C_c{}^2)H_{s,o} + 6.95(\beta - 1) + 0.102(\beta - 1)T_a - e^{0.0604Ta}\left(1 - \frac{f_a}{100}\right)1.145A^{-0.05}V_w + \frac{126NFV_w}{A} + \frac{10^{-6}H_m}{A} + \frac{1.8S_r}{A}\right] \div \left[\frac{q}{A} + 0.102 + (0.068e^{0.0604Ta} + 0.118)A^{-0.05}V_w + \frac{4.32NFV_w}{A}(3 + 1.75e^{0.0604Ta}) + \frac{10^{-6}UA_w}{A}\right] \tag{2.7}$$

Where

T_w is basin water temperature (C^0)

T_a is air temperature (C^0)

T_i is influent waste temperature (C^0)

q is flow rate(m^3/d)

A is surface area(m^2)

C_c is average cloud cover(0-1)

$H_{s,o}$ is average daily absorbed solar radiation under clear sky condition

U is heat transfer coefficient

β is atmospheric radiation factor

f_a is relative humidity, percent

N is number of aerator

F is aerator spray vertical cross section area(m^2)

V_w wind speed at tree top (m/s)

H_m 15.2 X 10^6p

S_r organic removal rate (kg/d)

A_w effective wall area(m^2)

Power required for aeration and mixing

Power required for aeration is calculated on the basis of oxygen to be supplied to the pond. Oxygen to be supplied can be written as:

Oxygen required- oxygen through reaeration – oxygen through photosynthesis. Suppliers of aeration systems indicate the oxygen transfer rates of their equipment. Actual oxygen transfer rates are different from these and can be obtained by the following expression:

$$AOTR = SOTR\left(\frac{\beta . C_{\bar{s}TH} - C_L}{C_{s,20}}\right)(1.024^{T-20}).\alpha.F \qquad (2.8)$$

Where

AOTR is actual oxygen transfer rate under field conditions, Kg O_2/h

SOTR is standard oxygen transfer rate in tap water at 20°C, and zero dissolved oxygen, Kg O_2/h

α is oxygen transfer correction factor for waste

β is salinity surface tension correction factor typically 0.95 to 0.98

C_l is operating oxygen concentration

$C_{s,20}$ is dissolved oxygen saturation concentration in clean water at 20°C and 1 atm

$C_{s,t,h}$ is average dissolved oxygen saturation concentration in clean water in aeration tank at temperature T and altitude H,

$$C_{\bar{s},t,h} = \left(C_{s,t,h}\right)\frac{1}{2}\left(\frac{P_d}{P_{atm}} + \frac{O_t}{21}\right) \qquad (2.9)$$

Where

$C_{\bar{s},t,h}$ is oxygen saturation concentration in clean water temperature T and altitude H

Pd is the pressure at the depth of air releases

p_{atm} is atmospheric pressure at altitude H

Ot is percent oxygen concentration leaving tank

Supplier usually indicates the power required for supplying a unit of oxygen under standard oxygen transfer rates. Using the actual oxygen transfer rates and the power rates, for supplying the desired oxygen power required is calculated as

$$hp = \frac{R_r}{AOTR} \quad (2.10)$$

Power required for mixing

$$hp = cd^n Nt^m \quad (2.11)$$

Where

d is impeller diameter

Nt is impeller speed, r/s

n varies from 4.8 to 5.3

m varies from 2 to 2.5

2.4.2. Designing of facultative ponds

The design of facultative ponds focuses on BOD removal. Mara (1976) and Marecos do Monte and Mara (1987) described how the design of facultative ponds is currently based on rational and empirical approaches. The empirical design approach is based on correlating performance data of existing Waste Stabilization Ponds(WSP). The rational design approach models the ponds performance by using kinetic theories of biochemical reactions in association with the hydraulic flow regime.

Complete mix flow model

In a complete-mix flow, the in-pond concentration is assumed to be the same and equal to the effluent BOD concentration. Marais and Shaw (1961) proposed a model of designing facultative ponds based on first order kinetics in a complete-mix reactor.

$$\frac{S_e}{S_o} = \left[\frac{1}{1+k.t}\right]^n \qquad (2.12)$$

$$k_t = k_{20}\theta^{(T-20)} \qquad (2.13)$$

Where;

Se = effluent BOD (mg/L)

So = influent BOD (mg/L)

k = completely mixed flow 1st order rate constant for BOD removal (d^{-1})

t = mean hydraulic retention time in facultative pond (days)

n = number of ponds in series

T = mean temperature of the coldest month (°C)

Value of k_{20} is taken as 0.3/d (Mara, 1986) and value of $\theta(Arrhenius\ constant)$ is taken as 1.05 (Silva and Mara, 1979). Utilization of this equation requires the assignment of Se. Mara (1976) recommends that Se should be in the range of 50-100 mg/l (usually 70 mg/l). Hydraulic retention time can be determined by;

$$t = \left(\frac{S_o}{S_e} - 1\right)\left(\frac{1}{k}\right) \qquad (2.14)$$

The pond area is then calculated by using equation;

$$A_f = \frac{Q.t}{H} \qquad (2.15)$$

Where;

Q = design flow (m3/d)

H = pond depth in m (1.5 - 2m)

A_f= area of facultative pond (m2)

Mara (1976) and Arthur (1981) suggested that the BOD removal constant rate retards exponentially with hydraulic retention time. Wehner and Wilhelm (1956) proposed the following dispersion number model for chemical reactors based on hydraulic flow pattern, length and longitudinal dispersion:

$$d = \frac{D}{ul} \qquad (2.16)$$

Where;

D = coefficient of longitudinal dispersion (m2/d)

u = mean longitudinal flow velocity along the reactor (m/d)

l = length of fluid travel path from influent to effluent (m)

d = dispersion number in facultative pond

The dispersion coefficient D in existing reactors can be obtained experimentally by means of tests with tracers. For the designing of new ponds, literature presents some empirical relationships for preliminary estimation of d:

1. Polprasert and Batharai (1983)

$$d = \frac{0.184.t.v(B+2.H)^{0.489}.B^{1.811}}{(L.H)^{1.489}} \qquad (2.17)$$

2. Yanez (1993)

$$d = \frac{L/B}{-0.261+0.254(L/B)+1.014(L/B)^2} \qquad (2.18)$$

3. Von Sperling (1999)

$$d = \frac{1}{L/B} \qquad (2.19)$$

Where;

L = length of the pond (m)

B = breadth of the pond (m)

H = depth of the pond (m)

v = kinematic viscosity of the water (m2/d)

t = hydraulic retention time (d)

The equation of Marais and Shaw (1961) assumes that the dispersion number (d) of Wehner and Wilhelm's equation is infinity in a complete-mix flow.

Plug flow model

A plug hydraulic flow model approach is considered as the most efficient approach which ensures that the wastewater pollutants attain the theoretical hydraulic retention time. Reed et al. (1988) proposed the following plug hydraulic flow regime model for the design of primary facultative ponds:

$$\frac{S_e}{S_o} = e^{-kt} \qquad (2.20)$$

Where;

Se = effluent BOD (mg/L)

So = influent BOD (mg/L)

t = mean hydraulic retention time in facultative pond (days)

k = plug flow first order rate constant for BOD removal (d^{-1})

k is related to any temperature as follows;

$$k_T = k_{20}\theta^{(T-20)} \qquad (2.21)$$

Reed et. al. (1988) opinioned that k_{20} depends on the BOD surface loading rate and a value of 0.1 d^{-1} could be confidently adopted. Value of θ is taken as 1.06. The plug-flow model is used to calculate the retention time required for specified BOD removal requirements.

Dispersed flow regime model

In reality, the hydraulic regime in a stabilization pond does not exactly follow the ideal complete-mix or plug flow models, but an intermediate model. The complete-mix and plug flow models constitute an envelope, inside which all the reactors in reality are located. Thirumurthi (1969) recommended that ponds be designed as dispersed flow reactors since they are neither plug flow nor completely mixed. He proposed the use of pond dispersion numbers (d) and the first order equation of Wehner and Wilhelm (1956).

His equations are as follows:

$$\frac{S_e}{S_o} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}} \qquad (2.22)$$

$$a = \sqrt{1 + 4k.t.d} \qquad (2.23)$$

$$d = \frac{D}{ul} \qquad (2.24)$$

$$k_t = k_{20}\theta^{(T-20)} \qquad (2.25)$$

Where;

Se = effluent BOD (mg/l)

So = influent BOD (mg/l)

k_t = dispersed flow reaction rate for BOD removal at any temp (d^{-1})

k_{20} = dispersed flow reaction rate for BOD removal (d^{-1}) at 20°C

t = mean hydraulic retention time in facultative pond (days)

d = dispersion number

D = coefficient of longitudinal dispersion (m^2/d)

u = mean velocity of travel (m/d)

l = mean path length of a typical particle in the pond (m)

T = minimum pond temperature (°C)

The difficulty which is encountered in designing facultative ponds using this equation lies in the fact that at the design stage the value of the dispersion number (d) and the first order reaction rate for BOD removal (k) are not known. Polprasert and Bhattaria (1985) proposed the following equation for a dispersed hydraulic flow model in facultative pond design:

$$d_f = \frac{0.184[tv(W+2H)]^{0.489} W^{1.511}}{(LH)^{1.489}} \quad (2.26)$$

Where;

d_f= dispersion numbers in facultative pond

v= kinematics viscosity of the pond liquid (m^2/s)

L = pond length (m)

W= pond width (m)

H = pond depth (m)

t = mean hydraulic retention time in facultative pond (days)

Polprasert and Bhattarai's equation and Wehner and Wilhelm's equation for dispersed flow model can be used to design a facultative pond by trial and error.

Empirical model for design of facultative ponds

The surface BOD loading method is the recommended approach for designing facultative ponds. According to the US Environmental Protection Agency (1983) and Reed et al. (1988), for every climate there is an appropriate value of surface BOD loading λs (kg BOD/ha/day) which can be applied to a pond for a given removal efficiency. The facultative pond area is calculated by using following equation;

$$A_f = \frac{10 L_i Q}{\lambda_s} \quad (2.27)$$

Where

Li = influent BOD (kg BOD5/d)

Q = flow rate (m^3/d)

λ_s= surface BOD loading (kg BOD/ha/d)

Design value of λs increases with temperature. An empirical equation proposed by Mara (1997), correlate the surface loading rate λs with temperature T, this equation has global applicability and is given below:

$$\lambda s = 350 \times (1.107 - 0.002 \times T)^{(T-25)} \qquad (2.28)$$

Where:

T = mean temperature in the coldest month (°c).

Beside this equation there is a set of equation used for calculating λ_s under various conditions

$$\lambda_s = 50(1.072)^T \text{.............................1}$$

$$\lambda_s = 357 \times (1.085)^{T-20} \text{.....................2}$$

$$\lambda_s = 10T \text{...................................3}$$

$$\lambda_s = 20T - 90 \text{..............................4}$$

$$\lambda_s = 20T - 60 \text{..............................5}$$

$$\lambda_s = 107 \times S_0 \text{...............................6}$$

$$\lambda_s = 375 - 6.25L \text{...........................7}$$

L is latitude of the site

Equation 1 is used for temperature >20°C

Equation 3 is used for temperature 10 to 20

Equation 7 is used for latitudes 8 to 36°N

λr is removal rate of organic matter in kg/ha.day of BOD

Once a suitable value of λs has been selected, the pond area can be calculated and retention time is calculated from

$$t = \frac{A_f H}{Q_{avg}} \qquad (2.29)$$

Where;

H = pond depth (usually 1.5m)

Q = average flow, m3.d

The average flow is the average of the influent and the effluent flow.

Qaverage = (Qinfluent + Qeffluent)/2

$$t = \frac{A_f H}{2(Q_i + Q_e)} \quad (2.30)$$

Mass balance for flow is like;

Qeffluent = Qinfluent + Qprecipitation – Qevaporation – Qinfiltration

The effluent flow corresponds to the influent flow less net evaporation and infiltration.

$$Q_{net\ evap.} = Q_{evap.} - Q_{precipitation}$$

$$Q_e = Q_i - Q_{net\ evap.} - Q_{infilteration}$$

If infiltration is negligible, Q_e is given by:

$$Qe = Qi - 0.001A_f\ e \quad (2.31)$$

Where, e is net evaporation rate (mm/day). Thus,

$$t = \frac{2.A_f.H}{(Q_i + 0.001A_f e)} \quad (2.32)$$

To minimize hydraulic short-circuiting and to prevent algal washout a minimum detention time of 5 days should be adopted for temperature below 20°c and 4 days for temperature above 20°C. In primary facultative ponds treating domestic sewage detention times usually vary between 15 and 45 days.

2.4.3. Design of maturation ponds

Design of maturation ponds for coliform removal

The design of maturation ponds is based on bacterial decay. Faecal bacteria, protozoa and viruses die off with time because of unfavourable environment in the pond. Main factors causing removal are sedimentation, scarcity of food, predators, ultra-violet light. The main parameter to be considered in bacterial die-off in ponds is retention time. Method of Marais (1974) is generally used to design a pond series for faecal coliform removal. This assumes that faecal coliform removal can be reasonably well represented by a first-order kinetic model in a completely-mixed reactor. The resulting equation for n number of ponds is given by:

$$\frac{N_e}{N_i} = \frac{1}{\left[1+k_b\frac{\theta}{n}\right]^n} \qquad (2.34)$$

Where;

Ne = number of effluent faecal coliform per 100ml

Ni = number of influent faecal coliform per 100ml

k_b= bacteria die–off coefficient (d-1)

θ = retention time (days)

n = number of maturation ponds

The coliform die–off coefficient k_b has a great influence on the estimation of effluent coliform conc. and its value changes with type of flow. For a series of anaerobic, facultative and maturation ponds, The Equation becomes:

$$\frac{N_e}{N_i} = \frac{1}{(1+k_b t_a)(1+k_b t_f)(1+k_b t_m)^n} \qquad (2.35)$$

Where; subscripts a, f, m refers to the anaerobic, facultative and maturation pond, and n is number of maturation ponds.

Marais (1974) suggested that the most efficient pond configuration would be achieved if all the maturation ponds were of equal size, such that they had the same hydraulic retention time. However, due to topographical limitations, the size of maturation ponds cannot always be the same, in which case the Marais model is modified into equation as follows:

$$\frac{N_e}{N_i} = \frac{1}{(1+k_b t_a)(1+k_b t_f)(1+k_b t_{m1})(1+k_b t_{m2})(1+k_b t_{mn})} \quad (2.36)$$

The value of is highly temperature-dependent. Marais (1974) found that it varies with temperature as:

$$k_{bT} = k_{b20}\theta^{(T-20)} \quad (2.37)$$

Where;

T = temperature (°C)

θ= temperature coefficient

The value of θ is 1.19, as reported by Marais (1974) and k_{b20} is 2.6.Thus, k_b changes by 19% for every change in temperature of 1°c. The following three conditions are set to ensure that maturation ponds are designed satisfactorily (Marais, 1974):

1. $t_m < t_f$

2. $t_m > t_m^{min}$, Where, t_m^{min} = 3-5 days

3. $\lambda_{sm1}(BOD) \leq 0.75\lambda_{sf}$(BOD)

Where;

t_m= hydraulic retention time in each maturation ponds (days)

t_f= hydraulic retention time in secondary facultative pond (days)

t_m^{min} = minimum hydraulic retention time in maturation ponds (days)

λ_{sm1}= surface BOD loading in first maturation pond (kg/ha/day)

λ_{sf}= surface BOD loading in facultative pond (kg/ha/day)

A check must be made on the BOD loading of the first maturation pond. This must not be greater than that of the preceding facultative pond. A significantly lower value is preferable. Maximum BOD loading in the first maturation pond should be 75% of that of the preceding facultative pond.

The complete-mix hydraulic flow regime proposed by Marais (1974) can only be achieved in maturation ponds if the length-to-width ratio is close to unity (Arceivala, 1983; Polprasert and Bhattaria, 1985; Von Sperling, 1999). Maturation ponds are geometrically designed to have a high length-to-width ratio (up to 10:1) such that they can simulate a plug-flow regime to enhance their performance (Mara et al. 1992). Under these conditions, the complete-mix hydraulic flow regime suggested by Marais (1974) in modeling faecal coliform removal is not realistic. Shilton and Harrison (2003) have suggested that an ideal completely mixed hydraulic flow regime in wastewater reactors cannot be achieved due to short-circuiting and dead spaces that are produced by changes in environmental conditions such as temperature and wind-driven patterns.

Thirumurthi (1974), Arceivala (1983), Polprasert and Bhattaria (1985), Marecos do Monte and Mara (1987), and Von Sperling (1999) have suggested that a completely mixed hydraulic flow regime cannot be realized in maturation ponds. They suggest that a dispersed hydraulic flow regime is the realistic non-ideal flow that simulates the real hydraulic flow pattern in WSP. Effluent coliform conc. from pond under dispersed flow can be calculated by;

$$\frac{N_e}{N_i} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1+a)^2 e^{-a/2d}} \tag{2.38}$$

$$a = \sqrt{1 + 4k_b.t.d} \tag{2.39}$$

Ni = coliform conc. in influent (org/100ml)

Ne = coliform conc. in effluent (org/100ml)

k_b=bacteria die – off coefficient (d-1) t = detention time (d)

d = dispersion number (dimensionless)

Design of maturation pond for Nutrient Removal

Nitrogen: Pano and Middlebrooks (1982) proposed a model for the removal of ammonical nitrogen in individual facultative and maturation ponds. The model incorporates values for hydraulic loading, pH, temperature and coefficients derived from empirical data. The equations assume first order removal kinetics and complete mixing.

For temperatures below 20°C the equation is:

$$C_e = \frac{C_o}{1+[(A/Q).(0.0038+0.000134T).e^{((1.041+0.044T).(ph-6.6))}]} \qquad (2.40)$$

For temperature more than 20°C the equation is:

$$C_e = \frac{C_i}{1+[6.086\times10^{-8}.(A/Q).e^{(1.540\times(ph-6.6))}]} \qquad (2.41)$$

Where;

C_e = ammonical nitrogen concentration in pond effluent, (mg N/L)

C_o = ammonical nitrogen concentration in pond influent, (mg N/L)

A = pond surface area, (m2)

Q = wastewater flow rate, (m3/d)

T = temperature, (°C)

pH = 7.3exp (0.0005A) [where A = influent alkalinity (mg CaCO3/L)

Total nitrogen removal in the individual facultative and maturation ponds was presented by Reed (1995), as follows:

$$C_e = C_o\exp\{-[0.0064(1.039)^{t-20}][t + 60.6(ph - 6.6)]\}$$

(2.42)

Where;

C_e= total nitrogen concentration in the pond effluent, (mg N/L)

C_o= total nitrogen concentration in the pond influent, (mg N/L)

T = temperature, (°C; range: 1-28°C)

t = retention time, (days; range: 5-231days)

pH = 7.3exp (0.0005A) [where A = influent alkalinity (mg CaCO3/L)

Phosphorous: There are no design equations for phosphorus removal in WSP. Huang and Gloyna (1984) indicate that, if BOD removal in a pond system is 90 percent, the removal of total phosphorus is around 45 percent. Effluent total P is around two thirds inorganic and one-third is organic

2.5. Application of Oxidation ponds

Oxidation Ponds are the most common method of treating the wastewater of small to midsized communities. These are based on natural process and provide a cost-effective and fairly successful means of treating wastewater, in terms of removing the wastewater organic pollutants, measured as Biochemical Oxygen Demand (BOD5) and Total Suspended Solids (TSS) and pathogen (Cameron, 1983; Hickey et al., 1989; Davies-Colley et al., 1995). These ponds requires some modification for improving the efficiency of removal in term of nutrient and BOD, for the tropical and subtropical regions these ponds are the best option of treating the wastewater where climatic condition is favourable for removal of substance.

2.6. Performance evaluation of Oxidation Ponds

A full evaluation of the performance of an oxidation pond system is a time consuming and expensive process, and it requires experienced personnel to interpret the data obtained. It is in many ways close to research, but it is the only means by which pond designs can be optimized for local conditions. It is often therefore a highly cost effective exercise.

It is not intended that all pond installations be studied in this way, but only one or two representative systems in each major climatic region. This level of investigation is most likely to be beyond the capabilities of local organizations, and it would need to be carried out by a state or national body, or by a university under contract to such a body. This type of study is also necessary when it is required to know how much additional loading a particular system can receive before it is necessary to extend it.

Samples should be taken and analyzed on at least five days over a five-week period at both the hottest and coldest times of the year. Samples are required of the raw wastewater and of the effluent of each pond in the series and, so as to take into account most of the weekly variation in influent and effluent quality, samples should be collected on Monday in the first week, Tuesday in the second week and so on (local factors, such as a high influx of visitors at weekends, may influence the choice of days on which samples are collected). Generally the analytical techniques described in the current edition of Standard Methods (APHA) are recommended.

Composite samples are necessary for most parameters; grab samples are required for pH and faecal coliforms; and samples of the entire pond water column should be taken for algological analyses (chlorophyll *a* and algal genera determination). Pond column samples should be taken from a boat or from a simple sampling platform (or the outlet structure) that extends beyond the embankment base. Data on at least maximum and minimum air temperatures, rainfall and evaporation should be obtained from the nearest meteorological station. On each day that samples are taken, the mean mid-depth temperature of each pond, which closely approximates the mean daily pond temperature, should be determined by suspending a maximum-and-minimum thermometer at mid-depth of the pond at 08.00-09.00 h and reading it 24 hours later.

It is also useful to measure on at least three occasions during each sampling season the diurnal variation in the vertical distribution of pH, dissolved oxygen and temperature. Profiles should be obtained at 08.00, 12.00 and 16.00 h.

Case study-1-Evaluation of Anaerobic-Aerobic Wastewater Treatment Plant Operations

Seven small wastewater treatment plants were chosen for evaluation. These WWTPs work on the principle of anaerobic pre-treatment and aerobic post-treatment and were made with the cooperation of Slovak Technical University. Wastewater treatment plants were made for 5600 PE. When operated at suitable conditions, the results match the directive water discharge from small wastewater treatment plants in the Slovak Republic.

The experimental results indicate that plant was operated properly during monitoring. The effluent organic pollution was around 85 mg/l (26-119 mg/l)for COD and 25 mg/l (14-43 mg/l) for BOD on average. The concentration of SS was 38 mg/l (15-61 mg/l) on average, which is almost twice as high as the prescribed (20mg/l). The higher presence of SS can be explained by the lengthy polluted pipeline. The low NH4-N concentration, which was 7.5 mg/l on average (2.4-20.9mg/l), indicates that the process of nitrification was also in progress, while denitrification was not occurring. This WWTP does meet the directive, because only BOD has to be monitored for this type of WWTP and has to be less than 40 mg/l.

Case study-2-Evaluation of STPs in Delhi

On the directives of Hon'ble Supreme Court, CPCB conducted a detailed STP evaluation of Delhi during November-December 2003.There are 30 sewage treatment plants (STPs) located at 17 locations in Delhi. Out of thirty STPs, three STPs (Ghitorni, Rohini and Keshopur-I) are not found in operation. The total treatment capacity of the 30 STPs was observed as 2330 MLD. The actual treatment of sewage during November-December 2003 was found only1478 MLD (about 63% of the treatment capacity). Out of 30 STPs, 20 are running under capacity, 5 are running over capacity and 3are nonfunctional and 2 are running to their capacity. Most of the STPs (23 Nos)are based on activated sludge process except 7 STPs work on either extended aeration (2) or high rate bio-filters (3)/Trickling filters (1) and Oxidation ponds (1). Most of treatment plants working on activated sludge process do not perform satisfactorily due to operational problems.

The performance of the STPs in terms of percent reduction in pollution load in each plant was carried out. Average reduction in BOD, COD and TSS load computed as 87%, 81% and 88% respectively. The existing capacity of the treatment plants is under-utilized due to deficiency in the collection system and chocking of existing sewerage failure of pump connections and trunk sewers, internal sewers and peripheral sewers. The trunk sewers are 136 kms and heavily silted. The large network of (6000 km) peripheral sewers is very old and some of them are under sized and also in damaged condition. Part of the wastewater generated is collected through underground sewers and transported to the treatment plants and balance flows into the river Yamuna through 22 drains. There are total 28 industrial estates in all around Delhi contributes 218 MLD wastewater (either treated or untreated) in to the open drains. It is concluded that out of 3267 mld of sewage generated, treatment capacity exists for 2330 MLD(71%), and actual treatment is given to only about 1478 mld (45% of total sewage generated). It is also estimated that out of 480 tonnes/day of BOD load generated in Delhi, 264 tonnes/day (or 55%) BOD load is reduced due to treatment.

CHAPTER – 3

METHODOLOGY

3.1. Introduction

This chapter presents the methodology followed for the study. For achieving the objectives of the study, the work was planned on the following work elements.

- Characterization and quantification of the sewage received by the STPs
- Performance monitoring of the STPs

Approach followed for completing each of the work elements are briefly described below

3.2. Characterization and quantification of sewage

To carry out the performance evaluation first of all characterization and quantification of sludge reaching to the STPs is needed. For quantification we calculated the amount of sewage pumped by last pumping station to the STP. Each time one pump is switched on and all the sewage pumped by it was diverted into one lagoon. Outlet of the pond was blocked, initial water level and time was noted. After filling the pond to desired level, both water level in the pond and time was noted and outlet of the pond was opened. Top dimensions of the pond, internal slope of the side wall, and depth from the top to the initial or final water level was recorded. Pumping rate was calculated by dividing the volume of sewage accumulated in the lagoon with time taken for filling the pond.

Then we collected the data of daily pumping hour for one week and calculate the mean which was used as daily pumping hour for quantification of sewage coming to the STP. By multiplying this Pumping hour with hourly pumping rate the amount of sewage is quantified which was daily pumped to the STPs. This exercise was done at the four STPs.

Sewage characterization was done by collecting composite sampling over the time period of 24 hours. At each of the STP sites (as long as sewage is flowing in) every hour raw sewage sample was collected at the inlet. One liter of this sample was used for preparing composite sample. For each of the samples pH and temperature was recorded and also ambient air temperature and time was recorded. One sample of MPN was also taken after every 3 hour. These results characterized the sewage coming to the various STPs. Sample taken was analyzed for Temperature, BOD,COD,TSS,VSS, organic-N, ammonical-N, nitrate-N, TKN, total-P, alkalinity, Ca, Mg, Na, sulphate, sulphide, oil and grease, chloride and MPN.

3.3. Performance monitoring of STPs

The two treatment systems (one system of two ponds connected in series and having jet aerators and the second system of two ponds with laminar flow inversion/oxygenation system) of the BOH area was monitored.

Each system was monitored at the following 7 locations:

- inlet of the first pond (**P1**)
- at two locations in the first pond along the length (at the $1/3^{rd}$ of length and at $2/3^{rd}$ length **P2, P3**)
- inlet to the second pond (also known as outlet of first pond **P4**)
- at two locations in the second pond along the length (at the $1/3^{rd}$ of length and at $2/3^{rd}$ length **P5,P6**)
- at the outlet of the second pond (**P7**)

Samples collected from the inlets and outlets of the lagoons were analyzed for the BOD, COD, TKN, Nitrate, Total-P, TSS, Turbidity Temperature, pH, Alkalinity and MPN*and For MPN the sample may be collected aseptically in a separate sterilized bottle

Within the pond at each of the sampling locations samples was collected at three different depths (top 30 cm, mid depth and bottom 40cm depth).Samples collected from the ponds were analyzed for the following parameters: pH, Water temperature, Turbidity, Alkalinity, Dissolved oxygen and Chlorophyll.

On the basis of place of analysis of samples they can be tabularized as following.

Table 3.1 Parameters on the basis of place of analysis

Sr.no	Place of testing	Parameters
1.	At the STP	Temperature, pH, DO, alkalinity,Turbidity, chlorophyll (up to extraction step)
2.	At TCIRD	BOD, COD, TKN, Nitrate, Total-P, MPN

Monitoring was supposed to support performance evaluation and design analysis of STP in question. After selecting the location of sample collection and the parameters for which sample should be analyzed were decided. The monitoring involved collection of grab sample on 6 hour basis for 48 hours which on compilation gives a compressed result of 3 hour basis. Date and time of sampling and temperature of ambient air and wastewater was recorded at the time of sampling. Samples of MPN were taken at inlet-1, Inlet-2 and outlet in a sterilized bottle free from contamination. The collected samples were brought to TCIRD immediately after collection and analyzed for parameters specified in the following table and preserved until the analysis was over. Selection of parameters is done on the basis of sampling point. As in ponds some parameters (BOD, COD,TSS, TKN, MPN) analyses is not carried out because these are needed only for calculating performance of plant which is sufficient at inlet and outlet. Same in case of DO, analysis is conducted for profiling of Oxygen in the depth of pond only.

Table 3.2 Parameters to be characterized at different sampling points:

Sampling Point	Alkalinity	Temp	Turbidity	pH	BOD	COD	TSS	Total-P	TKN	Nitrate	MPN	DO	Chl.
Inlet-1	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	N	N
Pond-1													
Top	Y	Y	Y	Y	N	N	N	N	N	N	N	Y	Y
Mid	Y	Y	Y	Y	N	N	N	N	N	N	N	Y	N
Bottom	Y	Y	Y	Y	N	N	N	N	N	N	N	Y	N
Inlet-2	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	N	N
Pond-2													
Top	Y	Y	Y	Y	N	N	N	N	N	N	N	Y	Y
Mid	Y	Y	Y	Y	N	N	N	N	N	N	N	Y	N
Bottom	Y	Y	Y	Y	N	N	N	N	N	N	N	Y	N
Outlet	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	N	N

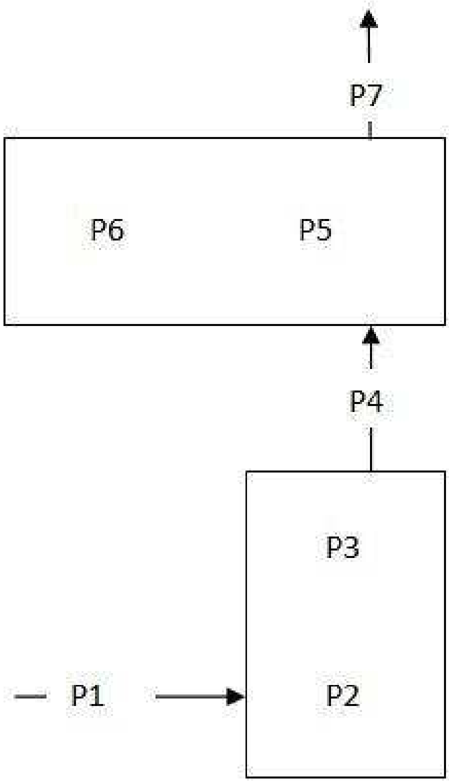


Figure 3.1. Flow diagram of Laminar flow inversion ponds at BOH STP

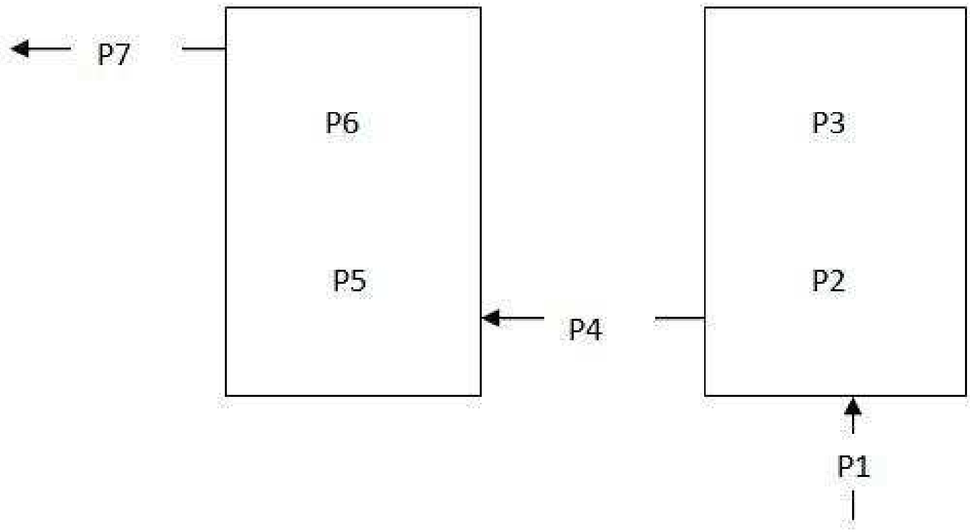


Figure 3.2. Flow diagram of Jet aerators based ponds at BOH STP

3.5. Analytical methods

Analysis is conducted onsite and offsite. In onsite analysis temperature, pH, turbidity, DO, alkalinity is done. Beside this chlorophyll is filtered and transported to lab along with other samples.

Table 3.3. Analytical techniques of Parameters characterization

Sr. No.	Parameter	Method	**References**
1	pH	Indicator method	
2	Temperature	Laboratory and Field methods	APHA (1999) "manual standard method" 20th edition (2550: B)
3	Turbidity	HACH-Spectrophotometer	HACH-Spectrophotometer Manual
4	Alkalinity	Titration Method	APHA (1999) "manual standard method" 20_{th} edition (2320: B)
5	DO	Titration Method	IS:3025 (Part 38): 1989
6	COD	Open reflux methods	APHA (1999) "manual standard method" 20_{th} edition (5220: C)
7	BOD	3 day BOD test	IS:3025 (Part 44): 1993
8	TSS	Total suspended solids dried at 103-105· C.	APHA (1999) "manual standard method" 20_{th} edition (2540: D)
9	Total-P	Vanadomolybedophosphoric Acid method	IS:3025 (Part 31): 1988
10	TKN	Macro kjehdahl method	APHA (1999) (4500Norg :B)
11	Nitrate	UV Spectrophotometry	APHA

12	Chlorophyll	Methanol Extraction Technique	Annex II in "Design Manual for WSP in India", Duncan Mara
13	MPN	Serial dilution method	APHA (1999) "manual standard method" 20th edition (9221:B,C)
14	Organic-N	Macro kjeldahl method.	APHA (1999) "manual standard method" 20th edition (4550- org: B)
15	Ammonical-N	Macro kjeldahl method	APHA (1999) (4500Norg :B)
16	Nitrate-N	Cadmium reduction Method	APHA (1999) "manual standard method" 20th edition (4500-No3: E
17	Ca	Titration Method	IS:3025 (Part 21): 1983
18	Mg	Titration Method	IS:3025 (Part 21): 1983
19	Na	Flame photometric method	APHA (1999) "manual standard method" 20th edition (3500: B)
20	Sulphate	Gravimetric method	APHA (1999) "manual standard method" 20th edition (4500: D)
21	Sulphide	Iodometric method	APHA (1998) "Standard method"(4500:F)
22	VSS	TSS dried at 550^0C	APHA (1999) "manual standard method" 20th edition (2540: G)
23	Oil and Grease	Gravimetric method	APHA (1998) "Standard method"(5520:B)

24	Chloride	Argentometric method	APHA (1999) "manual standard method" 20th edition (4500-CI^-: B)

CHAPTER 4
RESULTS AND DISCUSSION

4.1. Introduction

Results obtained from the quantification and characterization of the sewage received at the four STPs of the Ambala Military Station, and performance monitoring and evaluation of the two STPs (STP employing laminar flow inversion/oxygenation technology and STP comprising of oxidation ponds with jet aerators) at one of the four STPs sites (BOH area) are presented and discussed in this chapter.

4.2. Characterization and quantification of sewage

For the characterization a total of six composite samples of sewage were collected and analyzed in the TCIRD laboratories and the results obtained are presented in **table 4.1**. The results indicate that the sewage is of very low strength. In none of the cases BOD3 at 27C was above 100 mg/L. Raw sewage typically has 150-250 mg/L of BOD. Sewage pumped from the sewage sump 9 to the STP site at BOH was surprisingly of very low strength. High nitrogen content and sodium levels indicate that the sample is sewage and sufficiently treated.

Table 4.1: Characterization results of sewage at four STPs.

Parameter	Dairy Farm	Allen By	Panipat Line	BOH-7	BOH-8	BOH-9
BOD3 at 27°c(mg/L)	43	54	100	75	31	10
COD(mg/L)	132	202	350	155	69	60
TSS(mg/L)	85	134	200	64	21	28
VSS(mg/L)	49	30	52	18	11	12
Organic-N(mg/L)	6.72	10.64	4.31	7.84	6.72	5.6
Ammonical-N(mg/L)	45.58	8	30.18	17.94	22.4	26.32
Nitrate-N(mg/L)	1	1.7	<0.3	<0.3	<0.3	0.71
TKN(mg/L)	52.2	18.64	34.49	25.78	29.12	31.92
Total-P(mg/L)	3.4	5.63	4.66	6.11	2.19	0.3
Alkalinity(mg/L as $CaCO_3$)	479	292	473	456	429	457

Ca(mg/L)	64	46.4	56	40.8	46	41.6
Mg(mg/L)	27.33	19.18	56	21.58	12.23	9.11
Na(mg/L)	185	38.93	148	109	102	103.9
Sulphate(mg/L)	61.9	43.2	66.6	33.4	26.9	26
Sulphide(mg/L)	12.8	3.2	nil	Nil	nil	< 1
Oil and Grease(mg/L)	0.8	1	25.7	33.4	26.9	12.3
Chloride(mg/L)	118.7	34	51.4	36.7	23.7	25.3
MPN-1(CFU/100ml)	$1.4x10^7$	$1.7x10^7$	$5x10^5$	$1.7x10^6$	$5x10^6$	$8x10^5$

Sewage received at the STPs was quantified by calibrating the pumps being used for pumping the sewage to the STPs, recording the daily running hours the pumps and calculating the sewage received at the STP sites. The pumps were calibrated both by measuring the discharge of the pump with the help of an unltrasonic flow meter and by observing the volume of sewage pumped into one of the oxidation ponds per unit time of pumping. Results obtained from the sewage quantification efforts are given **table 4.2**. The results indicate, except in case of the BOH (sump-8), flow measured by the ultrasonic flow meter is higher than that measured by the volumetric method. This indicates significant leakage losses of sewage between the sewage pumps at the sumps and the corresponding STPs and/or significant seepage losses from the oxidation ponds being used for the flow measurement.

Table 4.2: Quantities of sewage received at four STPs.

Sr No.	Name of STP	Quantity of sewage(m^3/day) measured by volume per unit time method	Quantity of sewage(m^3/day) measured by ultrasonic flow meter
1	Dairy Farm	1218	1452
2	Allen By Line	508.48	728.8
3	Panipat Line	250	349.5
4.a.	BOH (Sump-7)	805	576
4.b.	BOH (Sump-8)	370	254.13
4.c.	BOH (Sump-9)	462.5	527.78

4.3. Performance monitoring of the STPs

Two STPs, both located at the BOH STP site, were monitored. Both the STPs are comprised of two ponds connected in series (**figures -4.1 and -4.2**). One of these was employing laminar flow inversion/oxygenation technology and the other one was using jet aerators in both the ponds transforming them into oxidation ponds. Performance monitoring included monitoring at 6 hour interval over two days from seven different points on each of the STPs. The sampling points were

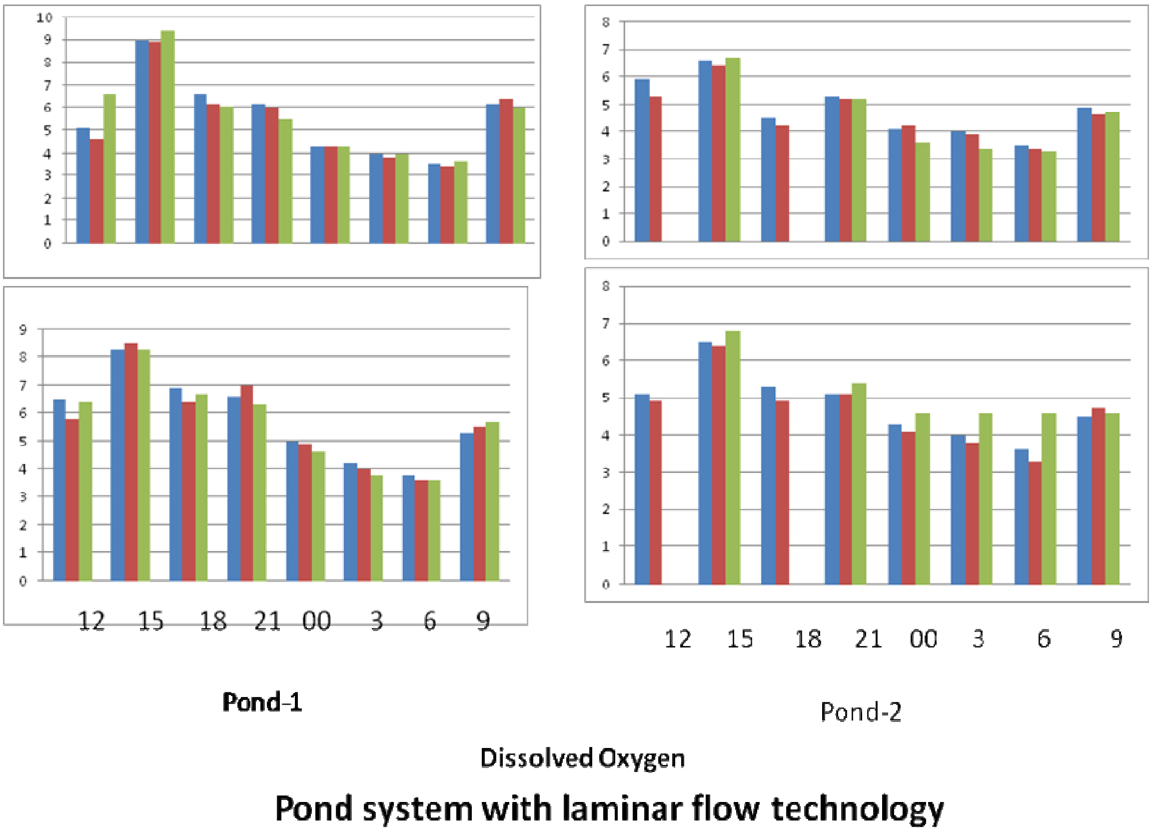
1. Inlet of the first pond - P1
2. In the pond-1 at 1/3rd distance from three different depths (top 30 cm depth, mid depth and bottom 40 cm depth) – P2.
3. In the pond-1 at 2/3rd distance from three different depths (top 30 cm depth, mid depth and bottom 40 cm depth) – P3.
4. Outlet of pond-1 / inlet of pond-2 – P4
5. In the pond-2 at 1/3rd distance from three different depths (top 30 cm depth, mid depth and bottom 40 cm depth) – P5.
6. In the pond-2 at 2/3rd distance from three different depths (top 30 cm depth, mid depth and bottom 40 cm depth) – P6.
7. Outlet of pond-2 – P7

For sample collection from different depths a hand driven peristaltic pump sampler was used. The collected samples were analyzed for some parameters on-site and for the rest of the parameters in the TCIRD laboratories. The performance monitoring results obtained are presented in the **Annexures -1 and -2**.

4.3.1 .STP incorporating laminar flow inversion/oxygenation technology

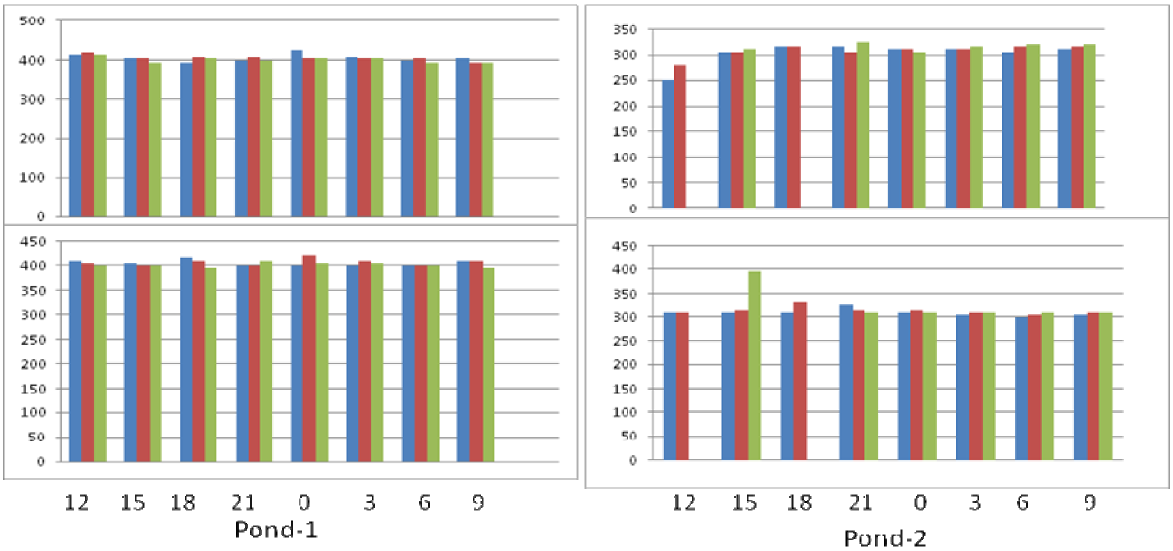
Performance monitoring of the STP was done during 19 to 23rd February 2010. Sewage loading to the STP was observed only during 900 hrs., 1500 hrs. and 1800 hrs. sampling. Dissolved oxygen levels in the ponds were found not getting affected by the sewage loading. Average DO of the pond-1 water (5.7 mg/L) was higher than that of pond-2 (4.8 mg/L). However, there were diurnal variations in the DO (a maximum of 9 mg/l at 1500 hours and a minimum of 3.5 mg/L at 600 hours at P1 location in the top 30 cm depth). Please see **table**

A.1.5 for details. All these indicate that the loaded sewage was very week. This is also evident from the BOD of the sewage being loaded (between 24-34 mg/L).



Graph; 4.1; DO in Pond system with laminar flow Technology

The pond waters were not thermally stratified. At none of the four locations difference in temperature of the water from the three different depths was >1C. Average temperature of the water in pond-1 and pond-2 were 18.3C and 18C respectively. DO levels were not showing much variations with depth (table A.1.5). pH of the water is same (6-7) throughout. Almost same is the case with turbidity (50 FTU) and alkalinity. However, alkalinity pond-1 (405 mg/L as CaCO3) was found to be greater than that in pond-2. The uniformity in the conditions of the pond water can be due to the use of laminar flow inversion/oxygenation technology. Higher alkalinity in pond-1 could be an indication of higher algal activity, and this is quite surprising.



Alkalinity

Pond system with Laminar flow inversion Technology

Graph; 4.2; Alkalinty in Pond system with laminar flow Technology

STP incorporating laminar flow inversion/oxygenation technology are designed for the removal of biodegradable organic matter. BOD and COD removals observed for the STP are given in table-4.3. When average inlet BOD is around 29 mg/L and COD is around 93 mg/L, it is very difficult to comment on the BOD and COD removal efficiencies for a pond system. The ponds must be working more like maturation ponds and hence no significant removals of BOD and COD can be expected. It is reported that the BOD and COD increase in maturation ponds because of algal growth. These things are also evident from the results shown in **table-4.3.**

Table 4.3: BOD and COD removal efficiencies for the STP incorporating laminar flow inversion/oxygenation technology

Parameter	Inlet (P1)	Outlet-1(P4)	Outlet-2 (P7)	Removal efficiency Pond-1	Removal efficiency Pond-2	Overall Efficiency
BOD	29(5.03)	32.62(3.33)	36.25(5.09)	-12.4	-11.12	-25
COD	93.33(12.86)	83.62(7.37)	87.25(8.396)	10.4	- 4.34	6.5

Note; Mean of result is taken, in ( ) Standard deviation is shown

Removal of other pollutants like suspended solids, nutrients and pathogens which occur coincidentally in the pond systems observed for the STP incorporating the laminar flow inversion technology are shown in table 4.4. Very low levels of TSS in the influent makes TSS removal efficiencies irrelevant for the STP in question. In case of pond systems the efficiencies are bound to depend on the algal cells washing out from the system. This is evident from the results presented in table 4.4. Since both the ponds were aerobic and DO levels in them were quite high, nitrification conversion of ammonical nitrogen to nitrate nitrogen is very much anticipated and thus, as expected, the nitrate nitrogen concentration was found increasing (table 4.4). The total nitrogen and the total Khjeldhal nitrogen (TKN) were showing reduction by 21.5 and 51.3% respectively. This removal is slightly lower than the expected removals of 79% (calculated from the empirical equations available from the literature). Coliform count was getting reduced the way it is anticipated. Overall reduction of the MPN was 99.9% (table 4.4) against the calculated removal of 99.5%.Removal efficiency of nitrate is too much negative (-502 and -473) due to nitrification conversion of ammonical nitrogen to nitrate beside that presence of algae also contribute to this pattern.

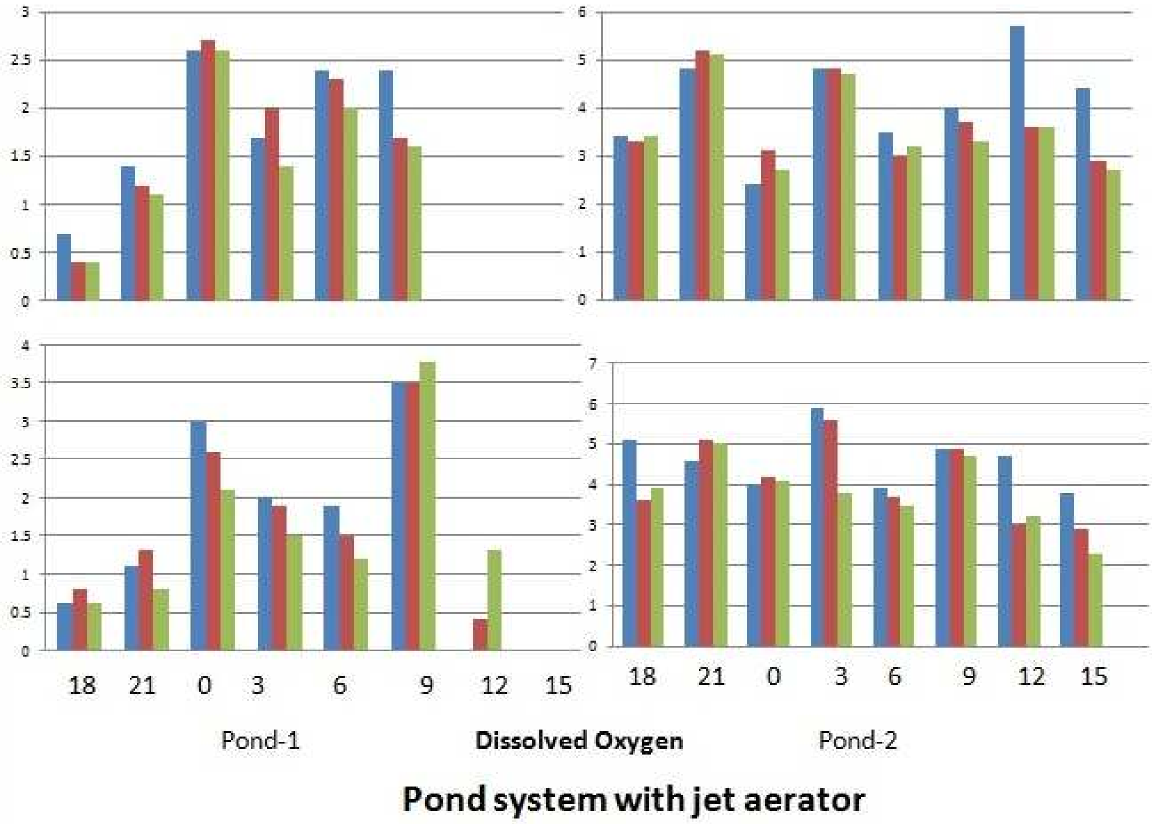
Table 4.4. TSS, nutrients and pathogens removal efficiencies for the STP incorporating laminar flow inversion/oxygenation technology

Parameters	Inlet-1(P1)	Oulet-1 (P4)	Outlet-2 (P7)	Removal efficiency (%) Pond-1	Removal efficiency (%) Pond-2	Overall Efficiency (%)
TKN	25.75(12.3)	27.75(12.13)	12.54(17.66)	-7.76	54.81	51.30
Nitrate	1.55(0.04)	1.47(0.1024)	8.89(0.2847)	4.8	-502	-473
Total nitrogen	27.3	29.22	21.43	-7.02	26	21.50
Total-P	1.58(0.25)	2.54(0.200)	2.05(0.253)	-60	19.13	-29.7
MPN	$6.9X10^{6}$	$1.6X10^{6}$	$6.4X10^{4}$	76	96	99.90
TSS	34.66(6.11)	36.5(2.56)	41.5(3.16)	-5.30	-13.6	-19.73

Note; Mean of result is taken, in ( ) Standard deviation is shown

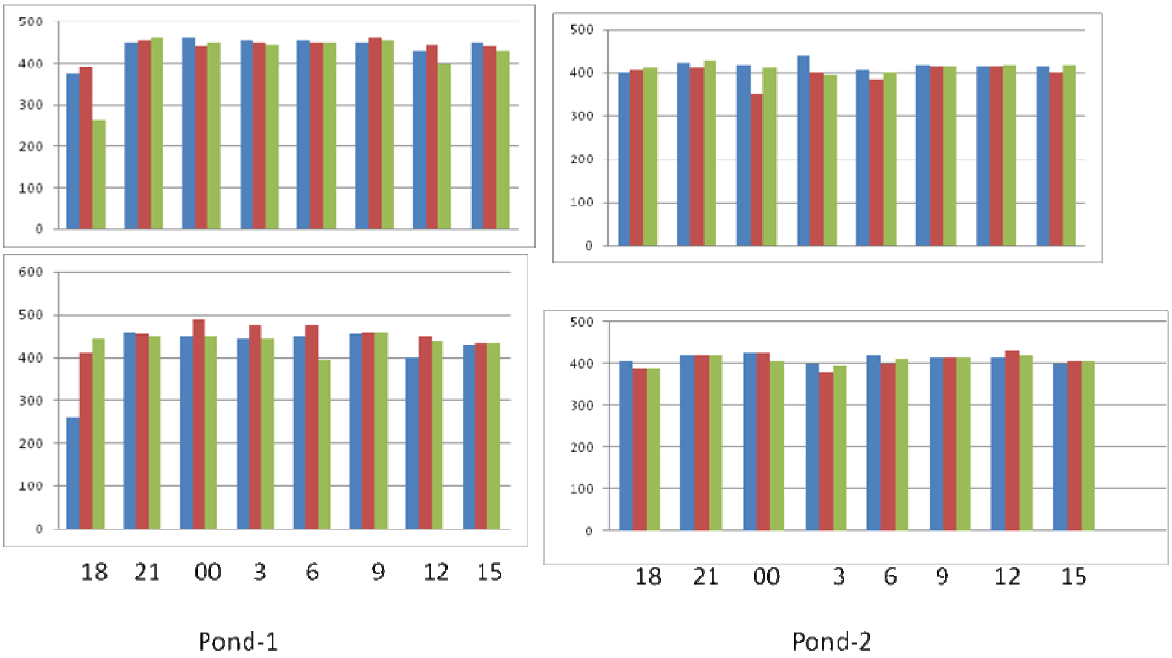
4.3.2 STP incorporating oxidation ponds with jet aerators

Performance monitoring of the STP was done during 4 to 6th March 2010. Sewage loading to the STP was observed only during 900 hrs. and 1200 hrs. sampling. Dissolved oxygen levels in the ponds were thoroughly disturbed during this period and DO reached zero mg/L throughout the depth because of the sewage loading. Average DO of the pond-1 water (1.4 mg/L) as expected was lower than that of pond-2 (4.0 mg/L). Diurnal variations in the DO were not significant. However, higher DO levels were observed during night hours. this could be because of the running of the jet aerators and because of the absence of loading of any sewage. Please see **table A.2.5** for details. All these indicate that the STP was experiencing shock loadings.



Graph; 4.3; DO in Pond system Jet Aerators

The pond waters were not thermally stratified. At none of the four locations difference in temperature of the water from the three different depths was >1C. Average temperature of the water in pond-1 and pond-2 were 24C and 24.1C respectively. DO levels however was showing slight variations with depth (table A.2.5) at the four points of monitoring. pH of the water is same (6-7) throughout. Almost same is the case with turbidity, but its value was higher pond-2 (66 FTU for pond-2 while for pond-1 it was 47) and alkalinity. Alkalinity was also not varying with depth and its average value was 441 mg/L as $CaCO_3$ for the pond-1 and 409 mg/L as $CaCO_3$ for the pond-2.



Alkalinity

Pond system with jet aerator

Graph; 4.4; Alkalinity in Pond system with Jet Aerators

The uniformity in the conditions of the pond water can be due to the running of jet aerators in the ponds. Higher alkalinity in pond-1 could be an indication of higher algal activity, and this is quite surprising. Hydraulic short circuiting almost bypassing half of the pond-1 could be responsible for this. This is evident from the average alkalinity experienced at the P2 sampling location. Please see table A.2.1 for details.

STP incorporating two oxidation ponds with jet aerators was designed for the removal of biodegradable organic matter. BOD and COD removals observed for the STP are given in table-4.5. As expected the BOD and COD removals were higher 74.4% and 69% respectively. Slight negative removal efficiencies observed in pond-2 for both BOD and COD could be because of the functioning of the pond-2 as a maturation pond.

Table 4.5. Jet aerators

Parameter	Inlet (P1)	Outlet-1(P4)	Outlet-2 (P7)	Removal efficiency Pond-1	Removal efficiency Pond-2	Overall Efficiency
BOD	121.5(37.48)	27.5(14.7)	30.87(18.5)	77	-12.25	74.4
COD	368(121.6)	111.25(53.9)	114.10 (77)	69.76	-2.56	68.99

Removals of other pollutants, like, suspended solids, nutrients and pathogens, which occur coincidentally in the pond systems, observed for the STP incorporating oxidation ponds with jet aerators are shown in table 4.6. TSS removal efficiencies were as expected quite high (83%). TSS levels in the treated effluent were in the expected range of 30-50 mg/L. Since both the ponds were mostly aerobic and DO levels especially in the pond-2 was quite high (4 mg/L), nitrification conversion of ammonical nitrogen to nitrate nitrogen is very much anticipated and thus, as expected, the nitrate nitrogen concentration was found increasing (table 4.6). The total nitrogen and the total Khjeldhal nitrogen (TKN) were showing reduction by 53% and 31% respectively. Expected removals of total nitrogen calculated from the empirical equations available from the literature were 77% (slightly on the higher side). Coliform count was getting reduced but unexpectedly less efficiently. While anticipated removals were 98.6% actual removals were just 79%. Hydraulic short circuiting could be responsible for this and this is evident from the results shown in table 4.6 (46% for pond-1 where the hydraulic short circuiting was there and 62% for pond-2).

Table 4.6. Jet aerators

Parameters	Inlet-1(P1)	Oulet-1 (P4)	Outlet-2 (P7)	Removal efficiency Pond-1	Removal efficiency Pond-2	Overall Efficiency
Nitrate	2.35(0.35)	1.38(0.2)	2.45(0.74)	41.2	-77.5	4.26
TKN	39.72(4.79)	33.93(3.90)	27.37(10.7)	14.57	19.33	31.09
Total Nitrogen	63.22	35.31	29.82	44.14	15.54	52.83
Total-P	4.66(0.99)	3.622(0.95)	3.30(1.04)	22.27	8.89	29.18
MPN	$2.6X10^7$	$1.4X10^7$	$5.38X10^6$	46.15	61.57	79.30
TSS	187(41.01)	21.12(8.15)	31.5(19.2)	88.70	-49.14	83.155

Note; Mean of result is taken, in ( ) Standard deviation is shown

Removal of BOD and COD from both the aeration system was taking place properly. The incoming sewage in laminar flow aeration pond is 29 mg/l which is less than the prescribed limit (30 mg/l). The high BOD in outlet shows that algal cells were present in the treated water which contributes to the high BOD. In both type of system second pond in series contribute to increase in BOD. In jet aerator system the removal efficiency is around 75 % and 30 mg/l BOD in outlet show that system is complying with the effluent limits. COD concentration in outlet of both systems was also complying with effluent limits.

CHAPTER 5
CONCLUSION

The STP was complying with the prescribed standards for effluent discharge effluent. Removal of BOD and COD from both the aeration system was taking place properly. The incoming sewage in laminar flow aeration pond is 29 mg/l which is less than the prescribed limit (30 mg/l). The high BOD in outlet shows that algal cells were present in the treated water which contributes to the high BOD. In both type of system second pond in series contribute to increase in BOD. In jet aerator system the removal efficiency is around 75 % and 30 mg/l BOD in outlet show that system is complying with the effluent limits. COD concentration in outlet of both systems was also complying with effluent limits. For other nutrients TSS removal efficiencies were as expected quite high (83%). TSS levels in the treated effluent were in the expected range of 30-50 mg/L. Since both the ponds were mostly aerobic and DO levels specially in the pond-2 was quite high (4 mg/L), nitrification conversion of ammonical nitrogen to nitrate nitrogen is very much anticipated and thus, as expected, the nitrate nitrogen concentration was found increasing (table 4.6). The total nitrogen and the total Khjeldhal nitrogen (TKN) were showing reduction by 53% and 31% respectively. Thus overall plant was complying with the prescribed effluent limits.

ANNEXURE-1

Results of performance monitoring of the STP employing laminar flow inversion/oxygenation technology

A.1.1: Alkalinity

	P1	P2			P3			P4	P5			P6			P7
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot	
1200 hrs	NA	415	420	415	410	405	400	410	250	280	NA	310	310	NA	305
1500 hrs	365	405	405	395	405	400	400	390	305	305	310	310	315	395	310
1800 hrs	375	395	410	405	415	410	395	405	315	315	NA	310	330	NA	305
2100 hrs	NA	400	410	400	400	400	410	410	315	305	325	325	315	310	315
0000 hrs	NA	425	405	405	400	420	405	405	310	310	305	310	315	310	305
0300 hrs	NA	410	405	405	400	410	405	400	310	310	315	305	310	310	305
0600 hrs	NA	400	405	395	400	400	400	405	305	315	320	300	305	310	310
0900 hrs	NA	405	395	395	410	410	395	400	310	315	320	305	310	310	310

A.1.2: Turbidity

	P1	P2			P3			P4	P5			P6			P7
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot	
1200 hrs	NA	54	60	58	52	50	50	54	52	52	NA	58	54	NA	54
1500 hrs	72	52	53	51	49	52	45	47	46	45	48	48	52	53	46
1800 hrs	98	56	52	52	52	52	52	56	56	64	NA	66	54	NA	56
2100 hrs	NA	52	67	58	48	50	51	39	46	45	45	54	49	49	47
0000 hrs	NA	49	51	49	46	46	46	46	46	48	50	48	49	45	52
0300 hrs	NA	48	50	47	45	45	45	46	46	48	50	48	49	45	46
0600 hrs	NA	47	48	46	47	44	48	48	48	49	51	52	51	45	49
0900 hrs	82	54	50	56	46	49	51	52	49	50	50	45	54	45	47

A.1.3: Temperature

	P1	P2			P3			P4	P5			P6			p7	Ambient
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot		
1200 hrs	21	20	20	20	20	20	20	19	19	19	NA	20	20	NA	18	23
1500 hrs	22	21	20	20	20	20	20	20	20	20	20	20	20	20	20	23
1800 hrs	19	18	19	19	18	19	19	17	18	18	NA	18	18	NA	18	19
2100 hrs	NA	18	19	19	18	19	19	18	18	19	19	18	19	19	18	15
0000 hrs	NA	17	17	18	17	18	18	18	17	17	17	16	16	17	17	13
0300 hrs	NA	16	17	17	16	17	17	16	16	17	17	16	17	17	17	10
0600 hrs	NA	16	17	17	16	16	16	16	15	16	16	14	14	14	14	9
0900 hrs	22	18	19	19	18	19	19	19	18	19	19	19	19	19	18	18

A.1.4: pH

	P1	P2			P3			P4	P5			P6		
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot
1200 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	NA	6-7	6-7	NA
1500 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
1800 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	NA	6-7	6-7	NA
2100 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0000 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0300 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0600 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0900 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7

Note; Range is used because pH was determined on site by pH strip method.

A.1.5: DO

	P1	P2			P3			P4	P5			P6		
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot
1200 hrs	*	5.1	4.6	6.6	6.5	5.8	6.4	*	5.9	5.3	NA	5.1	4.9	NA
1500 hrs	*	9	8.9	9.4	8.3	8.5	8.3	*	6.6	6.4	6.7	6.5	6.4	6.8
1800 hrs	*	6.6	6.2	6.1	6.9	6.4	6.7	*	4.5	4.2	NA	5.3	4.9	NA
2100 hrs	*	6.2	6	5.5	6.6	7	6.3	*	5.3	5.2	5.2	5.1	5.1	5.4
0000 hrs	*	4.3	4.3	4.3	5	4.9	4.6	*	4.1	4.2	3.6	4.3	4.1	4.6
0300 hrs	*	4	3.8	4	4.2	4	3.8	*	4	3.9	3.4	4	3.8	4.6
0600 hrs	*	3.5	3.4	3.6	3.8	3.6	3.6	*	3.5	3.4	3.3	3.6	3.3	4.6
0900 hrs	*	6.2	6.4	6	5.3	5.5	5.7	*	4.9	4.6	4.7	4.5	4.7	4.6

NA; Sample not available

(*); Sample not taken

A.1.6: BOD

	P1	P4	P7
1200 hrs	NA	31	41
1500 hrs	28	33	35
1800 hrs	24	28	41
2100 hrs	NA	31	35
0000 hrs	NA	37	37
0300 hrs	NA	31	38
0600 hrs	NA	38	25
0900 hrs	34	32	38

A.1.7: COD

	P1	P4	P7
1200 hrs	NA	96	100
1500 hrs	88	76	84
1800 hrs	108	92	96
2100 hrs	NA	76	80
0000 hrs	NA	79	83
0300 hrs	NA	84	92
0600 hrs	A	86	75
0900 hrs	84	80	88

A.1.8: TKN

	P1	P4	P7
1200 hrs	NA	22.5	2.7
1500 hrs	39.9	44.59	52.85
1800 hrs	19.9	24.4	3.2
2100 hrs	NA	49.28	23.38
0000 hrs	NA	24	3.5
0300 hrs	NA	18.55	4.9
0600 hrs	NA	20.86	4.9
0900 hrs	17.5	17.85	4.9

A.1.9: Total-P

	P1	P4	P7
1200 hrs	NA	2.7	2.1
1500 hrs	1.6	2.28	1.82
1800 hrs	2	2.3	1.9
2100 hrs	NA	2.33	1.96
0000 hrs	NA	2.7	2.6
0300 hrs	NA	2.75	1.91
0600 hrs	NA	2.7	2.2
0900 hrs	1.54	2.52	1.91

A.1.10: Nitrate

	P1	P4	P7
1200 hrs	NA	1.4	9.1
1500 hrs	1.54	1.4	9.31
1800 hrs	1.59	1.5	8.88
2100 hrs	NA	1.4	8.92
0000 hrs	NA	1.52	8.9
0300 hrs	NA	1.46	8.36
0600 hrs	NA	1.7	9
0900 hrs	1.52	1.42	8.66

A.1.11: Characterization results of TSS for Laminar flow inversion Technique

	P1	P4	P7
1200 hrs	NA	38	45
1500 hrs	28	40	42
1800 hrs	36	37	46
2100 hrs	NA	37	43
0000 hrs	NA	34	38
0300 hrs	NA	34	40
0600 hrs	NA	39	41
0900 hrs	40	33	37

A.1.12: Characterization results of MPN for Laminar flow inversion Technique

	P1	P4	P7
1200 hrs	NA	$5X10^6$	$2.5x10^4$
1500 hrs	$3x10^6$	$1.3X10^5$	$5x10^4$
1800 hrs	$1.6x10^7$	$1.4X10^6$	$5x10^4$
2100 hrs	NA	$2.2X10^5$	$3x10^4$
0000 hrs	NA	$8X10^5$	$1.4x10^5$
0300 hrs	NA	$2.2X10^5$	$5x10^4$
0600 hrs	NA	$5X10^6$	$8x10^4$
0900 hrs	1.7x106	$2.2X10^5$	$9x10^4$

A.1.13: Characterization results of Chlorophyll for Laminar flow inversion Technique

	P2	P3	P5	P6
1200 hrs	85.7	25.9	257	106.4
0900 hrs	109	184	85.7	41.5

ANNEXURE-2

Results of performance monitoring of the STP employing jet aerators

A.2.1: Alkalinity

	P1	P2			P3			P4	P5			P6			P7
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot	
1800 hrs	NA	375	390	260	410	445	435	450	400	405	410	405	390	390	395
2100 hrs	NA	450	455	460	455	450	450	450	425	410	430	420	420	420	415
0000 hrs	NA	460	440	450	490	450	455	445	420	350	410	425	425	405	415
0300 hrs	NA	455	450	445	475	445	450	425	440	400	395	400	380	395	400
0600 hrs	NA	455	450	450	475	395	455	440	405	385	400	420	400	410	410
0900 hrs	505	450	460	455	460	460	450	495	420	415	415	415	415	415	420
1200 hrs	635	430	445	400	450	440	445	460	415	415	420	415	430	420	410
1500 hrs	NA	450	440	430	435	435	460	425	415	400	420	400	405	405	420

A.2.2:Turbidity

	P1	P2			P3			P4	P5			P6			P7
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot	
1800 hrs	NA	46	43	46	44	43	43	45	64	66	66	62	63	64	67
2100 hrs	NA	36	40	38	36	38	39	58	58	62	61	60	62	59	71
0000 hrs	NA	50	40	40	44	44	46	45	68	69	69	72	68	73	73
0300 hrs	NA	43	43	46	46	43	44	44	68	70	71	70	69	70	74
0600 hrs	NA	50	53	55	46	48	49	48	68	70	69	70	72	68	78
0900 hrs	51	51	52	55	48	47	43	51	66	68	70	70	70	68	80
1200 hrs	172	59	63	56	55	54	46	61	58	61	64	58	63	65	85
1500 hrs	NA	52	53	50	52	52	53	54	69	63	65	68	65	64	80

A.2.3: Temperature

	P1	P2			P3			P4	P5			P6			P7	Ambient
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot		
1800 hrs	NA	25	25	25	24	24	24	25	24	24	24	24	24	24	24	26
2100 hrs	NA	24	25	25	24	25	25	24	24	24	24	24	24	24	24	20
0000 hrs	NA	23	23	23	23	23	23	23	22	23	23	22	23	23	22	20
0300 hrs	NA	23	23	23	23	23	23	23	23	23	23	23	23	23	23	20
0600 hrs	NA	21	22	23	22	23	23	23	22	23	23	23	23	23	23	16
0900 hrs	25	24	24	23	24	24	24	24	25	24	24	24	24	24	24	20
1200 hrs	25	27	26	25	25	26	26	26	28	27	26	27	26	26	27	26
1500 hrs	NA	26	25	25	26	25	25	26	26	25	25	26	26	26	26	25

A.2.4. pH

	P1	P2			P3			P4	P5			P6			P7
		Top	Mid	Bot	Top	Mid	Bot		Top	Mid	Bot	Top	Mid	Bot	
1800 hrs	NA	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
2100 hrs	NA	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0000 hrs	NA	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0300 hrs	NA	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0600 hrs	NA	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
0900 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
1200 hrs	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7
1500 hrs	NA	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7	6-7

Note; Range is used because pH was determined on site by pH strip method.

A.2.5: DO

	P2			P3		P	P5			P6		
	Top	Mid	Bot	Top	Mid	Bot	Top	Mid	Bot	Top	Mid	Bot
1800 hrs	0.7	0.4	0.4	0.6	0.8	0.6	3.4	3.3	3.4	5.1	3..6	3.9
2100 hrs	1.4	1.2	1.1	1.1	1.3	0.8	4.8	5.2	5.1	4.6	5.1	5
0000 hrs	2.6	2.7	2.6	3	2.6	2.1	2.4	3.1	2.7	4	4.2	4.1
0300 hrs	1.7	2	1.4	2	1.9	1.5	4.8	4.8	4.7	5.9	5.6	3.8
0600 hrs	2.4	2.3	2	1.9	1.5	1.2	3.5	3	3.2	3.9	3.7	3.5
0900 hrs	2.4	1.7	1.6	3.5	3.5	3.8	4	3.7	3.3	4.9	4.9	4.7
1200 hrs	0	0	0	0	0.4	1.3	5.7	3.6	3.6	4.7	3	3.2
1500 hrs	0	0	0	0	0	0	4.4	2.9	2.7	3.8	2.9	2.3

A.2.6: BOD

1800 hrs	NA	24	16
2100 hrs	NA	36	24
0000 hrs	NA	25	48
0300 hrs	NA	21	28
0600 hrs	NA	19	23
0900 hrs	148	12	69
1200 hrs	95	23	15
1500 hrs	NA	60	24

A.2.7: COD

	P1	P4	P7
1800 hrs	NA	142	113
2100 hrs	NA	165	130
0000 hrs	NA	90	152
0300 hrs	NA	66	128
0600 hrs	NA	61	136
0900 hrs	454	58	326
1200 hrs	282	104	87
1500 hrs	NA	204	83

A.2.8: TKN

	P1	P4	P7
1800 hrs	NA	38.18	24.02
2100 hrs	NA	26.99	48.66
0000 hrs	NA	33.25	20.94
0300 hrs	NA	33.87	26.48
0600 hrs	NA	33.26	25.25
0900 hrs	43.11	32.64	37.56
1200 hrs	36.34	33.25	19.71
1500 hrs	NA	40.03	16.35

A.2.9: Total-P

	P1	P4	P7
1800 hrs	NA	3.08	2.56
2100 hrs	NA	4.84	3.68
0000 hrs	NA	2.89	1.72
0300 hrs	NA	3.17	2.94
0600 hrs	NA	3.03	2.75
0900 hrs	3.96	2.75	5.13
1200 hrs	5.36	4	3.72
1500 hrs	NA	5.22	3.96

A.2.10: Nitrate

	P1	P4	P7
1800 hrs	NA	1.6	2.7
2100 hrs	NA	1.6	4.2
0000 hrs	NA	1.4	2
0300 hrs	NA	1.4	2.1
0600 hrs	NA	1.2	2.2
0900 hrs	2.6	1.2	2.2
1200 hrs	2.1	1.1	2.2
1500 hrs	NA	1.6	2

A.2.11: TSS

	P1	P4	P7
1800 hrs	NA	18	16
2100 hrs	NA	28	28
0000 hrs	NA	13	24
0300 hrs	NA	16	27
0600 hrs	NA	9	25
0900 hrs	216	26	77
1200 hrs	158	31	35
1500 hr	NA	28	20

A.2.12: MPN

	P1	P4	P7
1800 hrs	NA	$1.4X10^6$	$1.7X10^5$
2100 hrs	NA	$3.0X10^6$	$3.0X10^6$
0000 hrs	NA	$3.5X10^5$	$3.0X10^6$
0300 hrs	NA	$1.1X10^5$	$3.5X10^6$
0600 hrs	NA	$5.0X10^5$	$1.7X10^6$
0900 hrs	$2.8X10^7$	$2.2X10^6$	$2.2X10^7$
1200 hrs	$2.4X10^7$	$1.7X10^6$	$1.7X10^6$
1500 hrs	NA	$1.3X10^7$	$8.0X10^6$

A.2.13: Chlorophyll

	P2	P3	P5	P6
1800 hrs	83	132.5	85.7	51.9
0900 hrs	49	205	57	19
1200 hrs	228.5	145.4	15.5	77.9
1500 hrs	101	132	132	137.6

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