# Performance Evaluation of Waste Stabilization Pond Based Sewage Treatment Plant

A Thesis Submitted to

**THAPAR UNIVERSITY**

in the partial fulfilment for the award of the

Degree of

**MASTER OF TECHNOLOGY**
**in**
**ENVIRONMENT SCIENCE &TECHNOLOGY**

by

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# CANDIDATE'S DECLARATION

I hereby declare that the work presented in the dissertation entitled **"Performance evaluation of Stabilization Pond Based Sewage treatment plant"** in partial fulfillment of the requirement for the award of degree of **Master of Technology** in **Environment Science & Technology**, Thapar University, Patiala, is an authentic record of my own work during the period of twelve months from August 2008 to July 2009, under the supervision of Dr. A.S. Reddy, Department of Biotechnology and Environmental Sciences, Thapar University. This work has not been submitted to this or any other university till now for the award of any other degree, diploma or equivalent course.

POONAM KHATRI

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This is to certify that the above statement made by the candidate is correct and true to the best of our knowledge.

| | |
|---|---|
| **Dr. A. S. Reddy** | **Dr. N. Das** |
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## CERTIFICATE

This is to certify that the thesis report entitled **"Performance evaluation of Waste Stabilization Pond Sewage treatment plant"** submitted by Poonam Khatri in the partial fulfillment of the requirement for the award of degree of the **Master of Technology in Environmental Science & Technology** to the Thapar University, Patiala, is a record of student's own work carried out by her under my supervision and guidance. The report has not been submitted for the award of any other degree or certificate in this or any other university or Institution.

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# ABSTRACT

Waste stabilization 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 labour. 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 waste stabilization ponds has been found to be very good. WSPs 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. Thus knowing actual performance and capacity of the STP becomes very important. The current study is an attempt in this direction. Samples were collected over four months (Feb to May 09) from four sampling points; raw sewage [P-1], anaerobic pond outlet [P-2], facultative pond outlet [P-3], maturation pond outlet [P-4] to evaluate the performance of STP in question. Treated effluent of the STP was found not complying with the prescribed standards. Despite having maturation ponds, treated water was not fit even for restricted irrigation or for aqua-culturing. Average BOD (68 mg/l) and MPN ($2x10^6$ /100ml) of the treated effluent over the four monitoring months were much higher than the standard limits.

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# CHAPTER 1
# INTRODUCTION

## 1.1 Background information

The Satluj river is heavily polluted by the wastewater discharged specially by Phillaur, Phagwara, Ludhiana, Jalandhar, Kapurthala and Sulthanpur Lodi cities of the Punjab State. In an effort to improve the Satluj river water quality, National River Conservation Directorate (NRCD) of Ministry of Environment and Forests (MOEF), Government of India (GOI) has installed and commissioned till date as many as eight Sewage Treatment Plants (STPs) under the Satluj River Action Plan (SRAP) for the treatment of the municipal sewage generated by these cities prior to discharge into the Satluj river. Punjab water supply and sewerage board (PWSSB) is responsible for running these STPs and treating the municipal sewage and then discharging into the river Satluj. NRCD has entrusted Thapar Univeristy with the responsibility of monthly performance monitoring of these STPs and reporting to the NRCD.

Sewage Treatment Plants (STPs) are supposed to make the municipal sewage compatible for disposal into the environment (surface and underground water bodies or land), to minimize the environmental and health impacts of the sewage, and to make the sewage fit for recycling and reuse (agricultural and aqua-cultural uses and municipal and industrial uses). The STPs, installed and commissioned under the Satluj River Action Plan, are based on the UASB and the stabilization pond technologies. In both the cases the treatment systems are designed on the basis of empirical equations obtained from the past experience elsewhere. This necessitates evaluation of the design. Further, evaluation of the STPs for assessing whether they are performing as per the design and for knowing whether the STPs were designed for complying with the purposes to be served is also important.

This M. Tech. dissertation work is concerned with the performance evaluation and design analysis of the sewage treatment plant (based on stabilization pond technology), installed at Sultanpur Lodi.

## 1.2 Objectives of the study

Objectives of the present study can be stated as following:

1. To evaluate performance of the STP in question.
2. To analyse the design of the STP in question.

## 1.3 Importance and usefulness of the work

Waste Stabilization Ponds (WSP) have proven to be effective alternative for treating wastewater, and a low energy technology that use natural processes, in contrast to complex high-maintenance treatment systems. They will hopefully lead to more ecologically-sustainable wastewater treatment in the future. WSPs have the capability of meeting the demand for a high percentage removal of pathogenic organisms as compared to conventional technologies. Waste stabilization pond systems are widely believed as appropriate for sewage treatment in the developing countries. Waste stabilization ponds in their original format have many limitations and making them not very suitable. In the light of this, waste stabilization ponds based Sewage Treatment Plant has been worked on in this study.

Performance evaluation and design analysis of the STP has been carried out to comment on the proficiency of design, capacity and performance of the STP in question. Importance of the work carried out here can be stated as, it is the only mean by which pond design can be optimised for local conditions. Evaluation of pond performance and behaviour is extremely useful as it provides information on how under loaded or overloaded the system is, and thus by how much, if any, the loading on the system can be safely increased as the community it serves expands, or whether further ponds (in parallel or in series) are required. This work is related to and part of the Satluj river water quality monitoring project, and the results of the study will be submitted to NRCD, MOEF, GOI. For NRCD the work will prove useful for improving the performance of the STPs. Further, the information will prove an important input for future STP design. It can prove quite useful as compiled information for ready reference and use.

## 1.4 Contents of the report

Present thesis work on performance evaluation of Waste Stabilization Pond based Sewage Treatment Plant 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 Waste Stabilization Ponds in two sections. Their treatment process, constituents units are discussed in one section and their design methods are discussed in another 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 herein under two sections: performance analysis and design analysis.

**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.

## 1.4 Limitations of the study

The present study was carried out over a four month period (February to May, 2009), while proper performance evaluation required monitoring the STP both during winter and summer critical months. Monitoring involved grab sampling, while this type of study required composite sampling and repetition for at least 5 to 8 times. Grab sampling might have introduced inconsistencies and errors into the results. Empirical equations being used and not knowing the set of empirical equations actually used by the designer made specially the design analysis very difficult and inconclusive. It might have been possible to calibrate the empirical equations with the help of STP monitoring data for making them appropriate to the site conditions.

# CHAPTER – 2
# LITERATURE REVIEW

## 2.1 Waste Stabilisation Pond Systems

Waste stabilisation pond (WSP) technology offers important advantages and interesting possibilities when viewed in the light of sustainable energy and carbon management. WSP systems stand out as having significant advantages due to simple construction; low (or zero) operating energy requirements; and the potential for bio-energy generation through sunlight-powered aerobic treatment and disinfection; moreover, energy may be cost-effectively produced as biogas from anaerobic ponds. Conventional WSP requires little or no electrical energy for aerobic treatment as a result of algal photosynthesis. Sunlight enables WSP to disinfect wastewaters very effectively without the need for any chemicals or electricity consumption and their associated $CO_2$ emissions. WSPs have a relatively small carbon footprint considering the low-cost, energy production opportunities of anaerobic ponds and the potential of algae as a biofuel (Craggs et al. 1999; De Garie et al. 2000; Mara 2004; Park & Craggs 2007). The energy and carbon emission savings gained over electromechanical treatment systems are immense. Furthermore, WSP can be utilised as $CO_2$ scrubbers because algal photosynthesis consumes $CO_2$.

Waste stabilization ponds (WSPs) are usually the most appropriate method of domestic and municipal wastewater treatment in developing countries, where the climate is most favourable for their operation. WSPs are man-made earthen basins having low-cost, low-maintenance, highly efficient, entirely natural and highly sustainable. The only energy they use is direct solar energy, saving expenditure on electricity and more skilled operation. They do require much more land than conventional electromechanical treatment processes such as activated sludge – but land is an asset which increases in value with time. Natural biological and physical processes are used to treat the wastewater to the required effluent standard. The quality of the discharged effluent depends on both the process design and the physical design of the WSP.

Each type of WSP carries out a unique function. Anaerobic and facultative ponds are designed for BOD removal, while maturation ponds are designed to remove excreted pathogens. Well designed WSP, provided they are constructed and maintained properly and not overloaded, will provide a high level of wastewater treatment for very many years. Other wastewater treatment processes can do this as well, of course, but not at the low cost of WSP, nor with their simplicity.

Prior to treatment in the WSPs, the wastewater is first subjected to preliminary treatment (screening and grit removal) to remove large and heavy solids. Basically, primary treatment is carried out in anaerobic ponds, secondary treatment in facultative ponds, and tertiary treatment in maturation ponds. Anaerobic and facultative ponds are for the removal of organic matter (BOD) both soluble and suspended, Vibrio cholerae and helminth eggs; and maturation ponds for the removal of faecal bacteria and nutrients (nitrogen and phosphorus). Waste Stabilisation pond technology is the most cost effective wastewater treatment technology for the removal of pathogenic microorganisms. The treatment is achieved through natural disinfection mechanisms (Mara et al., 1995).

Anaerobic ponds are commonly 2-5m deep. They are the smallest units in the series are sized according to their volumetric organic loading (100 to 350 g $BOD_5/m^3$ day) depending on the design temperature. There is no dissolved oxygen present and the redox potential is negative. Anaerobic ponds work extremely well in warm climates: around 60 % $BOD_5$ removal at 20°C and over 70 % at 25°C can be achieved in a properly designed pond. Odour nuisance from anaerobic ponds, typically due to hydrogen sulphide, is a concern for design of anaerobic ponds. However, odour is not a problem provided that the anaerobic pond is properly designed and the sulphate concentration in the raw wastewater is less than 300 mg $SO_4^{2-}/l$ (Gloyna and Espino, 1969).

Facultative ponds follow anaerobic ponds in a WSP system. They are usually 1-2 m deep and are geometrically designed to have high length-to-width ratio (up to 10:1) to simulate a plug flow regime (Mara et al. 1992). They are designed for BOD removal on the basis of relatively low surface loading (100-400kg BOD/ha.d) to permit the development of a healthy algal population as the oxygen for BOD removal is generated by algal photosynthesis. The algae give facultative ponds a dark green colour. Ponds may occasionally appear red or pink (especially when overloaded) due to the presence of anaerobic purple sulphide-oxidising photosynthetic bacteria (Mara and Pearson, 1986).

Photosynthetic activity of the algae results in a diurnal variation of dissolved oxygen (DO) concentration and pH. DO concentration can rise to more than 20 mg/l (i.e., highly supersaturated conditions) and pH to more than 9.4 (these are both important factors in the removal of faecal bacteria and viruses; Curtis et al., 1992). Ammonia and sulphide toxicity have been observed to be pH-dependent (Konig et al. 1987). As the pH of a facultative pond increases, the unionized form of ammonia increases while sulphide production decreases. The effect of this toxicity is to inhibit algae growth and production and these mechanisms are thought to be self-sustaining (Konig et al. 1987). In primary facultative ponds BOD removal of about 70 % on an unfiltered basis and more than 90 % on a filtered basis can be achieved.

Maturation 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.

Treatment efficiency of waste stabilization pond systems is often compromised by poor hydraulic design. Problems such as hydraulic short-circuiting are prevalent in many ponds. Improved hydraulic design can reduce the concentration of pollutants that escape treatment and therefore improve the water quality of the receiving environment. Pond hydraulic behaviour is influenced by the inlet/outlet configuration, baffles and wind, but design information relating to these factors is still very limited (Shilton and Harrison, 2003).

Shilton and Harrison in 2003 reviewed Guidelines for the Improved Hydraulic Design of Waste Stabilisation Ponds. According to the authors, inlet design can have a significant influence on the flow regime in a pond. Poorly considered positioning of the inlet and the outlet can create hydraulic short-circuiting problems. Extensive testing undertaken on a wide range of baffle configurations showed how short stub baffles could provide improvements similar to longer "traditional" baffle designs and offer significant savings in construction costs.

Waste stabilisation ponds (WSP) are now regarded as the method of first choice for the treatment of wastewater in many parts of the world. WSP are very widely used for small rural communities (Boutin et al., 1987; Bucksteeg, 1987). In developing countries and specially in tropical regions sewage treatment by WSPs has been considered an ideal way of using natural processes to improve sewage effluents. Many characteristics make WSP substantially different from other treatment technologies.

WSP effluents bring additional benefits since the algae they contain add organic content to soil and improve soil structure and its water holding capacity. Waste Stabilisation Ponds (WSP) has high concentration of total suspended solids (TSS) in their effluent. These solids comprise suspended algal cells as their constituents. These algae can impose serious constraints for some potential areas of effluent reuse like agricultural applications (Saidam et al., 1995).

Treated wastewater is a reliable water resource, especially for periodic droughts and in arid areas. A study was implemented by Naddafi et al., 2004 to investigate the full scale application of stabilization ponds effluent of southern Hovaizeh Wastewater Treatment Plant located in Khuzestan Province for irrigation use to assess the health effects and feasibility of crop irrigation. Two experimental plots, each of about 0.5 ha. were constructed. One of the plots was irrigated by stabilization pond effluent and the other by Nisan River water. Basic parameters for both the plots, such as, type of cultivated crops, amount of fertilizer use and lack of soil contamination have been similar. The only difference was the type of water applied for irrigation. Results showed the growth rate and quality of crops were increased by using of stabilization pond effluent in comparison with Nissan River water.

Potential of natural treatment systems for the reclamation of domestic sewage in irrigated agriculture was studied by Giokas et al., 2005. Various systems consisting of waste stabilization ponds, shallow algal ponds and water hyacinth ponds were operated in parallel, series or mixed arrangement in order to find the optimum setting that enables efficient effluent quality to be reused for agricultural purposes. The results indicate that waste stabilisation ponds were very efficient for wastewater treatment, achieving an effluent quality able to be used for restricted irrigation. However coliform numbers were not always consistent with the proposed guidelines. To cope with the problem, a modified configuration employing water hyacinth ponds as the final pond was proposed.

Routine monitoring of the quality of final effluent of a pond system permits a regular assessment to be made of whether the effluent is complying with the local discharge or reuse standards (Mara, 1997). The evaluation of pond performance and behaviour is extremely useful as it provides information on how under loaded or overloaded the system is, and thus by how much, if any, the loading on the system can be safely increased as the community it serves expands, or whether further ponds (in parallel or in series) are required. It also indicates how the design of future pond installations in the region might be improved to take account of local conditions. A full evaluation of the performance of a WSP system is a time consuming and expensive process, it is the only means by which pond designs can be optimised for local conditions (Mara, 1997).

## 2.2 Design methods for WSP

The required and accepted quality of discharged wastewater is characterized by effluent limits. Hence, prior to design, these limits must be known since they will be used as the water quality design objectives. The general standards for the discharge of treated wastewaters into inland surface waters are given in the Environment Protection Rules (CPCB, 1996). The more important of these for WSP design are: BOD 30 mg/l (non-filtered), Suspended solids 100 mg/l and Total N 100 mg N/L.

Mara et al. (1992), Mara (1997) and Arthur (1983) list the most important input design parameters of WSP as temperature, net evaporation, design flow, per capita BOD and faecal coliform concentration. Helminth eggs are required if the effluent is to be reused for restricted crop irrigation. The US Environmental Protection Agency (1983), Reed at al. (1988) and Shilton and Harrison (2003) observed that poor hydraulic design reduces the theoretical hydraulic retention time due to short-circuiting and the formation of dead spaces. This results in incomplete removal of the wastewater pollutants. The resulting treated effluent then fails to meet the required standards.

It can be concluded that the performance of a WSP system depends on robust process and physical design methods. The process design should assume a realistic hydraulic flow regime that can be achieved by the physical design.

### 2.2.1 Design principles for anaerobic ponds

An empirical approach is the recommended method for designing anaerobic ponds. These are normally designed based on permissible volumetric organic loading rate ($\lambda_v$) expressed in g/m$^3$.d of BOD (Arthur, 1983; Mara, 1976; Mara and Pearson, 1986; Meiring et al., 1968). Meiring et al. (1968) proposed that permissible volumetric organic loading rates should be within a range of 100-400 g/m$^3$.d to ensure that anaerobic ponds function as intended. Volumetric organic loading rate of less than 100 g/m$^3$.d can cause anoxic reactions in anaerobic ponds. The upper limit of 400 g/m$^3$.d is established to avoid the risk of odour produced by hydrogen sulphide gas ($H_2S$).

It has been observed that BOD removal in an anaerobic pond is directly proportional to pond temperature. Mara and Pearson (1986) and Mara et al. (1997a) proposed the relationship between design temperature and design BOD removal for anaerobic ponds. Table 2.1 lists suitable design volumetric organic loading rates for various temperature ranges. Here the design temperature is the mean temperature of the coldest month.

**Table-2.1: Design values of permissible volumetric BOD loadings and percent BOD removals in anaerobic ponds at various temperatures.**

| Temperature (°C) | Volumetric loading (g/m3d) | BOD removal (%) |
|---|---|---|
| <10 | 100 | 40 |
| 10-20 | 20*T* -100 | 2*T* +20 |
| 20-25 | 10T+100 | 2*T* +20 |
| >25 | 350 | 70 |

*T* = temperature, °C.

*Source*: Mara and Pearson (1986) and Mara *et al.* (1997a).

Anaerobic pond volume is related to the BOD of the raw wastewater ($L_i$, mg/l), mean wastewater flow (Q, $m^3$/d) and volumetric organic loading rate ($\lambda_v$) by;

$$V = \frac{L_i Q}{\lambda_v} \quad (2.1)$$

Where

$L_i$ = influent BOD ($g/m^3$)

Q =flow, ($m^3$/d)

V = pond volume, ($m^3$)

$\lambda_v$ =volumetric organic loading rate ($g/m^3$.d)

Depths of anaerobic ponds are kept high (3.5-5m) in order to guarantee the predominance of anaerobic conditions. Anaerobic ponds are square or slightly rectangular, with typical length/breadth (L/B) ratios of 1:3. Once the removal efficiency (E) has been estimated from table-2.1 the effluent concentration ($BOD_{eff}$) of the anaerobic pond is calculated by:

$$E = (S_o - \mathrm{BOD_{eff}}).100/S_o \tag{2.2}$$

or

$$BOD_{eff} = (1 - E\,/\,100).S_o \tag{2.3}$$

Where:

$S_o$ = influent total BOD conc. (mg/L)

$BOD_{eff}$ = effluent total BOD conc. (mg/L)

### 2.2.2 Design principles of facultative ponds

The design of facultative ponds focuses on BOD removal. Mara (1976) and Marecos do Monte and Mara (1987) describe 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 WSP. The rational design approach models the ponds performance by using kinetic theories of biochemical reactions in association with the hydraulic flow regime.

#### *2.2.2.1 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.

$$\frac{S_E}{S_0} = \left[\frac{1}{1+k.t}\right]^n \tag{2.4}$$

$$k_T = k_{20}\theta^{(T-20)} \tag{2.5}$$

Where;

$S_e$ = effluent BOD (mg/L)

$S_o$ = influent BOD (mg/L)

k = completely mixed flow $1_{st}$ 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 $S_e$. Mara (1976) recommends that $S_e$ should be in the range of 50-100 mg/l (usually 70 mg/l). Hydraulic retention time ($t$) can be determined by;

$$t = \left(\frac{S_o}{S_e} - 1\right)\left(\frac{1}{k}\right) \tag{2.6}$$

The pond area is then calculated by using equation;

$$A_f = \frac{Q.t}{H} \tag{2.7}$$

Where;

Q = design flow ($m^3$/d)

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

$A_f$ = area of facultative pond ($m^2$)

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:

$$d = \frac{D}{ul} \tag{2.8}$$

Where;

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

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 design new ponds, literature presents some empirical relationships for preliminary estimation of d:

- Polprasert and Batharai (1983)

$$d = \frac{0.184.t.v(B+2.H)^{0.489}.B^{1.511}}{(L.H)^{1.489}} \tag{2.9}$$

- Yanez (1993)

$$d = \frac{L/B}{-0.261+0.254.(L/B)+1.014.(L/B)^2} \tag{2.10}$$

- Von Sperling (1999)

$$d = \frac{1}{L/B} \tag{2.11}$$

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 ($m^2/d$)

$t$ = 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.

### *2.2.2.2 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:

$$\frac{S_e}{S_o} = e^{-kt} \qquad (2.12)$$

Where;

$S_e$ = effluent BOD (mg/L)

$S_o$ = 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;

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

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 $\theta$ is taken as 1.06. The plug-flow model is used to calculate the retention time required for specified BOD removal requirements.

### *2.2.2.3 Dispersed flow regime model*

In reality, the hydraulic regime in a stabilisation 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:

$$\frac{S_e}{S_o} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}} \qquad (2.14)$$

$a = \sqrt{1 + 4k.t.d}$

$$d = \frac{D}{Ul}$$

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

Where;

$S_e$ = effluent BOD (mg/l)

$S_o$ = influent BOD (mg/l)

k = 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 ($m^2/d$)

v = 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:

$$d_f = \frac{0.184[tv(W+2H)]^{0.489}W^{1.511}}{(LH)^{1.489}} \quad (2.15)$$

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.

#### 2.2.2.4 *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 $\lambda_s$ (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;

$$A_f = \frac{10 L_i Q}{\lambda_s} \tag{2.16}$$

Where

$L_i$ = influent BOD (kg $BOD_5$/d)

Q = flow rate ($m^3$/d)

$\lambda_s$ = surface BOD loading (kg BOD/ha/d)

Design value of $\lambda_s$ 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:

$$\lambda s = 350 \times (1.107 - 0.002 \times T)^{(T-25)} \tag{2.17}$$

Where:

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

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

$$t = \frac{A_f H}{Q_{avg}} \tag{2.18}$$

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.

$$Q_{average} = (Q_{influent} + Q_{effluent})/2 \tag{2.19}$$

$$t = \frac{A_f.H}{2(Q_i + Q_e)} \tag{2.20}$$

Mass balance for flow is like;

$$Q_{effluent} = Q_{influent} + Q_{precipitation} - Q_{evaporation} - Q_{infiltration} \tag{2.21}$$

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

$$Q_{net\ evap.} = Q_{evap.} - Q_{precipitation} \tag{2.22}$$

$$Q_{e.} = Q_i - Q_{net\ evap.} - Q_{infiltration} \tag{2.23}$$

If infiltration is negligible, $Q_e$ is given by:

$$Q_e = Q_i - 0.001 A_f\, e \tag{2.24}$$

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

$$t = \frac{2.A_f.H}{(Q_i + 0.001 A_f e)} \tag{2.25}$$

In primary facultative ponds treating domestic sewage detention times usually vary between 15 and 45 days. To minimise 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 .

### 2.2.3 Design of maturation ponds

#### *2.2.3.1 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:

$$\frac{N_e}{N_i} = \frac{1}{\left[1+k_b\frac{\theta}{n}\right]^n} \tag{2.26}$$

Where;

$N_e$ = number of effluent faecal coliform per 100ml

$N_i$ = 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, Equation becomes:

$$\frac{N_e}{N_i} = \frac{1}{(1+k_b t_a)(1+k_b t_f)(1+k_b t_m)^n} \tag{2.27}$$

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:

$$\frac{N_e}{N_i} = \frac{1}{(1+k_b t_a)(1+k_b t_f)(1+k_b t_{m1})(1+k_b t_{m2})(1+k_b t_{mn})} \tag{2.28}$$

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

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

Where;

T = temperature (°C)

$\theta$ = temperature coefficient

The value of $\theta$ 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)

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

$\lambda_{sf}$ = 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 modelling faecal coliform removal is not realistic. Shilton and Harrison (2003) have suggested that an ideal 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;

$$\frac{N_e}{N_i} = \frac{4ae^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}} \qquad (2.29)$$

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

Where;

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

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

$k_b$ =bacteria die – off coefficient ($d^{-1}$)

t = detention time (d)

d = dispersion number (dimensionless)

### *2.2.3.2 Design of maturation pond for Nutrient Removal*

**Nitrogen:** Pano and Middlebrooks (1982) proposed a model for the removal of ammoniacal 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:

$$C_e = \frac{C_o}{1+\left[(A/Q).(0.0038+0.000134.T).e^{(1.041+0.044.T).(pH-6.6)}\right]} \qquad (2.30)$$

For temperature more than 20°C the equation is:

$$C_e = \frac{C_o}{1+\left[5.035\times10^{-3}.(A/Q).e^{(1.540\times(pH-6.6))}\right]} \qquad (2.31)$$

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, ($m^2$)

Q = wastewater flow rate, ($m^3$/d)

T = temperature, ($^{\circ}$C)

pH = 7.3exp (0.0005A) [where A = influent alkalinity (mg $CaCO_3$/L)

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

$$C_e = C_o \exp\{-[0.0064(1.039)^{T-20}][t + 60.6(pH - 6.6)]\} \quad (2.32)$$

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 $CaCO_3$/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 organic.

# CHAPTER – 3
# METHODLOGY

## 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:

- Getting background information on the STP.
- Monitoring
- Performance evaluation
- Design analysis

## 3.2 Getting background information of the STPs:

For completing this work element the STP in question was visited, surveyed and people working at the site were discussed with to know the scheme of treatment and dimensional and capacity details of all the facilities. Process flow diagram of the STP was made from the understanding obtained (see figure 3.1).

## 3.3 Monitoring:

The monitoring is supposed to support performance evaluation and design analysis of the STP in question. For facilitating this, the sampling locations were identified and the parameters for which the samples should be analysed were decided. The monitoring involved collection of grab samples on a monthly basis over four months (February to May 2009). Date and time of sampling, wastewater flow rate through the STP and temperature of both ambient air and wastewater were recorded at the time of sampling. The samples were collected in three containers of which one is a sterilized grass bottle (meant for MPN test). The collected samples were brought to the Environmental Laboratory of the Thapar Uuniversity within a few hours of collection, and analysed for the parameters indicated in table-3.1 until the analysis was over the samples were stored in a deep freeze. Methods followed for the analysis are indicated in table-3.2.

**Table 3.1 Parameters to be characterized at different sampling points:**

| Parameter | Sampling points | | | |
|---|---|---|---|---|
| | Inlet (P1) | Anaerobic Pond outlet (P2) | Facultative Pond Outlet (P3) | Outlet (P4) |
| pH | ✓ | ✓ | ✓ | ✓ |
| Temp. | ✓ | ✓ | ✓ | ✓ |
| BOD | ✓ | ✓ | ✓ | ✓ |
| COD | ✓ | ✓ | ✓ | ✓ |
| MPN | ✓ | ✓ | ✓ | ✓ |
| TSS | ✓ | ✓ | ✓ | ✓ |
| TDS | ✓ | | | ✓ |
| TKN | ✓ | | ✓ | ✓ |
| Nitrate+Nitrite | | | ✓ | ✓ |
| Chloride | ✓ | | | |
| Sulphate | ✓ | | | |
| Alkalinity | ✓ | | | ✓ |

**Table 3.2: Analytical techniques for testing of wastewater parameters:**

| Sr. No. | Parameter | Method | References |
|---|---|---|---|
| 1 | pH | Electrometric method | APHA (4500-h+ : B) |
| 2 | Temperature | Laboratory and field methods | APHA (1999) “manual standard method” 20th edition (2550: B) |
| 3 | Chemical Oxygen Demand (COD) | closed reflux method | APHA (1999) “manual standard method” 20th edition (5220: B) |
| 4 | Biochemical Oxygen Demand (BOD) | 5 day BOD test | APHA (1999) “manual standard method” 20th edition (5210: B) |
| 5 | Alkalinity | Titration method | APHA (1999) “manual standard method” 20th edition (2320: B) |
| 6 | TSS | Total suspended solids dried at 103-105°C. | APHA (1999) “manual standard method” 20th edition (2540: D) |
| 7 | TDS | Total dissolved dried at 180°C. | APHA (1999) “manual standard method” 20th edition (2540: C) |
| 8 | TS | Total solids dried at 103-105°C. | APHA (1999) “manual standard method” 20th edition (2540: B) |
| 9 | Ammonical nitrogen | Preliminary distillation step, titrimetric method. | APHA (1999) “manual standard method” 20th edition (4500-NH3: B, E) |
| 10 | Organic nitrogen | Macro kjeldahl method. | APHA (1999) “manual standard method” 20th edition (4550- org: B) |
| 11 | Nitrate nitrogen | Cadmium reduction method | APHA (1999) “manual standard method” 20th edition (4500-No3: E) |
| 12 | Nitrite nitrogen | Colorimetric method | APHA (1999) “manual standard method” 20th edition (45000-No2: B) |
| 13 | Total phosphorous | Stannous chloride method | APHA (1999) “manual standard method” 20th edition (4500-P: B, D) |
| 14 | MPN | Serial dilution method | APHA (1999) “manual standard method” 20th edition (9221:B,C) |
| 15 | Sulphates | Gravimetric method | APHA (1999) “manual standard method” 20th edition (4500-$SO_4^{2-}$: D) |

| | | | |
|---|---|---|---|
| 16 | Chlorides | Argentometric method | APHA (1999) "manual standard method" 20th edition ($4500\text{-}Cl^-$: B) |
| 17 | VSS | TSS dried at 550°C | APHA (1999) "manual standard method" 20th edition (2540: G) |
| 18 | VFA | Distillation method | APHA (1999) "manual standard method" 20th edition (5560: C) |
| 19 | Heavy metals (Cr, ZN, Ni, Pb, Fe) | Atomic Absorption method | APHA (1999) "manual standard method" 20th edition (3500: A,B) |

## 3.4 Performance evaluation:

Performance evaluation of the STP was done while using the monitoring data both at whole plant level and at the individual treatment units level. Performance of the individual units was assessed against the parameters for which the unit in question was designed and used. Performance evaluation was also done for the coincidental removal of pollutants from the wastewater. Performance inhibiting parameters anticipated in the wastewater were also looked into. By knowing the inlet and outlet concentration of different parameters, plant level removal efficiencies for various parameters were calculated.

## 3.5 Design analysis:

Design analysis of the STP was done against commonly used design equations available from the literature. Very often design of the STP was compared with typical values also available from the literature. Design analysis has proved very difficult due to the use of empirical formulae and thumb rules used in the design. the design analysis also involved comparison of actual values with the values obtained from the design equations. The design equations and the typical values against which the design analysis was carried out are given below:

### 3.5.1 Anaerobic pond:

Design basis for anaerobic ponds is volumetric loading rate of organic matter. Volumetric organic loading rate is calculated using following equation:

$$\lambda_v \text{ (g/m}^3\text{.d)} = \frac{L_i Q}{V} \qquad (3.1)$$

Where;

$L_i$ = influent BOD (g/m$^3$)

Q =flow, (m$^3$/d)

V = pond volume, (m$^3$)

BOD removal is directly proportional to pond temperature. For estimating BOD removal efficiency following equation is used

$$E = (S_o - \text{BOD}_{\text{eff}}).100/S_o \qquad (3.2)$$

Where:

$S_o$ = influent total BOD conc. (mg/L)

$BOD_{eff}$ = effluent total BOD conc. (mg/L)

Coliform removal in anaerobic ponds is supposed to occur only by coincidental removal and it can be estimated using following equation

$$\frac{N_e}{N_i} = \frac{1}{(1+k_b t)} \qquad (3.3)$$

### 3.5.2 Facultative pond:

Surface BOD loading is the design basis for facultative ponds. It is measured in units of kg BOD/ha/d and calculated as:

$$\lambda_s = \frac{10 L_i Q}{A_f} \qquad (3.4)$$

Where

$L_i$ = influent BOD (kg $BOD_5$/d)

Q = flow rate ($m^3$/d)

$A_f$ = Area of facultative pond ($m^2$)

Design value of $\lambda_s$ is calculated using following equation and compared to the observed value.

$$\lambda s = 350 \times (1.107 - 0.002 \times T)^{(T-25)} \quad (3.5)$$

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

Removal rate of organic matter is calculated as average value obtained from following 4 equations:

$$\lambda r = 0.725\ \lambda_s + 10.75 \quad (3.6)$$

$$\lambda r = 0.79\ \lambda_s + 2 \quad (3.7)$$

$$\lambda r = 0.83679\ \lambda_s - 4.86 \quad (3.8)$$

$$\lambda r = 0.956\ \lambda_s - 1.31 \quad (3.9)$$

Hydraulic retention time (t) of facultative ponds is then calculated as and matched with the observed retention times.

$$t = \frac{A_f H}{Q_{avg}} \quad (3.10)$$

Where;

H = pond depth (usually 1.5m)

Q = average flow, ($m^3$.d)

$A_f$ = Area of facultative pond ($m^2$)

**3.5.2 Maturation ponds:**

Coliform removal is the design parameter for maturation ponds and efficiency of pathogen removal for maturation pond can be calculated as:

$$\frac{N_i}{N_e} = \frac{1}{\left(1 + k_b \frac{\theta}{n}\right)^n} \quad (3.11)$$

Overall efficiency of WSP can be calculated as:

$$\frac{N_e}{N_i} = \frac{1}{(1+k_b t_a)(1+k_b t_f)(1+k_b t_m)^n} \qquad (3.12)$$

Where;

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

$N_0$ = coliform conc. in influent (org/100ml)

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

$t$=retention time

$k_b$= coliform die–off coefficient

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

Where $k_{b20}$ is coliform die–off coefficient at 20°C (2.6 as reported by Marais, 1974), T is temperature (°C) and $\theta$ is temperature coefficient (1.19, as reported by Marais, 1974)

The following three guidelines (Marais, 1974) have also been checked for in the design analysis of the maturation ponds:

2. $t_m < t_f$

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 (d)

$t_f$ = hydraulic retention time in secondary facultative pond (d)

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

$\lambda_{sm1}$= surface BOD loading in first maturation pond (kg/ha/d)

$\lambda_{sf}$ = surface BOD loading in facultative pond (kg/ha/d)

**Ammonical nitrogen removal**

Expected ammonical nitrogen removal can be calculated using following equations:

When temperature is below 20°C

$$C_e = \frac{C_o}{1+\left[(A/Q).(0.0038+0.000134.T).e^{(1.041+0.044.T).(pH-6.6)}\right]} \quad (3.13)$$

When temperature is more than 20°C

$$C_e = \frac{C_o}{1+\left[5.035\times10^{-3}.(A/Q).e^{(1.540\times(pH-6.6))}\right]} \quad (3.14)$$

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, ($m^2$)

Q = wastewater flow rate, ($m^3$/d)

T = temperature, (°C)

pH = 7.3exp (0.0005A)

[Where A = influent alkalinity (mg $CaCO_3$/L)]

**Total nitrogen removal**

For estimating total nitrogen removal following equation is used in case of facultative and maturation ponds (Reed, 1995):

$$C_e = C_o \exp\{-[0.0064(1.039)^{T-20}][t+60.6(pH-6.6)]\} \quad (3.15)$$

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 $CaCO_3$/L)

## 3.6 Sewage Treatment Plant being studied:

Waste Stabilization Pond (WSP) system based sewage treatment plant of 2.6 MLD capacity installed and commissioned in **Sultanpur Lodhi** by Punjab Water Supply and Sewerage Board under the Satluj River Action Plan was studied. Schematic diagram of the STP is given in figure-3.1. The system included the following units and facilities:

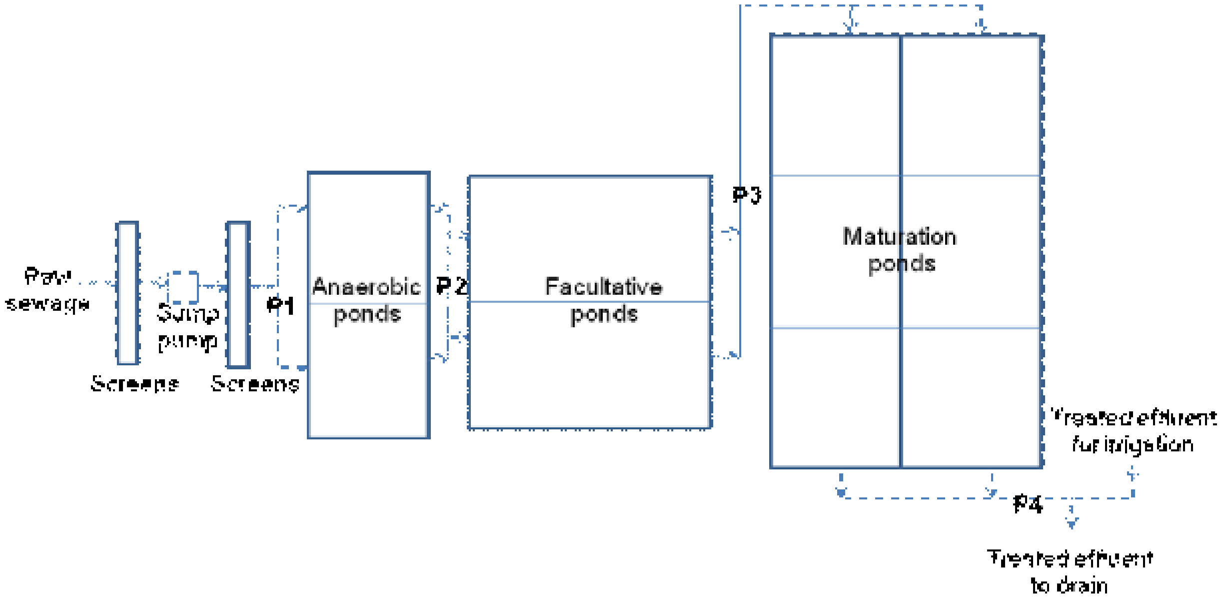
- Bar screen
- Sewage collection sump
- Raw sewage pumps
- Bar screen
- Anaerobic ponds (2 numbers connected parallely)
- Facultative ponds (2 numbers connected parallely)
- Maturation ponds (2 streets each of three ponds connected in series)

Dimensional and capacity details of these units are given in table 3.3. Schematic process flow diagram of the STP is shown in figure-3.1.

**Table 3.3: Dimensional and capacity details of various units and facilities of the STP**

| **Units** | **Dimensions** |
|---|---|
| Screen | Manual bar screen of 6mm thick bars with 40 mm spacing. |
| Pumps | 3 pumps, 2 of 15 hp capacity and 1 of 20 hp capacity. |
| Anaerobic ponds | 2 ponds each of 40.5m length, 23m width, 5.1 m liquid depth and 0.61 m freeboard. |
| Facultative ponds | 2 ponds each of 136 m length, 55m width, 1.4m liquid depth and 0.61m freeboard. |
| Maturation ponds | 2 streets of three ponds connected in series, each pond of 75.5 m length, 28m width, 1.3m liquid depth and 0.61 m freeboard. |

Wastewater conveyed to the STP is collected into a raw sewage sump through a bar screen and from there it is pumped with the help of 2 of the 3 raw sewage pumps and loaded to the anaerobic ponds after passing through a bar screen (preliminary treatment). Wastewater enters the two anaerobic ponds through a pipe at a depth of 2m from the top. The pumped raw sewage is metered with the help of an online flow meter. Screenings accumulating at the bar screens are manually removed and disposed off at regular intervals. Primary treatment of the sewage occurs in the anaerobic pond. Suspended biodegradable and non-biodegradable solids are removed and sludge stabilization occurs anaerobically. Wastewater from the anaerobic pond is allowed flow under gravity into the two facultative ponds. Here secondary treatment of the wastewater occurs. Algal photosynthesis and surface re-aeration provide the needed dissolved oxygen. Algal cells live the pond in symbiotic association with the heterotrophic bacteria bio-oxidizing the soluble biodegradable organic matter. Facultative have bottom anaerobic, middle facultative and top aerobic zones. Treated wastewater from the facultative pond is allowed to flow into the maturation ponds. In the maturation ponds pathogen number of the wastewater is dramatically reduced. Even nutrients and to a limited extent BOD are also removed in the maturation ponds. Treated effluent of the maturation is either used for irrigation or allowed to flow into the West Bein, a tributary to the river Satluj. There are provisions for bypassing the anaerobic ponds and also the maturation ponds. Water from facultative pond outlet by-passing the maturation ponds is often used for irrigation.




* P1 to P4 are the sampling points for performance monitoring.

**Figure 3.1 Flow diagram of Waste Stabilization Pond based STP**

# CHAPTER - 4
# RESULTS AND DISCUSSION

## 4.1 Introduction

Results obtained from the treatment process monitoring, performance evaluation and design analysis of the STP at Sultanpur Lodhi are presented and discussed in this chapter. The performance evaluation and design analysis study used the data obtained from the monitoring of the STP at four different locations, namely, inlet of the STP (P1), anaerobic pond outlet (P2), facultative pond outlet (P3) and final outlet of the STP (P-4) for over four months period (February to May 2009). The data obtained from the monitoring is given in the annexure -1 and annexure-2 of this M. Tech dissertation. All removal efficiency calculations for the months February to May 2009 and the period from February to May 2009 are given in tables 4.1 to 4.5 and figure 4.1. Figure 4.1 includes only BOD, TSS, total-N, Total-P and MPN removal efficiencies.

## 4.2 Performance analysis

### 4.2.1 Overall performance of the STP

Waste stabilization pond systems comprising of anaerobic pond, facultative pond and maturation ponds are usually designed for the removal of BOD (and COD) and pathogens. Usually BOD is removed to the tune of 90% (<25 mg/L of soluble BOD) and reduction of pathogens (total coliform count) by 4 to 5 log times to <1000/100 ml. Even nutrients are supposed to be removed to <15 mg/L of total nitrogen and <2 mg/L of total phosphorus. Because of algal cell contribution TSS of the treated effluent can be 50-150 mg/L.

BOD and COD removals observed for the WSP system were much lower than the expected (65 % for BOD and 56% for COD), and average BOD of the treated effluent was 68 mg/l which is much above the prescribed standard (30 mg/L). This could be because of algal biomass present in the treated effluent. This is evident from the high TSS (average of 60 mg/L) in the treated effluent. Total coliform count reduction was much below the expected, and the treated effluent was not fit even for restricted irrigation. MPN of the treated effluent was >1, 00,000 for all the months. One of reasons for this could be inefficient functioning of the anaerobic pond and consequent overloading of the facultative ponds.

Observations on nitrogen removals indicated that the grab sampling practiced made the monitoring results not very relevant for the performance evaluation of the STP. In many cases the removal efficiencies calculated were negative. Organic nitrogen removal was estimated as very low and highly varying as against the high removals expected. Very low organic nitrogen concentration in the raw sewage (0.5 to 1.7 mg/L) made the removal efficiency calculation highly erratic. It appears that the sewage treated in the STP is sufficiently putrefied by the time it is loaded to the STP. Presence of significant levels of organic nitrogen in the treated effluent might have been due to the presence of algal cells.

**Table -4.1: Performance of the STP during February 2009**

February 2009 (12:35 AM)

Water temp.:20.1°C Air temp.: 21.2°C

Flow rate: 1.87MLD

| Parameter | Conc. (mg/L) | | Treatment Efficiency (%) | | | |
|---|---|---|---|---|---|---|
| | Inlet | Outlet | Anaerobic pond | Facultative pond | Maturation pond | Overall |
| **BOD** | 230 | 79 | 35 | 40 | 12 | 66 |
| **COD** | 700 | 200 | 57 | **-66** | 60 | 71 |
| **MPN/100ml** | $16x10^7$ | $5x10^5$ | 89 | 18 | 96 | 99 |
| **TSS** | 190 | 100 | 32 | 83 | 16 | 47 |
| **NH4-N** | 36.6 | 38.6 | - | 19 | **-30** | **-5.5** |
| **Org-N** | 1.1 | 0.5 | - | 63 | **-28** | 54.5 |
| **Total-N** | 37.7 | 39.8 | - | 34.9 | **-31** | **-5.57** |
| **Total-P** | 0.08 | 0.063 | **-11** | 10 | 20 | 21.25 |

**Table -4.2: Performance of the STP during March 2009**

March 2009 (12:45 PM)

Water temp.: 22.9°C Air temp.: 32°C

Flow rate: 3.5MLD

| | **Conc. (mg/L)** | | **Treatment Efficiency (%)** | | | |
|---|---|---|---|---|---|---|
| **Parameter** | **inlet** | **Outlet** | **Anaerobic pond** | **Facultative pond** | **Maturation pond** | **Overall** |
| **BOD** | 205 | 106 | 51 | 43 | **-84** | 48.3 |
| **COD** | 450 | 250 | 33 | 66.6 | **-150** | 44 |
| **TSS** | 130 | 40 | 17 | 41 | 36.5 | 69 |
| **MPN/100ml** | $13x10^7$ | $11x10^5$ | 83 | 94 | 91 | 99 |
| **NH4-N** | 24 | 23.5 | - | **-29** | 24 | 2.3 |
| **Org-N** | 0.5 | 0.1 | - | **-33** | 99 | 80 |
| **Total-N** | 24.6 | 24.2 | - | **-19.6** | 50 | 1.2 |
| **Total-P** | 0.25 | 0.03 | **-89** | 9 | 92 | 93 |

**Table -4.3: Performance of the STP during April 2009**

April 2009 (01:05 PM)

Water temp.:26°C Air temp.: 36°C

Flow rate: 3.4MLD

| Parameter | Conc. (mg/L) | | Treatment Efficiency (%) | | | |
|---|---|---|---|---|---|---|
| | inlet | Outlet | Anaerobic pond | Facultative pond | Maturation pond | Overall |
| **BOD** | 164.5 | 51.8 | 62 | 2.4 | 15 | 68.5 |
| **COD** | 250 | 110 | 12 | 40 | 7 | 52 |
| **TSS** | 220 | 10 | 27 | 81 | 66.6 | 95 |
| **MPN/10ml** | $3x10^{7}$ | $22x10^{5}$ | 27 | 23 | 87 | 93 |
| **$NH_3$-N** | 27 | 23 | - | 46 | -56 | 16 |
| **Org-N** | 1.7 | 2.2 | - | | 80 | **-33** |
| **Total-N** | 29.4 | 30.7 | - | 11 | 2 | **-4** |
| **Total-P** | 8.2 | 13 | **-6** | **-11.6** | **-30** | **-45** |

**Table -4.4: Performance of the STP during May 2009**

May 2009 (12:35 PM)

Water temp.: 29°C Air temp.: 38.8°C

Flow rate: 3.3MLD

| Parameter | Conc. (mg/L) | | Treatment Efficiency (%) | | | |
|---|---|---|---|---|---|---|
| | Inlet | Outlet | Anaerobic pond | Facultative pond | Maturation pond | Overall |
| **BOD** | 165 | 36 | 30 | 26 | 58 | 78 |
| **COD** | 250 | 120 | 12 | 41 | 7.6 | 52 |
| **TSS** | 100 | 90 | 35 | **-35.4** | **-2.2** | 10 |
| **MPN/100ml** | $5x10^8$ | $14x10^6$ | 68 | 44 | 84 | 97 |
| **NH4-N** | 18.7 | 18.2 | - | **-0.5** | 3.7 | 2.9 |
| **Org-N** | 1.7 | 1.4 | - | **-** | 92.7 | 16.6 |
| **Total-N** | 20.4 | 20.1 | - | **-87** | 47 | 1.4 |
| **Total-P** | 5.2 | 4.6 | **-75** | 23 | 34 | 49 |

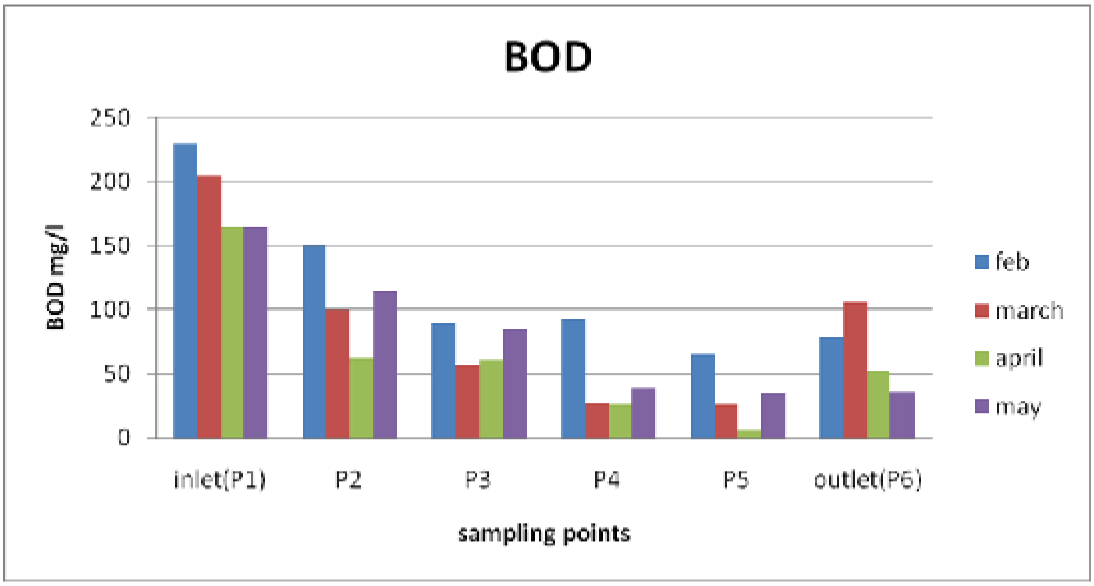



**Figure :4.1(a) BOD removal pattern**

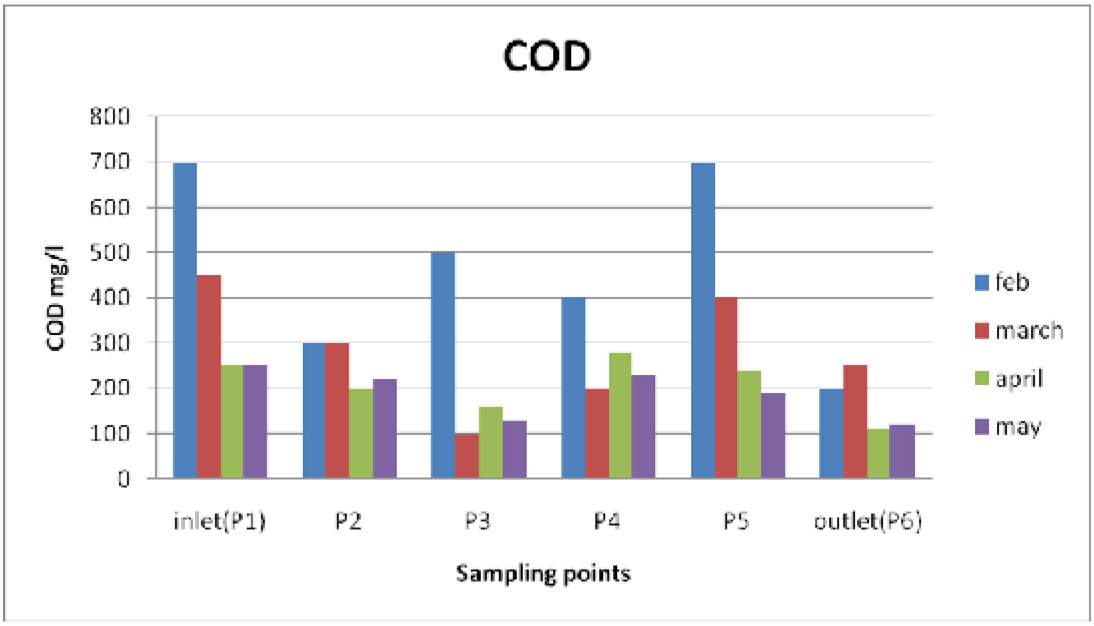



**Figure :4.1(b) COD removal pattern**

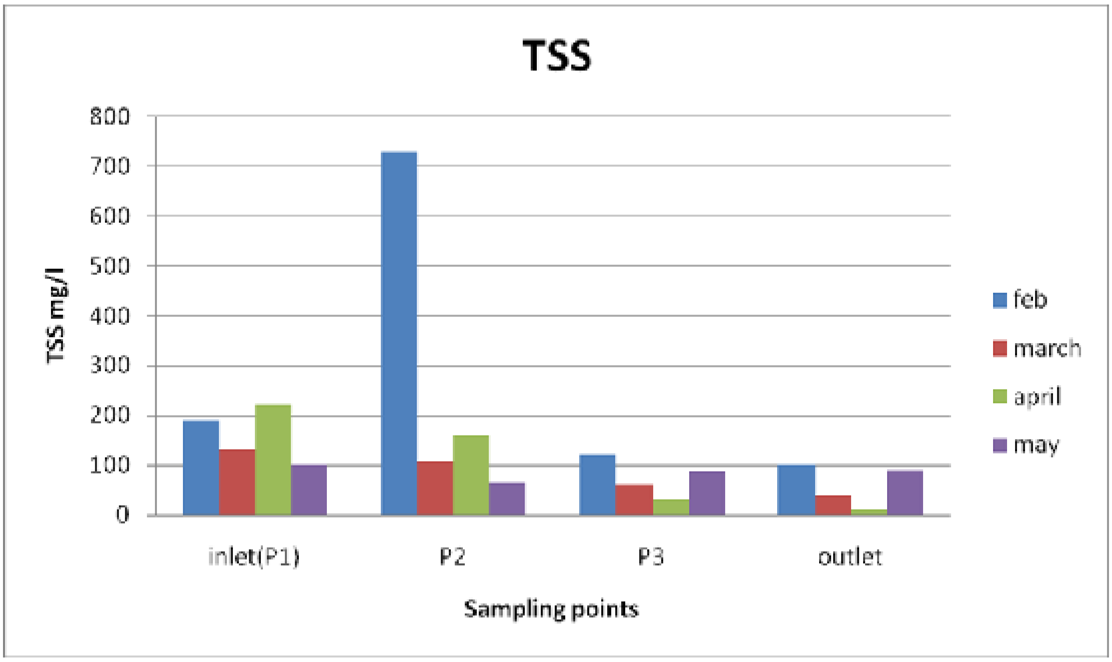



**Figure :4.1(c) TSS removal pattern**

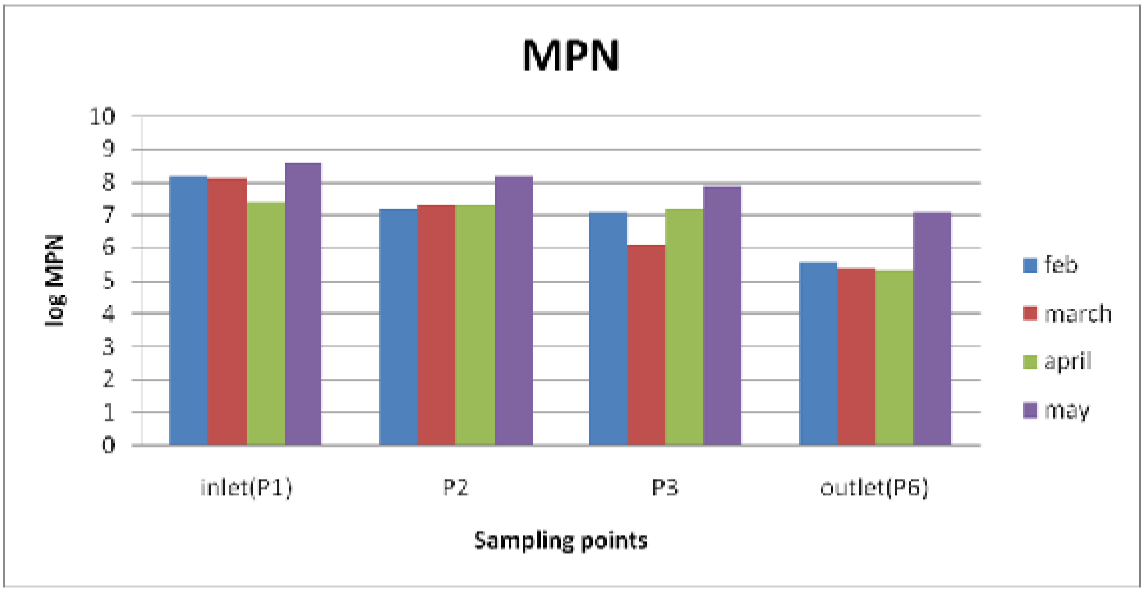



**Figure :4.1(d) MPN removal pattern**

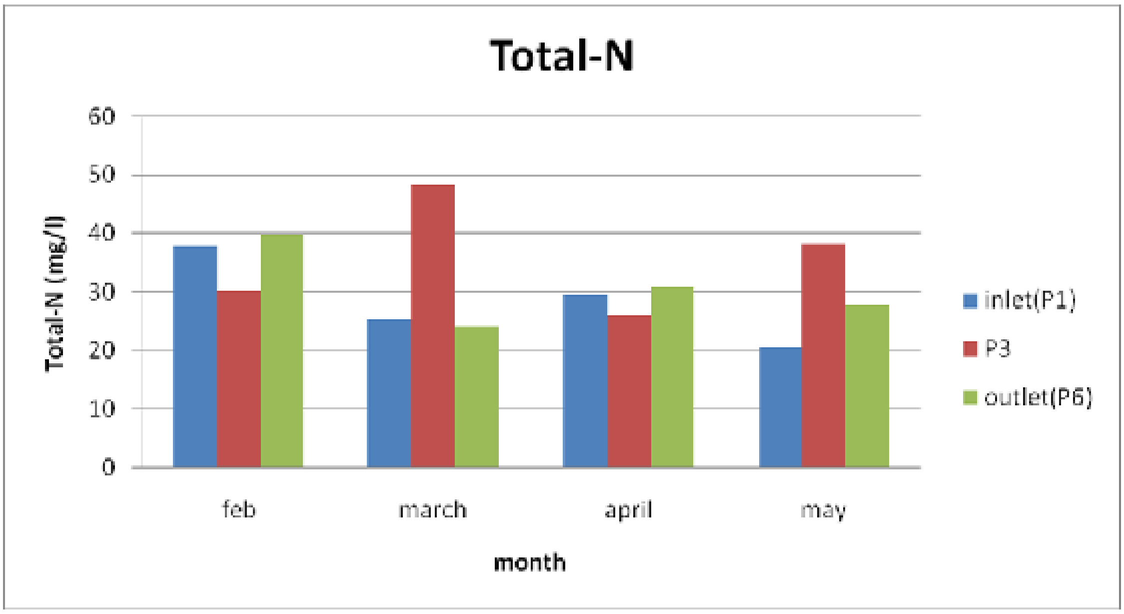



**Figure :4.1(e) Total N removal pattern**

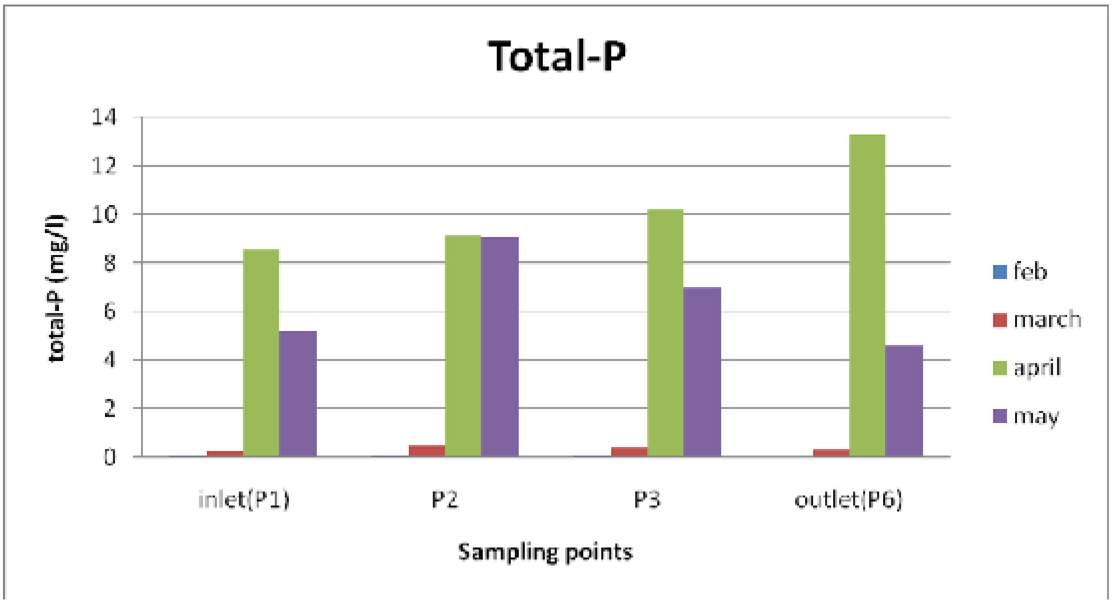



**Figure :4.1(f) Total Phosphorus removal pattern**

**Table -4.5: Average performance of the STP during February to May 2009**

Time of sampling: 12:30 – 1:05 pm

Water temperature: 20.1 -29°C

Air temperature: 21.2 -38.8°C

Sewage flow rate: 1.87 –3.5 MLD

| Parameter | Concentration (mg/L) | | Treatment efficiency (%) | | | |
|---|---|---|---|---|---|---|
| | Inlet | Outlet | Anaerobic Pond | Facultative Pond | Maturation Pond | Overall of the STP |
| **BOD** | 191(±32) | 68 (±30) | 44(±14.7) | 28(±18.5) | 1(±59.8) | 65(±12) |
| **COD** | 412 (±213) | 170 (±66) | 38(±19) | 15(±57) | **-51**(±93) | 56(±11) |
| **TSS** | 160 (±54) | 60 (±42) | 28(±155) | 25(±55.4) | 29(±29) | 55(±36) |
| **MPN/100ml** | $6x10^7$* | $2x10^6$* | 60.6* | 36* | 89.4* | 97* |
| **Total-P** | 3.5 (±4) | 4.5 (±6) | **-33**(±43) | 5.7(±14) | **-3.5**(±50) | **-30**(±57) |
| **Amm-N** | 1.2 (±7.5) | 1 (±8.8) | - | 10 (±8) | **-9.6**(±35) | 3.3(±9) |
| **Org-N** | 26.8 (±0.5) | 25 (±0.9) | - | 12 (±8.5) | 91(±59) | 13(±48) |
| **Total-N** | 28 (±7) | 28.7(±8.5) | - | **-15** (±53) | 68(±92) | **-7** (±28) |

*-geometric mean

### 4.2.2 Performance of the anaerobic ponds

In anaerobic ponds settling of SS and their stabilization supposed to occur. The settling in the present case was not effective. The anaerobic pond is supposed to remove BOD by about 54% efficiency during the coldest winter month and, depending on the wastewater temperature, up to and beyond 70% efficiency during other months. The BOD removal efficiencies observed were much lower than the expected efficiency at winter critical temperature, 54%. This could be because of heavy sludge accumulation (not desludged for the last 4 years) and the consequent reduced HRT and hydraulic short circuiting. This is also evident from the lower TSS removal efficiencies observed. Because of lower HRT relatively less reduction in total coliform count is expected. Monitoring results are also proved this point. Removal of helminth eggs and protozoan cysts might have been quite efficient in the anaerobic ponds but they were not monitored. Nitrogen removal in the anaerobic pond was not monitored. It is believed that organic nitrogen of the sewage was converted into ammonical nitrogen. Further, part of the nitrogen of the sewage was assimilated by the anaerobic sludge synthesized during the settled organic matter stabilization. Quantity removed through assimilation was estimated as 2% of the BOD removed in the anaerobic pond. Total phosphorus removal was observed as negative. Low pH conditions of the anaerobic pond must be resulting in solubilisation of the phosphorus present in the settled sludge and coming into the wastewater being treated.

### 4.2.3 Performance of the facultative ponds

Facultative ponds are supposed to reduce BOD by 70% during coldest winter month, and depending on the temperature, increase up to and beyond 90%. Presences of algal cells in the effluent from the facultative pond introduce error into the BOD removal efficiency estimations. Unless soluble BOD of the effluent is taken, because of algal cells, efficiencies will be much lower than the above mentioned. Because of the algal cells, contributing TSS in the treated effluent, commenting on the TSS removal is also not possible. Pathogen removal expected at the design temperature (17°C) is 92% and it is supposed to increase with increasing temperature and go beyond 98% as the temperature crosses 29°C. Designed total nitrogen and ammonical nitrogen removals, according to the equations 3.12 and 3.10 are 60% and 76% respectively.

Removal efficiencies observed, for the facultative pond, for organic matter (BOD and COD), TSS, pathogens and nutrients are shown in the tables 4.1 to 4.5. BOD and COD removals were around 28 and 15% respectively. These removals are much lower than the design removals. Algal cells in the effluent from the facultative pond could be responsible. Observed pathogen removals were unexpectedly lower than the design removal values (actual was 36% while expected removal is 92%). Observed ammonical nitrogen removal was around 10%, which is lower than the expected value of 24%. Observed total nitrogen removal was negative, while expected removal is 60%. Observed removals for all the forms of nitrogen were highly variable (standard deviations were much higher than means). This may be because of the grab sampling being used. Total phosphorus removal observed was also negative. In addition to grab sampling, effluents having higher concentration of algal cells (rich in nutrients) specially during 0:30 to 1:05 pm might have been responsible for the negative efficiencies.

### 4.2.4 Performance of the maturation ponds:

Maturation ponds are designed for pathogen removal. Significant nutrient removal also occurs in the maturation pond. In maturation ponds only small amount of BOD removal occurs. There are two streets of maturation ponds, each of three ponds in series, each treating half of the wastewater. Total area and volume of the three maturation ponds in each street are 6342 $m^2$ and 8245 $m^3$. HRT of the maturation ponds is 6.34 days. Design pathogen removal for the maturation ponds (at 17°C for 1.3 MLD flow per street) is 98.7%. Design nutrient removals are 60% for total nitrogen and 28% for ammonical nitrogen.

Actual removal efficiencies for pathogens, nitrogen, phosphorus, and BOD/COD observed are shown in tables 4.1 to 4.5 and figure 4.1. Observed pathogen removal efficiencies were unexpected lower (around 65%), specially for April month. Observed total nitrogen removal was 17% and observed ammonical nitrogen removal was negative (-9.6%). Even total phosphorus removal was negative (-3.5%). Grab sampling, and sample collection at around 1 PM might have been responsible for these observations. Observed BOD removal was 1% and COD removal was negative (-51%).

## 4.3 Design analysis of the STP

Average ambient air temperature for the coldest winter month of the year for Sultanpur Lodi is 17°C. Design capacity of the WSP is 2.6 MLD. Winter critical and summer critical temperatures of the wastewater in different ponds of the STP are not known. Similarly, characteristics of the wastewater assumed in the design of the WSP are also not available. Both ambient air temperature and wastewater temperature, and flow rates of the sewage were recorded at the time of sampling. See table 4.6 for details.

**Table-4.6: Temperature and flow rate of the sewage being treated**

| Parameter | February | March | April | May |
|---|---|---|---|---|
| Ambient air temperature (°C) | 21.2 | 32 | 36 | 38.8 |
| Water temperature (°C) | 20.1 | 23 | 26 | 29 |
| Flow rate (MLD)* | 1.87 | 3.5 | 3.4 | 3.3 |

* Daily sewage pumping and loading to the WSP is for 14 hours.

### 4.3.1 Design analysis of anaerobic pond

Volume of the anaerobic pond is $9314m^3$ and its design HRT is 3.58 days. The anaerobic pond has not been desludged for the last 4 years and is mostly filled with sludge. This might have significantly reduced effective volume of the anaerobic pond. Design volumetric loading rate according to the equation given in table 2.1 at 17°C was calculated as 240 $g/m^3.d$. According to this for 2.6 MLD capacity plant BOD of the sewage being loaded should be 430 mg/L. Treatment efficiency of the anaerobic pond according to the equation given in table 2.1 was estimated at 54%. On the basis of this BOD of the treated effluent from the anaerobic pond can be calculated as 198 mg/L. Design pathogen removal efficiency for the anaerobic pond according to the equation 3.3 was calculated as 85%. Total nitrogen removal in the anaerobic pond was estimated as 2% of the BOD removed on the basis of the net anaerobic synthesis and its nitrogen content.

Though the design equations were based on ambient air temperature, maximum volumetric loading allowed, and expected efficiency of organic matter removal and pathogen removal were calculated on the basis of the water temperature. As a consequence error in calculations was introduced. Further, the fact that the winter sewage temperature is usually higher than that of the ambient air, and that in summers the water temperature is lower than that of the ambient air was not taken into account in these calculations. Calculations relating to the design analysis of the anaerobic pond are given in table 4.7.

**Table -4.7: Design analysis calculations for the anaerobic ponds**

<table>
<tr><th colspan="2">Parameter</th><th>Feb</th><th>March</th><th>April</th><th>May</th></tr>
<tr><td colspan="2">Hydraulic retention time (days)</td><td>4.98</td><td>2.66</td><td>2.74</td><td>2.88</td></tr>
<tr><td rowspan="2">Volumetric organic loading ($g/m^3/d$)</td><td>Actual value</td><td>46</td><td>78</td><td>60</td><td>58</td></tr>
<tr><td>Maximum allowed at sewage temp.</td><td>301</td><td>330</td><td>350</td><td>350</td></tr>
<tr><td rowspan="2">Organic matter removal efficiency (%)</td><td>Actual efficiency</td><td>35</td><td>51</td><td>62</td><td>78</td></tr>
<tr><td>Expected efficiency at sewage temp.</td><td>60</td><td>66</td><td>70</td><td>70</td></tr>
<tr><td rowspan="2">Pathogen removal (%)</td><td>Actual % removal</td><td>89</td><td>83</td><td>26</td><td>68</td></tr>
<tr><td>Expected % removal at sewage temp.</td><td>90</td><td>94</td><td>96</td><td>98</td></tr>
<tr><td>Nutrient removal (mg/l)</td><td>Expected total-N of in the treated effluent</td><td>36</td><td>24</td><td>28</td><td>19</td></tr>
</table>

The anaerobic pond is hydraulically overloaded. Instead of 2.6 MLD the plant was treating as high as 3.5 MLD. But, volumetric loading is much lower than the designed loading rate of 240 g/m3.day. When looked from this angle, the anaerobic ponds have the capacity to treat as much as 5.58 MLD of sewage, provided it has around 200 mg/L of BOD. Except for the month of May (during all the other three months), actual organic matter removal efficiency was significantly lower than the expected removal efficiency. Similarly, pathogen removal was also on the lower side than the expected. This may be due to the reduced effective volume of the anaerobic pond from the accumulated sludge over the last four years. Further, the TSS removal efficiency was low (around 28% on the average) and this might also have affected the pathogen removal.

### 4.3.2 Design analysis of facultative Pond

There were two facultative ponds run parallel and total area of the facultative ponds is 14960 $m^2$ and its designed HRT is 8 days. Design surface loading rate, according to the equation 2.18 at 17°C, is 200 kg/ha.d. Designed organic matter removal efficiency for winter coldest month, according to the equations 3.6 to 3.9 is 83.5% (average). Expected pathogen removal efficiency according to the equation 3.6 is 92%. Expected total nitrogen and ammonical nitrogen removal efficiencies are 60% and 29% respectively.

Calculations relating to the design analysis of the facultative ponds are given in table 4.8. In the design analysis, though the design equations are actually based on average ambient air temperature of the coldest month of the year, actual temperature of the wastewater was used for estimating maximum surface loadings allowed, and expected efficiency of organic matter removal and pathogen removal. As a consequence error in calculations was introduced. Further, the fact that the winter sewage temperature is usually higher than that of the ambient air, and that in summers the water temperature is lower than that of the ambient air was not taken into account in these calculations.

Actual surface loading rate of the organic matter (BOD) was higher than the design surface loading rate during two of the four months of the study. However, this may not be of much concern, because of higher temperatures of the wastewater and the ambient air during these months. But, there are indications of overloading of the facultative ponds. For tackling the overloading problem either the anaerobic ponds may be managed for maximum efficiency (mainly through desludging) or the sewage loading rate to the STP may be reduced. Actual removal efficiencies were much lower than the expected removals (around 29% removal was observed against expected 95%). This may be due to high algal cell concentration in the treated effluent. TSS in the facultative pond outlet was around 75 mg/L.

**Table -4.8: Design analysis calculations for the facultative ponds**

| Parameter | | Feb | March | April | May |
|---|---|---|---|---|---|
| **Hydraulic retention time (days)** | | 7.9 | 4.2 | 4.4 | 4.5 |
| **Surface BOD loading rate (kg/ha/d)** | Actual value | 188 | 237 | 143 | 253 |
| | Maximum allowed at sewage temperature | 255 | 308 | 369 | 400 |
| **Organic matter removal efficiency (%)** | Actual efficiency | 40 | 42.5 | 2.4 | 30.3 |
| | Expected efficiency at sewage temperature | 95 | 95 | 94.6 | 95 |
| **Pathogen removal (%)** | Actual % removal | 18 | 94 | 23 | 44 |
| | Expected % removal | 95 | 97 | 98 | 99 |
| **Nutrient removal (%)** | Actual % removal of total nitrogen | 20 | **-97** | 11.4 | **-5** |
| | Expected efficiency of total-N removal in the treated effluent | 64 | 67 | 72 | 75 |

### 4.3.3 Design analysis of maturation Pond

Maturation ponds are designed for pathogen removal. Significant level of nutrient removal also occurs in the maturation ponds. BOD removals are usually not high and variable. There are two streets of maturation ponds, each with three ponds in series. Area of one street of maturation ponds is 6342 $m^2$ and volume is 8245 m3. HRT of the maturations ponds at designed flow of 2.6 MLD is 6.34 days. Designed pathogen removal efficiency for 2.6 MLD flow and 17°C is 98.7%. Expected nutrient removal at design flow and design temperature conditions is 60% for total nitrogen and 29% for ammonical nitrogen. Calculations relating to the design analysis of the maturation ponds are given in table 4.8. In the design analysis, though the design equations are actually based on average ambient air temperature of the coldest month of the year, actual temperature of the wastewater was used for estimating the pathogen removal and the nutrient removal. As a consequence error in calculations was introduced. Further, the fact that the winter sewage temperature is usually higher than that of the ambient air, and that in summers the water temperature is lower than that of the ambient air was not taken into account in these calculations.

**Table -4.9: Design analysis calculations for the maturation ponds**

| Parameter | | Feb | March | April | May |
|---|---|---|---|---|---|
| **Hydraulic retention time (days)** | | 9 | 4.7 | 4.85 | 5 |
| **Pathogen removal** | Actual % removal | 97 | 80 | 28 | 84 |
| | Expected % removal at sewage temp. | 99.60 | 99.90 | 99.97 | 99.99 |
| **Nutrient removal** | Actual % removal | **-31** | 50 | 2 | 47 |
| | Expected % removal at sewage temp. | 64 | 67.5 | 72 | 76 |

HRT of the maturation ponds (6.34 days) is on lower side than the typical value (10 days). Achieving the desired pathogen removal for making the treated effluent fit for unrestricted irrigation, in addition to maturation pond will also depend on the facultative and anaerobic ponds, which also contribute to pathogen removal. Actual pathogen removal (65%) was relatively lower than the expected removal (>99%) and in the month of April it was unexpectedly very low (28%). Temperatures were high and pathogen removals were expected to be higher. Still, for the reasons little known, the observed pathogen removal was low. Just as in the case of facultative ponds, grab sampling and sampling at noon hours apparently have made the nutrient removal calculations highly varying and erratic.

# CHAPTER – 5

# CONCLUSION

Performance evaluation and design analysis of a waste stabilization pond system based sewage treatment plant was carried out in order to comment on its design adequacy and to evaluate performance of the STP and its key constituent units. The STP was found not complying with the prescribed standards and the treated effluent, despite having maturation ponds, was not fit for unrestricted irrigation or for aqua-culturing. Average BOD and MPN of the treated effluent over the four months of monitoring were 68 mg/l and $2x10^6$ respectively. Algal cells in the treated effluent (TSS averaging 60 mg/L in the treated effluent) are apparently responsible for the non-compliance in case of BOD. Accumulation of sludge and dramatic reduction in the effective volume of the anaerobic pond might also have contributed to the non-compliance problems. The inefficiently functioning anaerobic pond must be overloading the facultative ponds and becoming a cause of concern. Hydraulic overloading of the STP (loading as high as 3.5 MLD while design capacity is 2.6 MLD) might also be causing problems.

**Annex. 1: Concentration for different parameter at different sampling point during month of February.**

**Sampling time: 12:35PM**

| Parameter | Inlet sewage (P1) | Anaerobic pond outlet (P2) | Facultative pond outlet (P3) | 1st Maturation pond outlet (P4) | 2nd Maturation pond outlet (P5) | Final Outlet (P6) |
|---|---|---|---|---|---|---|
| Flow rate ($m^3$/hr) | 134.2 | | | | | |
| TSS(mg/l) | 190 | 730 | 120 | | | 100 |
| pH | 7.56 | | | | | 7.54 |
| Temperature (°C) | 20.1 | | | | | 18.2 |
| BOD (mg/l) | 230 | 150 | 90 | 92.5 | 65 | 79 |
| COD (mg/l) | 700 | 300 | 500 | 400 | 700 | 200 |
| NH4-N(mg/l) | 36.68 | | 29.68 | | | 38.64 |
| Org-N (mg/l) | 1.06 | | 0.39 | | | 0.50 |
| Sulphate(mg/l) | 30 | | | | | |
| Chloride(mg/l) | 75.54 | | | | | 116.93 |
| Total-P (mg/l) | 0.079 | 0.088 | 0.079 | | | 0.063 |
| Total coliform (MPN/100) | $1.6\times10^8$ | $1.7\times10^7$ | $1.4\times10^7$ | | | $5\times10^5$ |
| Fecal coliform (MPN/100) | $1.3\times10^7$ | | | | | $1.7\times10^5$ |
| Nitrate (mg/l) | | | 0.184 | 0.047 | 1.068 | 0.592 |
| Nitrite (mg/l) | | | 0.086 | 0.063 | 0.025 | 0.054 |

**Annex. 2: Concentration for different parameter at different sampling points in the month of March.**

**Sampling time: 12:48PM**

| Parameter | Inlet sewage (P1) | Anaerobic pond outlet (P2) | Facultative pond outlet (P3) | 1st Maturation pond outlet (P4) | 2nd Maturation pond outlet (P5) | Final Outlet (P6) |
|---|---|---|---|---|---|---|
| **Flow rate ($m^3$/hr)** | 253 | | | | | |
| **TSS(mg/l)** | 130 | 107 | 63 | | | 40 |
| **pH** | 6.92 | | | | | 7.29 |
| **Temp. (°C)** | 22.9 | | | | | 25.2 |
| **BOD (mg/l)** | 205 | 100 | 57.5 | 27.5 | 26.5 | 106 |
| **COD (mg/l)** | 450 | 300 | 100 | 200 | 400 | 250 |
| **NH4-N (mg/l)** | 24.08 | | 31.08 | | | 23.52 |
| **Org-N (mg/l)** | 0.5 | | 17.25 | | | 0.1 |
| **Sulphate (mg/l)** | 139.94 | | | | | |
| **Chloride (mg/l)** | 101.84 | | | | | 87.22 |
| **Total-P (mg/l)** | 0.251 | 0.475 | 0.432 | | | 0.323 |
| **Total coliform (MPN/100)** | $1.3\times10^8$ | $2.2\times10^7$ | $1.3\times10^6$ | | | $2.6\times10^5$ |
| **Faecal coliform (MPN/100)** | $7\times10^7$ | | | | | $1.1\times10^5$ |
| **Nitrate (mg/l)** | | | 0.225 | 0.091 | 1.084 | 0.655 |
| **Nitrite (mg/l)** | | | 0.019 | 0.010 | 0.004 | 0.007 |

**Annex. 3: Concentration for different parameter at different sampling point during month of April.**

**Sampling time: 1:05PM**

| Parameter | Inlet sewage (P1) | Anaerobic pond outlet (P2) | Facultative pond outlet (P3) | 1st Maturation pond outlet (P4) | 2nd Maturation pond outlet (P5) | Final Outlet (P6) |
|---|---|---|---|---|---|---|
| Flow rate ($m^3$/hr) | 244 | | | | | |
| TSS(mg/l) | 220 | 160 | 30 | | | 10 |
| TDS(mg/l) | 660 | 470 | 570 | | | 650 |
| pH | 6.92 | | | | | 7.29 |
| Temperature (°C) | 26.0 | | | | | 31 |
| BOD (mg/l) | 164.5 | 62.5 | 61 | 26.5 | 6.5 | 51.8 |
| COD (mg/l) | 250 | 220 | 130 | 230 | 190 | 120 |
| NH4-N (mg/l) | 27.72 | | 14.84 | | | 23.24 |
| Org-N (mg/l) | 1.68 | | 11.2 | | | 2.24 |
| Sulphate (mg/l) | 362.2 | | | | | |
| Chloride (mg/l) | 113.13 | | | | | 103.73 |
| Total-P (mg/l) | 8.582 | 9.126 | 10.192 | | | 13.296 |
| Total coliform (MPN/100) | $3\times10^7$ | $2.2\times10^7$ | $1.7\times10^7$ | | | $2.2\times10^5$ |
| Faecal coliform (MPN/100) | $11\times10^6$ | | | | | $7\times10^5$ |
| $NO_2$ +$NO_3$ (mg/l) | | | | | | 5.267 |

**Annex. 4: Concentration for different parameter at different sampling point during month of May.**

**Sampling time: 1:05PM**

| Parameter | Inlet sewage (P1) | Anaerobic pond outlet (P2) | Facultative pond outlet (P3) | 1st Maturation pond outlet (P4) | 2nd Maturation pond outlet (P5) | Final Outlet (P6) |
|---|---|---|---|---|---|---|
| **Flow rate ($m^3$/hr)** | 244 | | | | | |
| **TSS(mg/l)** | 100 | 65 | 88 | | | 90 |
| **pH** | 6.92 | | | | | 7.29 |
| **Temp. (°C)** | 26.0 | | | | | 31 |
| **BOD (mg/l)** | 165 | 115 | 85 | 39 | 35 | 36 |
| **COD (mg/l)** | 250 | 220 | 130 | 230 | 190 | 120 |
| **NH4-N(mg/l)** | 18.76 | | 18.9 | | | 18.2 |
| **Org-N (mg/l)** | 1.68 | | 19.3 | | | 1.4 |
| **Sulphate (mg/l)** | 260 | | | | | |
| **Chloride (mg/l)** | 219.7 | | | | | 143.8 |
| **Total-P (mg/l)** | 5.17 | 9.08 | 6.98 | | | 4.63 |
| **Total coliform (MPN/100)** | $5\times10^8$ | $1.6\times10^8$ | $9\times10^7$ | | | $14\times10^6$ |
| **Faecal coliform (MPN/100)** | $1.3\times10^7$ | | | | | $1.4\times10^5$ |
| **$NO_2$ +$NO_3$ (mg/l)** | | | | | | 5.267 |

**Annexure-5: Metal concentration in the effluent**

| Month | Metals | Inlet sample | Outlet sample |
|---|---|---|---|
| Feb | **Fe** | 1.21 | 4.7 |
| | **Pb** | 0.16 | 0.25 |
| | **Cr** | **BDL** | **BDL** |
| | **Zn** | 0.55 | 0.067 |
| | **Ni** | 0.05 | 0.05 |
| March | **Fe** | 0.56 | **BDL** |
| | **Pb** | 0.03 | 0.00 |
| | **Cr** | **BDL** | **BDL** |
| | **Zn** | 0.03 | 0.07 |
| | **Ni** | 0.07 | 0.09 |
| April | **Fe** | 2.3 | 0.04 |
| | **Pb** | **BDL** | **BDL** |
| | **Cr** | 0.001 | BDL |
| | **Zn** | 0.12 | 0.03 |
| | **Ni** | 0.03 | 0.06 |

**BDL -** Concentration beyond detectable limit.

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