Performance Evaluation of UASB + Polishing pond Based Sewage Treatment Plant

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by

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CANDIDATE'S DECLARATION

I hereby declare that the work presented in the dissertation entitled **"Performance evaluation of Sewage treatment plant"** in partial fulfillment of the requirement for the award of degree of **Master of Technology** in **Environment Science and 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 and Mr. K. S. Babu, Department of Biotechnology and Environment Sciences, Thapar University, patiala. 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.

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CERTIFICATE

This is to certify that the thesis report entitled **"Performance evaluation of Sewage treatment plant "** submitted by Pinki Sharma 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

Upflow Anaerobic Sludge Blanket Reactor is an extremely fast and effective method for removal of bulk organic compounds from wastewater and has been successfully used to treat variety of industrial as well as domestic wastewaters. Anaerobic reactors hardly produce effluents that comply with usual discharge standards established by environmental agencies. Therefore, the effluents from anaerobic reactors usually require a post-treatment step as a means to adapt the treated effluent to the requirements of the environmental legislation and protect the receiving water bodies. The main role of the post-treatment is to complete the removal of organic matter, as well as to remove constituents little affected by the anaerobic treatment, such as nutrients and pathogenic organisms. The UASB + polishing pond configuration has been studied in this dissertation work. This alternative looks more attractive when the effluent from the pond can be used for agricultural purposes, since the polishing ponds aim mainly at the removal of pathogenic organisms.

UASBs are mostly designed using empirical formulas derived either from past pilot scale studies 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], UASB Reactor outlet [P-2], Polishing Pond outlet [P-3] to comment on its design adequacy and to evaluate performance of the STP and its key constituent units. The treated effluent was found almost complying with the prescribed standards. BOD, COD, TSS and MPN of the treated effluent were 33±11 mg/L, 137±43 mg/L, 75±69mg/L and 3.3×10^6 respectively. Excepting for MPN, if not for the role played by the algal cells in the treated effluent, the treatment process is satisfactory indicating that plant is working properly.

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CHAPTER-1

INTRODUCTION

1.1 Background

The Satluj River is heavily polluted by the wastewater discharged specially by Phillaur, Phagwara, Ludhiana, Jalandhar, Kapurthala and Sultanpur 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 University 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 Upflow Anaerobic Sludge Blanket reactor + Polishing Pond), installed at Bhattia, Ludhiana.

1.2 Objectives

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

Upflow Anaerobic Sludge Blanket reactor (UASB) has proven to be effective alternative for treating wastewater. In contrast to aerobic process, anaerobic treatment process has many advantages. The organic matter (COD) presents in the wastewater in the absence of oxygen is mainly converted in to biogas, which is a valuable product. Very little portion is converted in to the sludge. No major inputs are required to operate the system. They will hopefully lead to more ecologically-sustainable wastewater treatment in the future. Upflow Anaerobic Sludge Blanket (UASB) reactor has been widely used to treat variety of industrial and domestic wastewaters all over the world. The wastewater coming of UASB usually requires post treatment in order to meet the prescribed standards. In the light of this, UASB + Polishing pond 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 adequacy of design, capacity and performance of the STP in question. Evaluation of reactor 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. Since this work is related to and part of the project of monitoring of STPs running under Satluj River Action Plan, it will help the ministry in getting information regarding the status of STP performance and improvement suggestions. 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 Upflow Anaerobic Sludge Blanket Reactor + Polishing Pond based Sewage Treatment Plant includes altogether five chapters and a reference section.

Chapter - 1 is **“Introduction”.** It provides brief background information, 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 critical review on UASB reactor. Their treatment process, design constituents and their post treatment options are covered in this chapter.

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.

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.

1.5 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 Upflow Anaerobic Sludge Blanket (UASB) reactor for Sewage treatment

Successful use of anaerobic reactors (especially up-flow anaerobic sludge blanket reactors, (UASB) for the treatment of raw domestic sewage in tropical and sub-tropical regions opened the opportunity to substitute the aerobic processes with the anaerobic technology for the removal of organic matter from the influent wastewater. Despite the success, effluent from the anaerobic reactors, treating domestic sewage, requires post-treatment in order to achieve the discharge standards prevailing in most countries. The use of UASB technology for sewage treatment has been explored as a feasible option in many developing countries like Colombia, Indonesia, Brazil, China, and India (Alaerts *et al*., 1993). Microbial granulation involves different trophic bacterial groups, and physico-chemical and microbiological interactions (Hulshoff Pol, 1989). Capital costs for the UASB process are lower than those for other anaerobic processes since the separation of gas, liquid and solids takes place entirely in the reactor and no support medium for bacterial attachment is required (Maat and Habets, 1987). An USAB reactor has four major components: (1) sludge bed; (2) sludge blanket; (3) gas-solids separator (GSS); and (4) settling zone. Influent wastewater enters the reactor at the bottom and is biologically degraded in both the sludge bed and the sludge blanket. Gas is separated from the liquid by the GSS device. A quiescent zone is created in the settling zone. Most of the sludge particles that have entered the settling zone can settle back to the reactor while the rest are washed out via the effluent.

TÖNÜK (2004) studied treatment of domestic wastewater in the anaerobic upflow sludge blanket reactors (UASBR), and determined the hydraulic and kinetic factors affecting the performance of the system in the process of develop a compact treatment system for the regions of Turkey having a mild and warm climate. Hydraulically, the

UASBR used in this study can be considered as a completely mixed compartment and a plug flow compartment in series. Volume of the plug flow compartment constitutes about 10 % of the whole system. Characteristics of the feed were different in each stage (like flow, hydraulic retention time, organic loading rate).Organic loading rates were varied in the range of 0.2–1 kg COD/m^3.d. Domestic wastewaters can be anaerobically treated in mesophilic UASBR's with 70% COD removal without any chemical treatment. The effluent suspended solid concentrations were very low and rarely exceeded 50 mg/l. The results obtained perfectly fit the second order multiple substrate kinetics model at steady state operating conditions.

Mirsepasi et.al (2006) carried out the performance evaluation of full scale UASB reactors in treating stillage wastewater. In this study two full-scale UASB reactors (420 m^3) were investigated. Conventional parameters such as pH, temperature and efficiency of COD, BOD, TOC (total organic carbon) removal and also the upflow velocity, organic loading rate (OLR) and hydraulic retention time were investigated. It was concluded that COD removal efficiency can be enhanced by enhancing of organic loading rates (OLRs) and upflow velocity, by decreasing hydraulic retention time (HRT) and by operating the reactors with new sludge.

Elmitwalli and Otterpohl (2007) studied feasibility of grey water treatment in an upflow anaerobic sludge blanket (UASB) reactor operated at different hydraulic retention time (16, 10 and 6 h) and controlled temperature of 30 °C. The results showed a total COD removal of 52–64% at HRT between 6 and 16 hr. The UASB reactor also removed 22–30% and 15–21% of total nitrogen and total phosphorous, respectively.

Ruiz (1998). Domestic wastewater from the city of a Coruna (NW Spain) was treated in laboratory scale UASB digester at 20°C at HRT > 24 hr. The COD & SS removal efficiencies remained practically constant and higher than 85%. By reducing the HRT from 25 to 5 hr COD removal efficiency decreased 53% and SS removal to 63%. The methane recovered in biogas amounted to 25-30% of influent COD. Increase of efficiency by about 5% was reported when the UASB was used with a completely mixed sludge digester system for external digestion & stabilization of accumulated solids into the UASB.

Granulation of sludge is an indication of successful operation of the UASB system. Efficient performance of the reactor can be obtained with the formation of highly settleable sludge. Three stages in the granulation of seed sludge are identified (Hulshoff Pol *et al.,* 1983 and W. Wu *et al., 1987).* In the first stage, organic loading applied was lower than 2 kg COD/m^3/d and the sludge bed expands significantly as a result of gas production. During the second stage, organic loading rate was increased to about 2-5 kg COD/m^3/d, an increase in gas production rate results in the wash-out of fluffy sludge. Growth is predominant in the form of dense heavy sludge particles. In the third stage, organic loading is even higher and can be set to its maximum design value.

Bhunia and Ghangrekar in 2008 carried out a study to correlate biogas-induced mixing and granulation in upflow anaerobic sludge blanket (UASB) reactors, treating low-strength as well as high-strength biodegradable wastewaters. A dimensionless granulation index (GI) was framed taking into account the mixing in sludge bed due to produced biogas. Analysis of full-scale, pilot-scale and lab-scale UASB reactors treating actual wastewaters reveals the significance of biogas-induced mixing, represented by GI, on the granulation of biomass in the reactors. For obtaining proper granulation resulting in higher chemical oxygen demand (COD) removal efficiency, it is recommended to maintain GI values in the range of 15,000–57,000.

Yu et.al (2000) studied the effect of Fe^{2+} on sludge granulation in Upflow Anaerobic Sludge Blanket Reactors. In the experiment 6 identical reactors operated in parallel, 1 being control (with addition of $FeCl_2.H_2O$) were dozed with varying concentration of Fe^{2+} (150 to 800 mg/l). Introduction of Fe^{2+}at concentrations 300 and 450 mg/L enhanced the granulation process in UASB reactors, while a low dosage of Fe^{2+} at 150 mg/L had little effect on the sludge granulation. The Fe^{2+} concentration in granules was proportional to the influent Fe^{2+} concentration, FeS and the compounds formed by iron and exo-polysaccharide polymers were the main precipitates in the granules. Specific activity of the granules decreased with increasing Fe^{2+} concentration in the feed. Presence of a large amount of minerals deposited within the granules, significant decrease of water content in granules, and the possible toxicity of high-concentration Fe^{2+} accumulated inside granules might have been responsible.

Manoj et.al., 2006 reported that the process of granulation is affected by environmental and operational conditions in the reactor. According to the authors granule composition strongly depends on the operational temperature. Sudden temperature change could results in granule disintegration. Divalent ions such as Ca^{+} & Fe^{2+} enhance the granulation by ionic bridging. Preferred conditions for the granulation are high partial pressure of H_2 and neutral pH.

Kripa Shankar et.al (2006) studied the effects of Sludge Blanket Height, Flow Pattern and temperature in 2 upflow anaerobic sludge blanket (UASB) reactors (operated for approximately 900 days) treating municipal wastewater under low temperature conditions. A modified solid distribution model has been formulated by incorporating the variation of biogas production rate with change in temperature. It was confirmed by experimental observations of solid profile along the height of the reactor that the model simulated the solid distribution well. Mathematical analysis of tracer curves indicated the presence of a mixed type of flow pattern in the sludge-bed zone of the reactor. It was found that the dead-zone and bypass flow fraction were impacted by the change in operating temperatures.

Maximum biological loading rate that can be allowed in the reactor depends on the methanogenic activity of the sludge. For domestic sewage, the methanogenic activity usually ranges from 0.3 to 0.4 kg COD/VS.d. (Chernicharo, 2007). Methanogenic bacteria are highly sensitive to temperature and the temperature at which the process is operated is a very crucial factor (Gujer and Zehnders, 1983). Operations under both mesophilic and thermophilic conditions have been investigated (Hulshoff Pol et al, .1983; Wiegant and Man, 1986; Hulshoff Pol and Lettinga 1986; Wiegant et.al, 1985; Brummeler and Hulshoff Pol, 1985; Stronach et al.,1987; Hulshoff Pol et al.,1982). Granular sludge is formed under mesophilic conditions ,30-35°C (Lettinga and Vinken, 1980; Hulshoff Pol et al.,1983; Zeeum and Lettinga, 1983; Dold et al., 1987; Wu et al.,1987*)* and also under thermophilic conditions, 55°C (Wiegant and Man1986; Wiegant and Lettinga,1985). Although granulation is faster under thermophilic conditions, the mechanism underlying the granulation process under both conditions is similar. At low temperatures, growth of active biomass may be so slow that it is very difficult and time-consuming to accomplish the granulation process. It has been reported that specific activity of sludge at 35°C is more than twice to that at 20°C, and about six times to that at 10°C. For this reason, process start-up should be done in mesophilic (or thermophilic) conditions even for reactors designed to be operated at low temperatures (Brunetti et al.,1983). In all circumstances, a sharp temperature change is detrimental to microorganisms and should be avoided (Souza, 1986; Lettinga et.al, 1984).

Singh and Viraraghvan (2003) studied feasibility of treating municipal waste water by UASB system under low temperature conditions and the effect of HRT and temperature on COD, BOD and SS removal. It was found that up to temp. 11°C and HRT 6hr, reactor performed well in terms of removal efficiency. A decrease in removal efficiency of COD & BOD was observed when HRT was reduced to 4hr and 3hr, which was severe at low temperature. Temperature did not affect significantly the SS removal efficiency. The study concluded that a UASB reactor could be start-up successfully for application at low temperature for municipal waste water in cold region (average temperature 15-20°C).

UASB can be optimally designed for 6-10 hr HRT for low temperature operation up to 11°C with a reasonable COD, BOD & SS removal efficiency (70-90%).

Bhunia and Ghangrekar in 2007 studied three kinetic models for the performance appraisal of upflow anaerobic sludge blanket (UASB) reactors treating wastewater with COD in the range of 300–4000 mg/L, namely, Monod, Grau second-order, and Haldane model. Both linear and nonlinear regressions were performed to examine the best-fit among the kinetic models. Grau second-order multi-component substrate removal model was fitting well for estimating kinetic coefficients in UASB reactors.

Pontes and Chernicharo et.al (2003) studied the influence of excess sludge produced in Trickling filter (TF) on the performance of a UASB reactor used for combined treatment of domestic wastewater and trickling filter sludge. Experiment was conducted in 2 phases. In the 1st phase the UASB/TF system was fed with domestic sewage directly. In 2nd phase, beside domestic sewage aerobic sludge from TF was also fed to the UASB reactor. It was found that the return of excess aerobic sludge produced in TF, has not affected the performance of the UASB rector and final quality of effluent was even better during phase 2nd, in term of COD, BOD & TSS.

Halalsheh et.al (2004) studied the two stages versus single stage UASB reactor treating domestic sewage operated at ambient temperature and concluded that a single stage UASB reactor operated at relatively longer HRT perform well above the two stage reactor.

Tawfik et.al (2003) studied the treatment of domestic sewage in a combined Upflow Anaerobic Sludge Blanket Reactor and Rotating Biological Contactor (UASB/RBC) system for irrigation purposes and concluded that an efficient pre-treatment of sewage implies a substantial reduction of organic loading rate (OLR) applied to the RBC and consequently improved the residual of total COD, ammonia and *E. coli* in the final effluent. The results supported the use of combined system UASB/RBC for treatment of domestic wastewater for reuse in irrigation.

Start-up process and operation of UASB reactor are the two main factors that affect granulation of the anaerobic sludge. **Álvarez et al (2005)** found that start-up of UASB digesters is feasible with or without inoculums. Three experiments were carried out. In the 1st experiment start-up of self-inoculated UASB, working with dilute raw municipal wastewater (330 mg TCOD/L) at 12 h HRT and 20–18 °C occurred after 120 day. In the 2nd experiment digested primary sludge was used as inoculums working with extremely dilute wastewater (136 mg TCOD/L) at temperatures below 14°C, the start-up occurred in a period of 75 day. In the 3rd experiment treating medium strength raw municipal wastewater at about 19 °C and 15 hr HRT, hydraulically adapted sludge was used. This enabled rapid sludge bed development and high TSS removal in only three weeks.

UASB cannot fully replace aerobic treatment. A combined anaerobic/aerobic system may provide a more reliable and cost effective way for the wastewater treatment. **(Kwan Chowlin and Zhenxiang Yang, 1990).** Recent investigations have demonstrated that it is feasible to utilize a combined technology, consisting of an upflow anaerobic sludge bed (UASB) reactor for anaerobic pre-treatment, followed by aerobic post treatment, to efficiently treat municipal wastewater (Enrique,2007). In such systems the excess sludge produced in the aerobic stage is recycled to the anaerobic unit for stabilization. This configuration is an attractive alternative for secondary wastewater treatment because the costs associated with sludge digestion are eliminated.

Enrique et al (2007) favoured the use of a combined upflow anaerobic sludge bed (UASB)/ aerobic solids contact system (ASC) for the treatment of municipal wastewater and demonstrated the technical feasibility of using the UASB process as both a pre-treatment unit and a waste activated sludge digestion system. Although the UASB reactor had low TSS and TCOD removal efficiencies, the overall UASB/ASC system was capable of meeting secondary-effluent water quality requirements with an overall HRT of at least 5 hr. UASB produces methane gas at an average rate of 6.47 ml per litre of the sewage treated. UASB/ASC process was not only effective for providing secondary wastewater treatment, but it also minimized the surplus sludge production and produced a well-stabilized sludge.

Enrique et al (2008) compared the two anaerobic pre-treatment technologies, namely, the anaerobic fluidized bed reactor (AFBR) and the UASB, and demonstrated that both have similar performances with regard to chemical oxygen demand (COD) removal, suspended solids removal, and gas generation. Much more efficient sludge stabilization was achieved in the UASB. In addition, the UASB required significantly lower energy for effluent recirculation than the AFBR. Thus it concluded that UASB would be more economical to operate.

Nidal Mahmoud (2008) investigated treatment of high strength sewage in a one-stage upflow anaerobic sludge blanket (UASB) reactor and a UASB digester system. Both perform almost similar under high temperature conditions, However at lower temperature performance of the UASB-digester system was better. It is evident that the problem of solids accumulation during the cold period of the year was handled successfully by incorporating a sludge digester.

Tawfik et al (2005) studied the performance of up-flow anaerobic sludge blanket (UASB) in combination with down-flow hanging sponge (DHS) system for sewage treatment at an average wastewater temperature of 15°C for 6 months. The results showed that a combined system operated at a total HRT of 10.7 hr and total solid retention time (SRT) of 88 days represented a cost effective sewage treatment process. It proved the most efficient combined process and it not only removed COD total (90%), BOD5 total (98%), TSS (94%), ammonia (86%) and faecal coliform (99.92%) but also reduced the excess sludge production.

2.2 Facultative pond

Anaerobic reactors hardly produce effluents that comply with usual discharge standards established by environmental agencies. Taking into consideration the intrinsic limitations associated with the anaerobic systems and the need to develop technologies that are more appropriate to the reality of developing countries, it is important to include a post-treatment stage for the effluents generated in anaerobic reactors. Therefore, the effluents from anaerobic reactors usually require a post-treatment step as a means to adapt the treated effluent to the requirements of the environmental legislation and protect the receiving water bodies.

The main role of the post-treatment is to complete the removal of organic matter, as well as to remove constituents little affected by the anaerobic treatment, such as nutrients (N and P) and pathogenic organisms. The UASB reactor + polishing pond configuration is a very interesting alternative from the technical–economical–environmental point of view. This alternative is even more attractive when the effluent from the pond can be used for agricultural purposes, since the polishing ponds aim mainly at the removal of pathogenic organisms.

Facultative ponds are largely used for post-treatment of effluents from anaerobic ponds. When an efficient anaerobic pre-treatment is applied prior to the sewage discharge into a pond, the concentrations of organic matter and suspended solids are largely reduced, and consequently it will be required only a complementary removal of these two constituents, needing much lower hydraulic retention times. In these conditions, the limiting factor that determines the minimum retention time, (therefore, the volume and the area of a pond system), will usually be the removal of pathogenic organisms, and not the stabilisation of the organic matter. For this reason, the nomenclature polishing pond has been adopted to name those ponds intended for the post-treatment of effluents from efficient anaerobic systems, thus distinguishing them from the stabilisation pond, which treats raw sewage (Cavalcanti et al. 2001a, b).

Wastewater treatment plants using UASB reactors followed by polishing ponds also have a very simplified flow sheet (Figure: 2.1). Besides the preliminary treatment units (screen and grit chamber), the flow sheet comprises the anaerobic treatment unit, the polishing pond (either a single baffled pond or ponds in series), and the dewatering unit for the sludge produced in the UASB reactor which is already thickened and stabilised.

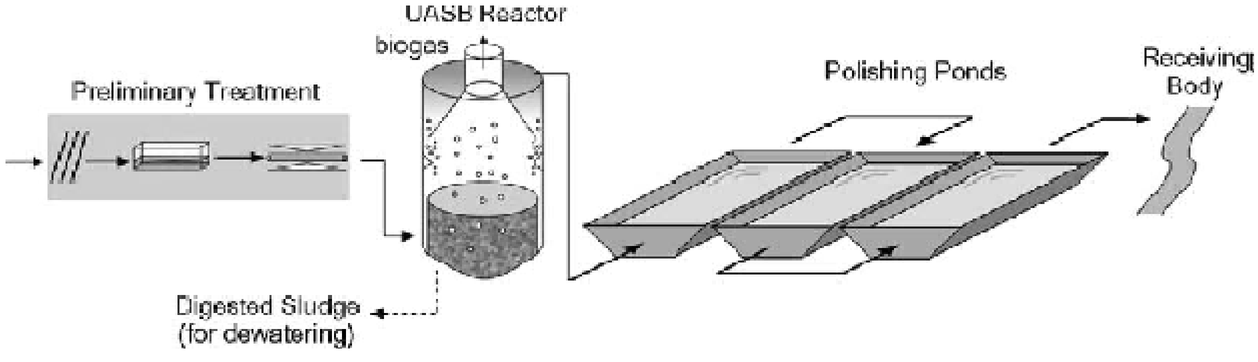

Figure: 2.1 typical configuration of a treatment plant with UASB reactor and polishing ponds (von Sperling & Chernicharo 2005).

Von Sperling and Mascarenhas (2004) have conducted a study and have shown that a domestic sewage treatment system comprised of a UASB reactor followed by four very shallow (0.40 m-depth) polishing ponds in series, operated with very low detention times (1.4–2.5 days in each pond), was able to achieve excellent results in terms of BOD and E. coli removal, and also good results in terms of ammonia removal. The average concentrations observed in the final effluent were 44 mg BOD/L), $3.8\text{x}10^2$ MPN/100ml) and 7.3 mg NH_4-N/L). Polishing pond systems are capable to produce effluents with helminth eggs concentrations predominantly equal to zero, and satisfying the WHO guidelines for unrestricted and restricted irrigation (von Sperling et al. 2002, 2004).

2.3 Design considerations of the UASB reactor

One of the most important aspects of the UASB reactors is its ability to develop and maintain high-activity sludge of excellent settling characteristics. For this purpose, several measures should be taken in relation to the design and operation of the system. Main design criteria for UASB reactors are presented as:

2.3.1 Volumetric hydraulic load and Hydraulic detention time

The volumetric hydraulic load is the amount (volume) of wastewater loaded per unit volume of the reactor per unit time. The hydraulic detention time is reciprocal of the volumetric hydraulic load.

$$= - \qquad\qquad --- (2.1)$$

Where:

= volumetric hydraulic load ($m^3/m^3.d$)
= flow rate (m^3/d)
= total volume of the reactor (m^3)

$$t = \frac{V}{Q} \qquad\qquad --- (2.2)$$

Where:

t = hydraulic detention time (d)

Experimental studies demonstrated that the volumetric hydraulic load should not exceed the value of 5.0m3/m3.d (Chernicharo, 2007), which is equal to a minimum hydraulic detention time of 4.8 hr. Recommended hydraulic detention times for UASB reactors treating domestic sewage are shown in table-3.1 (Chernicharo, 2007). Hydraulic detention time parameter is directly related to the upflow velocity in the reactor also depends on the size of the reactor. For an average temperature close to 20°C, HRT can vary from 6 to 16 hrs, depending on the type of the wastewater (Chernicharo, 2007). The detention time for maximum flow rate, should not be shorter than 4 hrs, and the maximum flow peaks should not extend beyond 4 to 6 hrs.

By knowing the influent flow rate and assuming HRT, volume of the reactor can be calculated as:

$$V = Q \times t \qquad --- (2.3)$$

Table-2.1: Recommended HRTs for UASB reactors treating domestic sewage

Sewage temperature (°C)	Hydraulic Retention Time	
16 to 19	**daily average**	**minimum (during 4-6 hr)**
	>10 to 14	> 7 to 9
20 to 26	> 6 to 9	> 4 to 6
> 26	> 6	> 4

Source: Chernicharo, 2007 (adapted from Lettinga and Hulshoff Pol, 1991)

2.3.2 Organic loading rate (Lv)

Volumetric organic loading rate is defined as the amount of organic matter loaded per unit volume of the reactor per unit time.

$$Lv = \frac{Q * So}{V} \qquad --- (2.4)$$

Where:

Lv = volumetric organic loading rate (kgCOD/m^3.d)
Q = flow rate (m^3/d)
So = influent substrate concentration (kgCOD/ m^3)
V = total volume of the reactor (m^3)

Assuming certain design volumetric organic load (L_v), volume of the reactor can be calculated as:

$$V = \frac{Q \times So}{Lv} \qquad --- (2.5)$$

In the case of industrial effluents with a high concentration of organic matter, literature reports extremely high organic loads successfully applied to the pilot facilities (45 kg COD/m^3.d), although the organic loads adopted in the design of full scale plant have been lower than 15 kg COD/m^3.d (Chernicharo, 2007). For high strength, volumetric organic load actually defines the reactor volume. Recommended volumetric organic loading for UASB reactors are shown in table-3.2 (Makarand and Ghangrekar, 2005). The domestic sewage with relatively low concentration of organic matter (1000 mg COD/L), the volumetric organic load to be applied is much lower (2.5 to 3.5 kg COD/m^3.d). Higher load can results in excessive hydraulic loads and, consequently, excessive upflow velocities.

Table 2.2: Recommended volumetric organic loading range for UASB reactors.

Category of waste water	COD (mg/L)	OLR, Kg COD/m^3.d	SLR, Kg COD/kg VSS. d	HRT, hours	Liquid upflow velocity, m/h	Expected efficiency, %
Low strength	Upto 750	1.0-3.0	0.1-0.3	6-18	0.2-0.7	70-75
Medium strength	750–3000	2.0-5.0	0.2-0.5	6-24	0.25-0.7	80-90
High strength	3000–10,000	5.0-10.0	0.2-0.6	6-24	0.15-0.7	75-85
Very high strength	> 10,000	5.0-15.0	0.2-1.0	>24	--	75-80

Source:http://www.waterandwastewater.com/www_services/ask_tom_archive/toc.htm

2.3.3 Biological loading rate (sludge loading rate)

Biological loading rate refers to the amount (mass) of organic matter loaded daily to the reactor, per unit of biomass present.

$$\mathrm{Ls} = \frac{\mathrm{Q} \times \mathrm{So}}{\mathrm{M}} \qquad --- (2.6)$$

Where:

Ls = biological or sludge lodging rate (kg COD/kg VS. d)
Q = average influent flowrate (m^3/d)
So = influent substrate concentration (kg COD/m^3)
M = mass of microbes present in the reactor (kg VS/m^3)

Literature recommends that the initial biological loading rate during start-up should be in the range of 0.05to0.15 kg COD/kg VS. d (Chernicharo, 2007), depending on the type of the effluent being treated. These loads should be gradually increased, according to the efficiency of the system. The maximum allowed biological loading rate depends on the methanogenic activity of the sludge. For domestic sewage, the methanogenic activity usually ranges from 0.3 to 0.4 kg COD/VS. d. (Chernicharo, 2007)

2.3.4 Upflow velocity and reactor height

The upflow velocity of the liquid is calculated as:

$$\mathrm{v} = \frac{\mathrm{Q}}{\mathrm{A}} \qquad --- (2.7)$$

Where:

v = upflow velocity (m/hr)
Q = flow (m^3/hr)
A = cross sectional area of the reactor

Alternatively, the upflow velocity can also be calculated by:

$$\mathrm{v} = \frac{\mathrm{Q} \times \mathrm{H}}{\mathrm{V}} = \frac{\mathrm{H}}{\mathrm{t}} \qquad --- (2.8)$$

Where H is the height of reactor (m)

Maximum upflow velocity allowed in the reactor depends on the type of the sludge present and the load applied. Table-3.3 gives recommended upflow velocities for UASB reactors treating domestic sewage (Chernicharo, 2007). For a reactor operating with flocculent sludge and the organic loading rates ranging from 5 to 6 kg COD/m^3.d, the average upflow velocities should be 0.5 to 0.7 m/hr, with temporary peaks up to 1.5 to 2.0 m/hr being tolerated for 2 to 4 hr duration. For reactors operating with granular sludge, the upflow velocities can be significantly higher (e.g upto 10 m/hr).

Table-2.3: Recommended upflow velocities for design of UASB reactors treating domestic sewages.

Influent flowrate	**Upflow velocity (m/hr)**
Average flow	0.5 to 0.7
Maximum flow	<0.9to 1.1
Temporary peak flow	<1.5

Source: Chernicharo, 2007 (adapted from Lettinga and Hulshoff Pol,1995)

2.3.5 UASB reactor efficiencies

Efficiencies of the UASB reactors are estimated mainly by means of empirical relations. IWA task group developed models for efficiency estimations using operational results of 16 reactors treating domestic sewage under tropical conditions (Batstone et al.,2002). The COD and BOD removal efficiency are substantially affected by the hydraulic detention time of the system, ranging from 40 to 70% for COD removal and 45-90% for BOD removal.

$$E = 100 \times (1 - 0.68 \times t^{\cdot}) \qquad - - - (2.9)$$

Where:

E = efficiency of UASB reactor in term of COD removal (%)
t = hydraulic detention time (hr.)
0.68 = empirical constant
0.35 = empirical constant

$$E = 100 \times (1 - 0.70 \times t^{\cdot}) \qquad - - - (2.10)$$

Where:

E = efficiency of UASB reactor in term of BOD removal (%)
t = hydraulic detention time (hr.)
0.70 = empirical constant
0.50 = empirical constant

From the efficiency expected for the system, the COD and BOD concentration in the final effluent can be estimated as below:

$$C = So - \frac{\cdot}{} \qquad - - - (2.11)$$

Where:

C = effluents total COD and BOD concentration (mg/l)
So = influent total COD and BOD concentration (mg/l)
E = COD and BOD removal efficiency (%)

2.3.6 Biogas production

The biogas production can be evaluated from the influent COD load to the reactor by knowing the portion of COD converted into methane. The portion of COD converted into methane gas can be determined by:

$$\mathrm{COD} = \mathrm{Q} \times (\mathrm{So} - \mathrm{S}) - \mathrm{Y} \quad \times \mathrm{Q} \times \mathrm{So} \qquad - - - (2.12)$$

Where:

COD = COD load converted in to methane (Kg COD_{CH4}/d)
Q = average influent flow (m^3/d)
So = influent COD concentration (kgCOD/m^3)
S = effluent COD concentration (kgCOD/m^3)
Y = coefficient of solid production in the system, in term of COD (0.11 to 0.23 kg COD sludge/kg COD applied)

The methane mass (Kg COD_{CH4}/d) can be converted into volumetric production (m^3CH_4/d) by using the following equation:

$$\mathrm{Q} = \frac{\mathrm{COD}}{\mathrm{K(t)}} \qquad - - - (2.13)$$

Where:

Q = volumetric methane production (m^3/d)
$\mathrm{K(t)}$ = correction factor for the operational temperature of the reactor (kg COD/m^3)

$$\mathrm{K(t)} = \frac{\mathrm{P} \times \mathrm{K}}{\mathrm{R} * (273 + \mathrm{T})} \qquad - - - (2.14)$$

Where:

P = atmospheric pressure (1atm)
K = COD corresponding to 1mole of CH_4 (64gCOD/mol)
R = gas constant (0.08206 atm.L/mole.K)
T = operational temperature of the reactor (°C)

The total biogas production can be estimated from the expected methane content. For the domestic sewage, the CH_4fraction in the biogas is usually in the range of 70-80%.

2.3.7 Sludge production:

Estimation of the mass of sludge produced in UASB reactors can be done by:

$$P = Y * COD \qquad --- (2.15)$$

Where:

P = production of solids in the system (kg TSS/d)

Y = yield or solids production coefficient (kg TSS/kg COD_{app})

COD = COD load applied to the system (kg COD/d)

Values of Y reported for the anaerobic treatment of domestic sewage are in order of 0.10to0.20 kg TSS/kg COD_{app}.

Volumetric sludge production can be estimated by:

$$V = \frac{P}{\gamma * (C \ /100)} \qquad --- (2.16)$$

Where:

V = volumetric sludge production (m^3/d)

γ = sludge density (usually in order of 1020 to 1040 kg/m^3)

C = solid concentration in the sludge (%)

2.4 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.

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 λ_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;

$$= \frac{\quad}{\quad} \qquad ---(2.17)$$

Where

L_i = influent BOD (kg BOD_5/d)

Q = flow rate (m^3/d)

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

Design value of λ_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:

$$\lambda s = 350 \times (1.107 - 0.002 \times T)^{(T-25)} \qquad ---(2.18)$$

Where T is 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

$$= \frac{\quad}{\quad} \qquad ---(2.19)$$

Where;

H = pond depth (usually 1.5m)

Q = average flow, m3.d

Coliform removal

It can be estimated using following equation:

$$\frac{\;}{\;} = \frac{\;}{\;} \qquad ---(2.20)$$

Where;

Subscript f refers to the facultative pond

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

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

= hydraulic retention time

= coliform die–off coefficient

$$= \quad ^{(\quad)} \qquad ---(2.21)$$

Where;

= coliform die–off coefficient at 20°C, taken as 2.6 (Marais, 1974)

T = temperature (°C)

= temperature coefficient, taken as 1.19 (Marais, 1974)

Ammonical nitrogen removal

Equation used when temperature is below 20°C:

$$= \frac{\;}{1 + [(\ /\ ).(0.0038 + 0.000134.\ ).^{(\cdot \quad \cdot \ \cdot).(\quad \cdot)}]} \qquad ---(2.22)$$

Equation used when temperature is more than 20°C:

$$= \frac{\;}{1 + \ 5.035 \times 10 \quad .(\ /\ ).^{\cdot \quad \times(\quad \cdot)}} \qquad ---(2.23)$$

Where;

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

= 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

Equation used in case of facultative and maturation ponds (Reed, 1995):

$$= \quad \exp\{-[0.0064(1.039) \quad][\ + 60.6(\quad - 6.6)]\} \quad - - - (2.24)$$

Where;

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

= 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)

CHAPTER-3

METHODOLOGY

3.1 Introduction

This chapter presents the methodology followed in the present study of performance evaluation and design analysis of an UASB and Polishing pond based sewage treatment plant. For achieving the objectives of the study, work was planned on the following work elements:

- Getting background information of the STPs
- Monitoring
- Performance evaluation
- Design analysis

3.2 Survey of the STP for getting background information

For completing this work element the STP in question was visited, and peoples working at the site were discussed with to know about the treatment scheme, about dimensions and capacities of different units and facilities of the STP and to understand the treatment process. On the basis of the survey visit a schematic process flow diagram was constructed (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. Monitoring involves collection of grab samples on a monthly basis for 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). Sludge samples were also collected from the UASB from different depths (only once) for analysis. The collected samples were brought to the Environmental Laboratory of the Thapar University 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:

	Sampling point				
Parameter	**Inlet (P1)**	**UASB outlet (P2)**	**Pre-areation outlet (P3)**	**Outlet (P4)**	**Sludge (P5)**
pH	✓			✓	
Temp.	✓	✓		✓	✓
BOD	✓	✓		✓	
COD	✓	✓		✓	
MPN	✓	✓	✓	✓	
TSS	✓			✓	
TDS	✓			✓	
TKN	✓	✓	✓	✓	
Nitrate+Nitrite	✓	✓		✓	
Chloride	✓			✓	
Sulphate	✓			✓	
Alkalinity	✓			✓	
VSS					✓
VFA					✓

Table 3.2 Analytical techniques for testing of wastewater parameters:

Sr. No.	Parameter	Method	Reference
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 103-105c.	APHA (1999) "manual standard method" 20th edition (2540: D)
7	TDS	Total dissolved dried at 180c.	APHA (1999) "manual standard method" 20th edition (2540: C)
8	TS	Total solids dried at 103-105c.	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-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 the 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:

3.5.1Upflow Anaerobic Sludge Blanket Reactor

3.5.1.1 Volumetric hydraulic load and Hydraulic detention time:

$$= - \qquad\qquad ---(3.1)$$

Where:

= volumetric hydraulic load ($m^3/m^3.d$)
= flow rate (m^3/d)
= total volume of the reactor (m^3)

$$t = \frac{V}{Q} \qquad\qquad ---(3.2)$$

Where:

t = hydraulic detention time (d)

Table 3.3 Recommended HRTs for UASB reactors treating domestic sewage

Sewage temperature (°C)	**Hydraulic Retention Time**	
	Daily average	**Minimum (during 4-6 hr)**
16 to 19	>10 to 14	> 7 to 9
20 to 26	> 6 to 9	> 4 to 6
> 26	> 6	> 4

Source: Chernicharo, 2007 (adapted from Lettinga and Hulshoff Pol, 1991)

3.5.1.2 Organic loading rate (Lv)

$$\mathrm{Lv} = \frac{\mathrm{Q} * \mathrm{So}}{\mathrm{V}} \qquad --- (3.3)$$

Where:

Lv = volumetric organic loading rate (kgCOD/m^3.d)

Q = flow rate (m^3/d)

So = influent substrate concentration (kgCOD/ m^3)

V = total volume of the reactor (m^3)

3.5.1.3 Upflow velocity and reactor height

The upflow velocity of the liquid is calculated as:

$$\mathrm{v} = \frac{\mathrm{Q}}{\mathrm{A}} \qquad --- (3.4)$$

where:

v = upflow velocity (m/hr)

Q = flow (m^3/hr)

A = cross sectional area of the reactor

Table 3.4 Recommended upflow velocities for design of UASB reactors treating domestic sewages.

Influent flowrate	Upflow velocity (m/hr)
Average flow	0.5 to 0.7
Maximum flow	<0.9to 1.1
Temporary peak flow	<1.5

Source: Chernicharo, 2007 (adapted from Lettinga and Hulshoff Pol,1995)

3.5.1.4 UASB reactor efficiencies:

Efficiencies of the UASB reactors are estimated mainly by means of empirical relations.

$$\mathrm{E} = 100 \times (1 - 0.68 \times \mathrm{t}^{\cdot}) \qquad - - - (3.5)$$

$$\ln(1 - \mathrm{E}) = \ln(\mathrm{a}) - \mathrm{b} \ \ln \mathrm{t} \qquad - - - (3.6)$$

Where:

E = efficiency of UASB reactor in term of COD removal (%)
t = hydraulic detention time (hr.)
0.68 = empirical constant
0.35 = empirical constant

$$\mathrm{E} = 100 \times (1 - 0.70 \times \mathrm{t}^{\cdot}) \qquad - - - (3.7)$$

$$\ln(1 - \mathrm{E}) = \ln(\mathrm{a}) - \mathrm{b} \ \ln \mathrm{t} \qquad - - - (3.8)$$

Where:

E = efficiency of UASB reactor in term of BOD removal (%)
t = hydraulic detention time (hr.)
0.70 = empirical constant
0.50 = empirical constant

In the above expressions there are two parameters. Numerical values for these two parameters were obtained for the STP in question through least squares method of linear curve fitting. Equations used for this include;

Slope (b_1 or b_2): $= \frac{}{}$ $\qquad - - - (3.9)$

Intercept (a_1 or a_2):

$$\mathrm{a} = \mathrm{y} - \mathrm{b}.\mathrm{x} \qquad - - (3.10)$$

Here S_{xx} and S_{yy} are:

$$S \quad = n \quad x \; - \qquad x \qquad\qquad --- (3.11)$$

$$S \quad = \; n \quad x\,y \; - \qquad x \qquad y \quad --- (3.12)$$

Effluent total COD or BOD:

$$C \quad = So - \frac{\cdot}{\quad} \qquad\qquad --- (3.13)$$

Where:

C = effluents total COD and BOD concentration (mg/L)

So = influent total COD and BOD concentration (mg/L)

E = COD and BOD removal efficiency (%)

3.5.1.6 Biogas production

Portion of COD converted into methane gas:

$$COD \quad = \; Q \times (So - S) - Y \quad \times Q \times So \qquad --- (3.14)$$

Where:

COD = COD load converted in to methane (Kg COD_{CH4}/d)

Q = average influent flow (m^3/d)

So = influent COD concentration (kg COD/m^3)

S = effluent COD concentration (kg COD/m^3)

Y = Coefficient of solid production in the system, in term of COD (0.11 to 0.23 kg COD sludge/kg CODapplied)

3.5.1.7 Sludge production:

Estimation of the mass of sludge produced in UASB reactors can be done by:

$$\mathrm{P} = \mathrm{Y} * \mathrm{COD} \qquad ---(3.15)$$

Where:

P = production of solids in the system (kg TSS/d)
Y = yield or solids production coefficient (kg TSS/kg $\mathrm{COD_{app}}$)
COD = COD load applied to the system (kgCOD/d)

Values of Y reported for the anaerobic treatment of domestic sewage are in order of 0.10to0.20 kg TSS/kg $\mathrm{COD_{app}}$.

Volumetric sludge production can be estimated by:

$$\mathrm{V} = \frac{\mathrm{P}}{\gamma * (\mathrm{C} \ \ /100)} \qquad ---(3.16)$$

Where:

= volumetric sludge production (m^3/d)
= sludge density (usually in order of 1020 to 1040 kg/m^3)
= solid concentration in the sludge (%)

3.5.2 Polishing pond:

For design analysis purposes the polishing pond has been considered as similar to a facultative pond.

3.5.2.1 Surface BOD loading ((kg BOD/ha/d):

$$= \frac{\qquad}{\qquad} \qquad ---(3.17)$$

Where:

L_i = influent BOD (kg BOD_5/d)

Q = flow rate (m^3/d)

= Area of facultative pond (m^2)

Design value of λ_s :

$$\lambda s = 350 \times (1.107 - 0.002 \times T)^{(T-25)} \quad ---(3.18)$$

Where:

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

Organic matter removal efficiency:

$$\lambda \ = 0.725\lambda \ + 10.75 \qquad --- (3.19)$$

$$\lambda \ = 0.79\lambda \ + 2 \qquad --- (3.20)$$

$$\lambda \ = 0.83679\lambda \ - 4.86 \qquad --- (3.21)$$

$$\lambda \ = 0.956\lambda \ + 1.31 \qquad --- (3.23)$$

Retention time (t) was calculated from:

$$= \frac{\quad}{\quad} \qquad --- (3.24)$$

Where;

H = pond depth (usually 1.5m)

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

= Area of facultative pond (m^2)

3.5.2.2 Coliform removal

$$\frac{\quad}{\quad} = \frac{1}{(1+ \qquad)\ 1+ \qquad (1+ \qquad)} \qquad --- (3.25)$$

Where;

Subscript f refers to the facultative pond

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

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

= hydraulic retention time

= coliform die–off coefficient

$$= \qquad (\quad) \qquad --- (3.26)$$

Where:

= coliform die–off coefficient at 20°C, taken as 2.6 (Marais, 1974)

T = temperature (°C)

= temperature coefficient, taken as 1.19 (Marais, 1974)

3.5.2.3 Ammonical nitrogen removal

Equation used when temperature is below 20°C:

$$= \frac{}{1+[(\ /\ ).(0.0038+0.000134.\ ).^{(\cdot \quad \cdot \quad \cdot).(\quad \cdot)}]} \qquad ---(3.27)$$

Equation used when temperature is more than 20°C:

$$= \frac{}{1+\ 5.035\times 10\ \ .(\ /\ ).^{\cdot \quad \times(\quad \cdot)}} \qquad ---(3.28)$$

Where;

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

= 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)

3.5.2.4 Total nitrogen removal

Equation used in case of facultative and maturation ponds (Reed, 1995):

$$= \quad \exp\{-[0.0064(1.039) \quad][\ +60.6(\quad -6.6)]\} \qquad ---(3.29)$$

Where;

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

= 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 (STP) studied

111 MLD capacity UASB and Polishing Pond STP, installed and commissioned in Bhattian, Ludhiana by Punjab Water Supply and Sewerage Board under the Satluj Action Plan was studied. Schematic diagram of the STP is given in figure-3.1. The STP included the following units and facilities:

- Bar screen
- Sewage collection sump
- Raw sewage pumps
- Bar screen
- Grit chambers/channels (both manual and mechanical)
- UASB reactor
- Pre-aeration tank
- Polishing pond
- Chlorination unit (not yet commissioned)
- Sludge drying beds
- Biogas storage, handling and flaring system

Dimensional and capacity details of these units are given in table 3.5. Schematic process flow diagram of the STP is shown in figure-3.1. Wastewater conveyed to the STP is collected into a raw sewage sump through mechanically cleaned bar screens, and from there, it is pumped with the help of 10 of 140hp number raw sewage pumps and passed through different units of the STP. The pumped raw sewage is metered with the help of an online flow meter. The pumped wastewater is first passed through a screen then degritted in both mechanical grit chambers and manual grit channels. The degritted sewage is passed through 3 division boxes and uniformly distributed among 10 distribution boxes. From the distribution boxes the wastewater is loaded to the 9 UASB cells at the bottom through distribution tube for getting uniform upflow velocity in the UASB reactor. In the UASB primary treatment of the wastewater occurs. Suspended biodegradable and non-biodegradable solids are removed and stabilized anaerobically producing biogas. Wastewater from the UASB is allowed flow under gravity into the polishing pond through pre-aeration tank (where toxic and inhibitory gases are removed by air stripping). Secondary treatment of the wastewater occurs in the polishing pond. 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 come out as treated effluent. This treated effluent is supposed to be chlorinated for pathogen removal prior to discharge. Sludge accumulated in the UASB reactor as and when needed is drained and loaded on the sludge drying beds for dewatering and drying. Biogas produced in the UASB reactor after mist elimination is collected into a floating gas dome and excess biogas is metered and flared.

Table3.5: Dimensional details of various units and facilities of the STP.

Unit	Dimensions
Bar screen (both manual and mechanical)	Manual screen having 6mm width, 40mm spacing. Mechanical reciprocating type of screen.
Sewage collection sump	2 sumps both are interconnected.
Raw sewage pumps	12 pumps, 10 of 140 hp capacity and 2 of 20 hp capacity.
Bar screen	6mm width, 10mm spacing
Grit chambers/channels (both manual and mechanical)	Mechanically operated grit chamber with scraper.
UASB reactor	9 cells of UASB each of 30m length, 32m width, 5.06m liquid depth and 0.80 m freeboard.
Pre-aeration tank	Single unit having size of 12m length, 20m width, 3.5m depth.
Polishing pond	Single pond having dimensions of 630m length, 270m width, 1.8m of liquid depth and 0.2m freeboard. Two baffles at distance of 90m along the width.
Chlorination unit	not yet commissioned
Sludge drying beds	72 beds each of 256m^2 area and depth of 0.25m.
Biogas storage, handling and flaring system	Gas stored in gas holder of 16m dia, 1000m^3 of capacity.

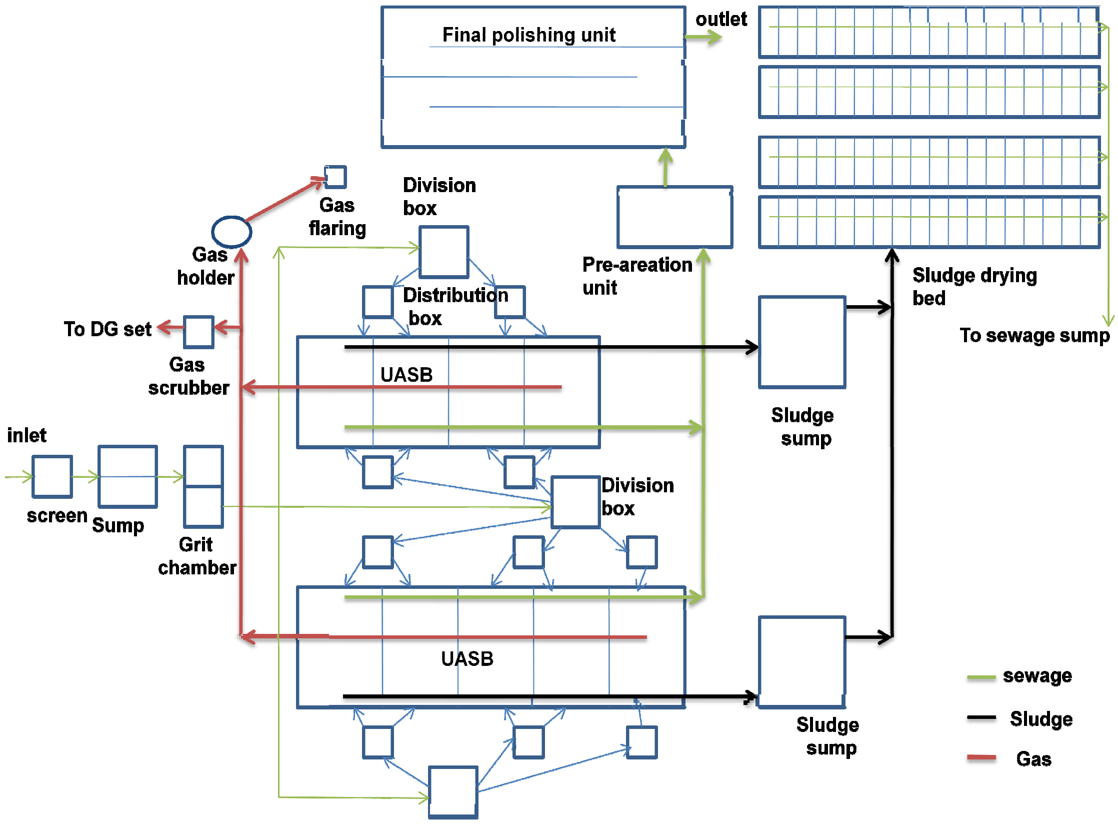

Figure -3.1: Schematic Process Flow diagram of UASB and Polishing Pond based STP

CHAPTER - 4

RESULTS AND DISCUSSION

4.1 Introduction

Results obtained from the treatment process monitoring, performance evaluation and design analysis of the STP at Bhattia, Ludhiana are presented and discussed. 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), UASB reactor outlet (P2), inlet of final polishing unit (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. Overall plant level and individual unit level actual removal efficiencies for different pollutants obtained for the four monitoring months are given in tables 4.1 to 4.4 and in figure 4.1. Figure 4.1 includes only BOD, TSS, total-N, Total-P and MPN removal efficiencies Average removal efficiencies for the period February to May, 2009 are given in table -4.5.

4.2 Performance Evaluation

4.2.1 Overall performance of the STP

STP at Bhattia, Ludhiana mainly included an UASB reactor and a polishing pond. here the polishing pond can be considered similar to a facultative pond of a waste stabilization pond system. The STP was designed for treating 111 MLD flow. Characteristics of the sewage for which the STP was designed are not known. The STP is supposed to treat the wastewater to comply with the applicable effluent standards of BOD: 30 mg/L, TSS: 100 mg/L, and Pathogens: 100000 cfu/100mL. Nutrient and pathogen removal is coincidental here. For avoiding eutrophication of the receiving water body, nutrients removal to <15 mg/L in case of total nitrogen and <2 mg/L in case of total phosphorus may be needed. Algal cell concentration (in the form of TSS) is supposed to be higher (more than 50 to 150 mg/L) than that expected for Waste Stabilization Ponds (WSP).

BOD removal by the STP was around 86% and the treated effluent BOD was around 33 mg/L. similarly COD removal was 78% and the treated effluent COD was around 137 mg/L. from the angle of BOD and COD removal, performance of the STP is quite satisfactory. Relatively higher BOD and COD in the treated effluent is actually due to the higher algal cell concentration. TSS of the treated effluent was around 75 mg/L which is within the limits prescribed. Slight modifications to the outlet of the polishing pond can reduced the algal cell washout and ensure meeting the effluent standards prescribed.

As expected, pathogen removal efficiency was not very high. It was around 95% while the requirement is removal by 4 to 5 log units. It is widely reported that UASBs are highly inefficient in the pathogen removal and facultative ponds can remove the pathogens to the level recommended for the reuse of the effluent for restricted use. If looked from this angle the effluent was not complying with the prescribed requirement. The coliform count was reduced from 3.3×10^7 to 2.7×10^6 cfu/100 mL. Observations on nutrient 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 and highly variable (standard deviations were much higher than means). Source for most of these variations and errors was highly variable and low nutrient level in the raw sewage. Organic nitrogen removals were very low (around 39%). This may be mainly because of high algal cell concentration in the treated effluent (around 75 mg/L TSS in the treated effluent). Low ammonical nitrogen removal efficiencies are quite common with UASB reactors and with facultative pond systems because nitrification and denitrification processes are very insignificant. Phosphorus and nitrogen levels in the treated effluent were high enough (phosphorus around 4.1 mg/L and total nitrogen around 32 mg/L) to cause eutrophication problems in the receiving water bodies.

Table -4.1: performance of the STP during February 2009

February 2009 (11:30 AM)
Water temp.:22.0°C Air temp.: 14.0°C
Flow rate: 149MLD

Parameter	**Inlet**	**Outlet**	**ŋ(UASB)%**	**ŋ(FPU)%**	**ŋ(overall)%**
BOD (mg/l)	270	37	72	50	86
COD (mg/l)	750	200	33	60	73
MPN/100	$8x10^6$	$1.7x10^6$	37	66	78
TSS (mg/l)	510	140	68	12	72
NH4-N (mg/l)	52	41.7	36	**-26**	19
Org-N (mg/l)	7.06	6.16	17	**-5**	12
Total-N (mg/l)	59.1	48.7	35	**-23**	17
Total-P (mg/l)	0.081	0.092	-45	22	**-13**

Table -4.2: performance of the STP during March 2009

March 2009 (10:15 AM)

Water temp.:24.7°C Air temp.: 25.4°C

Flow rate: 122.5MLD

Parameter	Inlet	Outlet	ŋ(UASB)%	ŋ(FPU)%	ŋ(overall)%
BOD (mg/l)	293	17	84	60	93
COD (mg/l)	1000	129	40	78	87
MPN/100	$5x10^6$	$2.3x10^6$	30	34	54
TSS (mg/l)	450	10	74	91	97
NH4-N (mg/l)	45.3	23.2	19	36	48
Org-N (mg/l)	2.1	1.7	**-39**	83	19
Total-N (mg/l)	47.4	25.3	2.1	50	46
Total-P (mg/l)	0.383	0.076	75	19.	80

Table -4.3: performance of the STP during April 2009

April 2009 (10:30 AM)
Water temp.:28°C Air temp.: 32.5°C
Flow rate: 66MLD

Parameter	**Inlet**	**Outlet**	**ŋ(UASB)%**	**ŋ(FPU)%**	**ŋ(overall)%**
BOD (mg/l)	200	36	73.	32	82
COD (mg/l)	347	100	40	16	50
MPN/100	$1.3x10^{7}$	$2.2x10^{6}$	46	63	83
TSS (mg/l)	420	20	80	75	95
NH4-N (mg/l)	28.8	21.8	33	**-13**	24
Org-N (mg/l)	5.3	1.9	-40	73	63
Total-N (mg/l)	34.1	27	23	3.8	20
Total-P (mg/l)	7.037	7.452	**-16**	8.8	**-5.8**

Table -4.4: Performance of the STP during May 2009

May 2009 (11:25 AM)

Water temp.:30.8°C Air temp.: 36.6°C

Flow rate: 84MLD

Parameter	Inlet	Outlet	ŋ(UASB)%	ŋ(FPU)%	ŋ(overall)%
BOD (mg/l)	218	44	78	7.3	79
COD (mg/l)	500	120	68	25	76
MPN/100	2.4×10^8	7×10^6	33	95	97
TSS (mg/l)	140	130	24	**-22**	7.1
NH4-N (mg/l)	34.7	26.3	**-0.8**	24	24
Org-N (mg/l)	2.8	0.56	10	77	80
Total-N (mg/l)	37.5	27.3	2.7	25	27
Total-P (mg/l)	6.3	8.8	**-9.7**	**-27**	**-39**

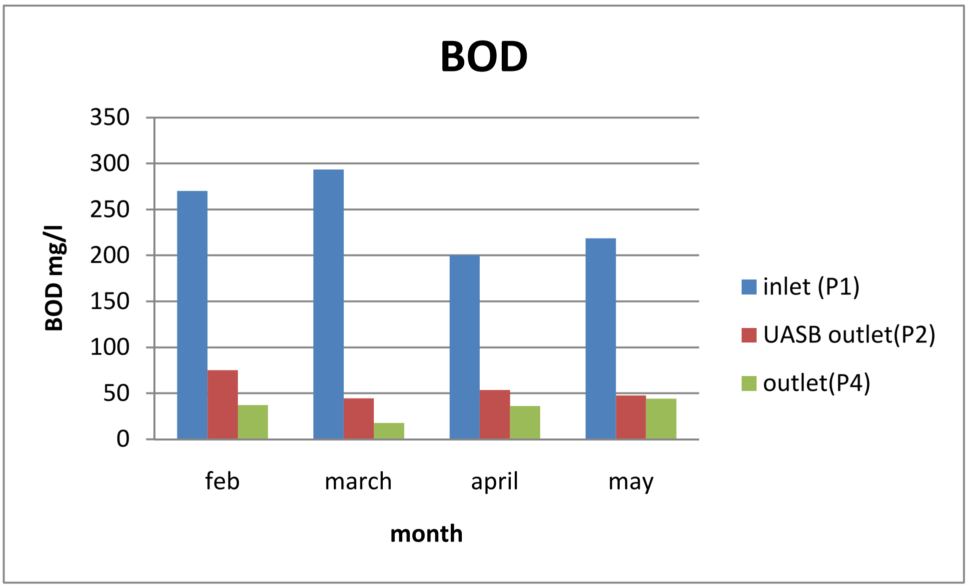

Figure :4.1(a) BOD removal pattern

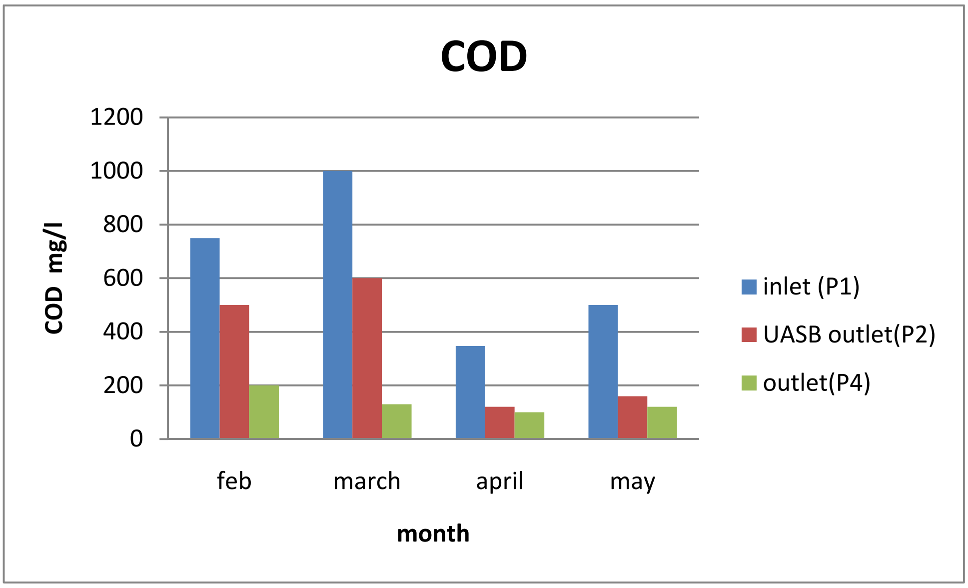

Figure :4.1(b) COD removal pattern

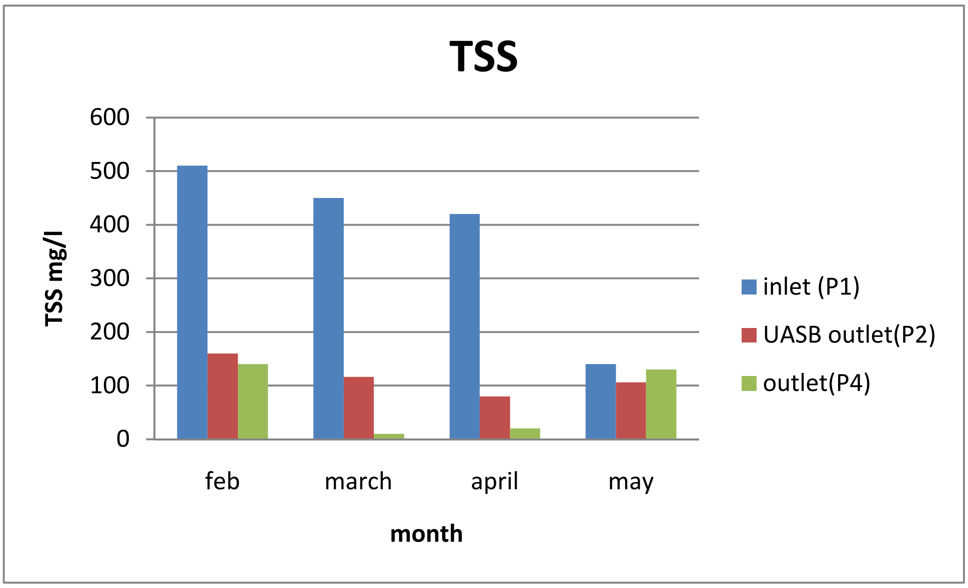

Figure :4.1(c) TSS removal pattern

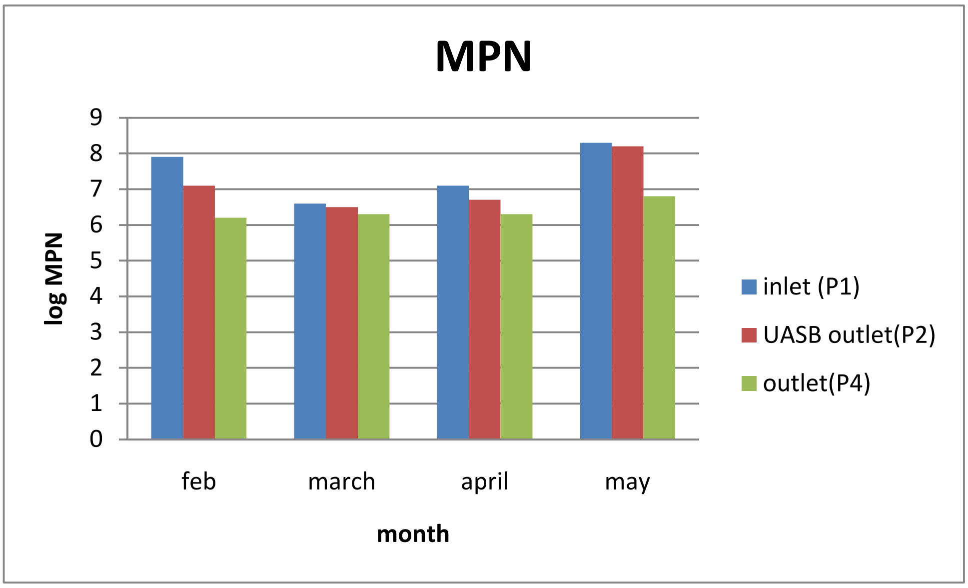

Figure :4.1(d) MPN removal pattern

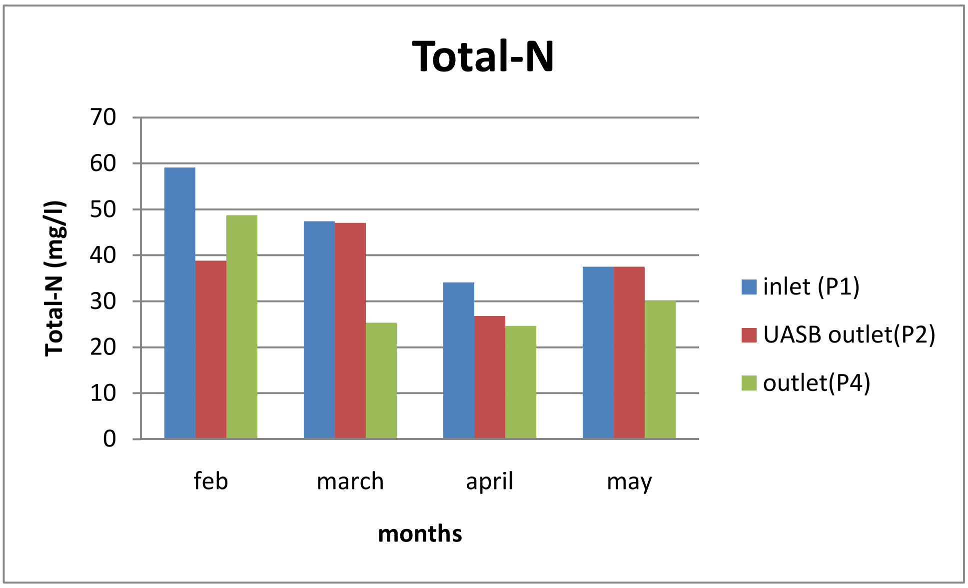

Figure :4.1(e) Total nitrogen removal pattern

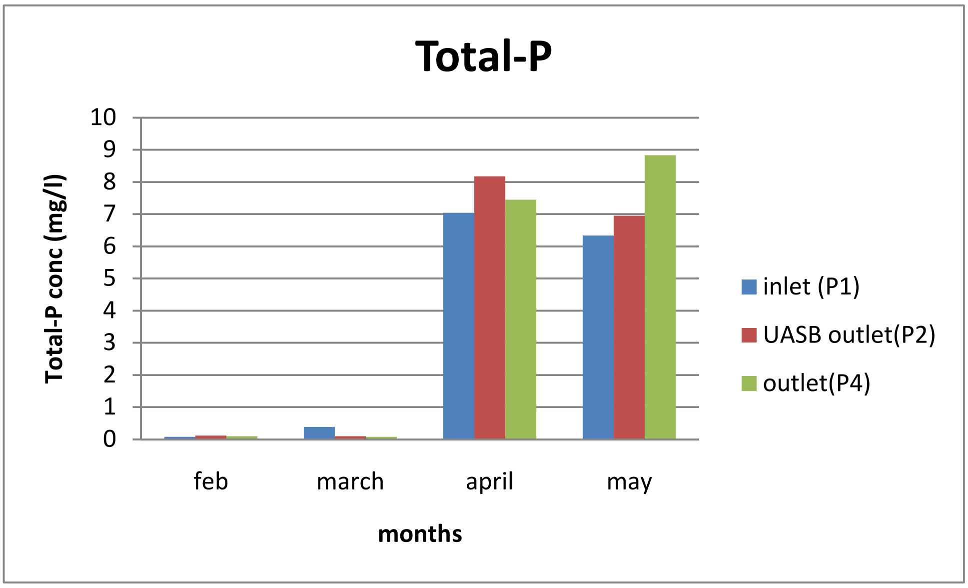

Figure :4.1(f) Total Phosphorus removal pattern

4.2.2 Performance of the UASB reactor

UASB reactor is supposed to work as a primary treatment unit and stabilize the sludge getting separated in the reactor with typical COD removal efficiency of 70-75% (http://www.waterandwastewater.com/www_services/ask_tom_archive/toc.htm). Removal of pathogens and nutrients is very small. The organic matter removed and the sludge stabilized is mostly converted into biomass and a small fraction is transformed into active anaerobic biological sludge. Nutrient removal mostly occurs through assimilative use by the newly synthesized anaerobic active biomass. Pathogen removal is very negligible due to two reasons: very low HRT (around 8 hours).

The COD and BOD removal efficiencies observed for the UASB in question are rather low (around 46% and 77% respectively). This may be due to the higher fraction of industrial wastewater, specially from textile dyeing industry of Ludhiana city. This is evident from the low BOD to COD ratio (0.37) of the raw sewage entering the STP. Grab sampling apparently introduced errors in case of nutrient removal efficiency estimations. While ammonical nitrogen and total nitrogen removals were positive (around 22% and 14% respectively), organic nitrogen and total phosphorus gave negative removal efficiencies. Pathogen removal was around 57%. This is lower but closer the expected removal efficiency estimated by the equation – 3.25. The expected removal efficiencies are around 97-98%.

4.2.2 Performance of Final Polishing Unit (FPU)

Facultative ponds are supposed to reduce BOD by about 83.5% during coldest winter month and do not change much with increasing temperature. Presence of algal cells in the effluent from the facultative pond introduces 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 not possible. Pathogen removal expected at the design temperature (17C) is 65% and it is supposed to increase with increasing temperature and go beyond 76% as the temperature crosses 30C. Designed total nitrogen and ammonical nitrogen removals, according to the equations 3.24 and 3.22, are 25% and <5% 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 38 and 44% respectively. These removals are much lower than the design removals. Algal cells in the effluent from the facultative pond could be responsible. COD removal higher than the BOD removal might have been due to the settling removal of non-biodegradable suspended organic matter in the facultative ponds. Observed pathogen removals were higher than the design removal values (actual was 80% while expected removal is 65%). The higher removal might have been due to the higher temperature (22 to 30.8°C while design temperature was 17°C). Observed ammonical nitrogen removal was around 9%, which is closer to the expected value of 5% (higher temperature might have been responsible for this). Observed total nitrogen removal was however lower than the expected (around 12% while design removal is 25%). Observed removal values for all 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 negative. In addition to grab sampling, effluents having higher concentration of algal cells (rich in nutrients) specially during 10-15 to 11-30 AM might have been responsible for the negative efficiencies.

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

Feburary to May 2009 (10:10am to 11:30AM)

Water temp.: 21°C-31°C Air temp.: 14°C- 37°C

Flow rate: 65-150MLD

	Concentration (mg/L)Mean ± S.D		**Treatment Efficiency (%) Mean ± S.D**		
Parameter	**Inlet**	**Outlet**	**Ŋ(UASB)**	**Ŋ(FPU)**	**Ŋ(overall)**
BOD(mg/l)	245±43.6	33±11	77±5.5	38±23	86±6
COD(mg/l)	649±286	137±43	46±15	44±30	78±15.5
TSS(mg/l)	380±164	75±69	69±25.4	35±53	80±42
MPN/100	6.6x10^7	3.3x10^6	57±7	80±25	95±17
Amm-N(mg/l)	40.2±10	28.2±9	22±16	8.8±29.5	29±13
Org-N(mg/l)	4.3±2.2	2.5±2.4	**-51±30**	60±41	39±33
Total – N(mg/l)	44.5±11.2	32±11.1	14±16.5	12±29.3	27.5±13
Total-P(mg/l)	3.4±3.2	4.1±4	**-10±51**	**-7.2±22**	**-18±51**

4.3 Design analysis of the STP

Average ambient air temperature for the coldest winter month of the year for Bhattia is 17°C. Design capacity of the STP is 111MLD. Both ambient air temperature and wastewater temperature, and flow rates of the sewage were recorded at the time of sampling. Grab sampling was practiced and the samples were mostly collected between 10:10 AM and 11:30 AM.

4.3.1 UASB reactor:

Volume and area of the UASB reactor are 43718.4 m^3 and its designed is HRT 9.4 hr. Volume of the digestion zone is 33350 m3 and design HRT of the digestion zone is 7.2 hours. Design volumetric loading rate according to the equation 3.1 was calculated as 3.0 m^3/m^3.d. Typical organic loading rate (kg COD/m3.day) for medium strength of waste water is 2.4 kg/m^3.d (table 2.2). Upflow velocity in the UASB reactor for the design flow is 0.52 m/hour. Methane production per kg of COD removed, according to the equation 2.13 is 0.35 Nm3 and biogas production is 0.53 Nm3 (assuming 65% methane in the biogas). Treatment efficiencies of the UASB reactor, according to the empirical equations 3.5 and 3.7, expected are 66% for COD and 74 for BOD.

Nutrient removals in the UASB are usually insignificant and can be equated to the nutrients assimilated by the microbial biomass synthesised. For nutrient assimilation removal calculations, net biomass yield coefficient was taken as 0.1 of the COD removed and the microbial biomass was assumed to have 12.3% nitrogen and 2.3% phosphorus. For pathogen removal calculations the equation used for anaerobic ponds of the waste stabilization pond system was used. Numerical constants of the empirical equations used for BOD and COD removal efficiency calculations corrected to the conditions of the UASB in question by linear curve fitting.

The design analysis calculations which included volumetric loading are presented in the tables 4.6 and 4.7. Volumetric loading rates were highly variable and ranged between 1.98 and 4.47 m3/m3.day and as a consequence the upflow velocity was also highly varying from 0.32 to 0.7 m/hour. Please see table 4.6 for details. This must be resulting in operational instability and reduced efficiency of working. Despite this, the treatment efficiencies were observed to be higher than the expected. Observed efficiencies were 71-87% for COD and 79-93% for BOD while expected efficiencies calculated according to the equations 3.9 to3.12 are 62-72% and 70-80% respectively. See table 4.7 for details. This indicates that the equations used were underestimating the efficiency, and this may be because of the differences in the characteristics of the sewage being treated. The equations used may require calibration. The STP was frequently overloading instead of 111 MLD the STP was loaded with as high as149 MLD sewage.

Biogas production rates were not monitored. However expected biogas production rates have been estimated on the basis of the amount of COD actually being converted into methane or biogas. Amount of COD removed in the UASB minus the amount used up in the synthesis of active anaerobic microbial biomass was taken as the COD converted into methane. The amount of COD utilized in the biomass synthesis was taken 14.2%. Further, methane content of the biogas generated was taken as 65%. It appears that with increasing organic loading amount of biogas generated per unit COD removal also increases. And the organic loading rates were also highly variable from 0.7 to 3.7 kg/m3.day.

Table 4.6 Design analysis calculations for UASB reactor

Month	Parameters				
	Volumetric hydraulic loading rate (m^3/m^3.d)	**Volumetric Organic loading rate (kgCOD/m^3.d)**	**Upflow velocity (m/hr)**	**HRT (hr)**	**Estimated CH_4 production rate (Nm^3/d)**
Feb	4.5	3.3	0.7	5.2	23,577
March	3.7	3.7	0.6	6.4	32,527
April	2.0	0.7	0.32	12	4,754
May	2.5	1.25	0.42	9.3	9,567

Table 4.7 Efficiencies calculation for the UASB reactor

	COD removal efficiency (%)		BOD removal efficiency (%)	
Month	**Expected**	**Observed**	**Expected**	**Observed**
Feb	62	73	70	86
March	65	87	73	93
April	72	71	80	82
May	69	76	77	79

In view of underestimation of the removal efficiencies, the equations available from literature were calibrated using the monitoring data of the present study. For calibration the two numerical values were considered as constants/parameters ‘a and b’ (in COD removal efficiency equations they were considered as a1 and b1 and in BOD removal efficiency equations as a2 and b2 as shown below. Then the two equations were linearized into Y=mX+C. After this through linear curve fitting values for the parameters a and b were obtained. With the help of the new parameters the removal efficiencies were estimated. Results of these calibrations are given in table 4-8. New values for the numerical constants/parameters are 0.61, 0.62, 0.25 and 0.32 for a1, a2, b1 and b2 respectively.

Table 4.8: Results of the efficiency equations calibration

	‘a’ value		‘b’ value		(New) Expected % removal efficiency	Actual % removal efficiency
	Original	New	Original	New		
COD	0.68	0.61	0.35	0.25	60	73
					62	87
					68	71
					66	76
BOD	0.7	0.62	0.5	0.37	68	86
					69	93
					76	82
					73	79

4.3.2 Design of Final Polishing Pond:

Facultative pond

Area of the polishing pond is 170100 m^2 and its designed HRT is 3days. Designed surface loading rate according to the equation 2.18 at 17°C was calculated as 200 kg/ha.d. Designed organic matter removal efficiency for winter coldest month, according to the equation 3.23 is expected as 94%. Expected designed pathogen removal efficiency calculated according to the equation 3.25 is 65%. Expected designed total nitrogen removal efficiency calculated from equation 3.29 is 25%.

Calculations relating to the design analysis of the facultative ponds are given in table 4.9. 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 one of the four months of the study. The reason for this could be the variation in the hydraulic loading rate. Actual removal efficiencies were much lower than the expected removals (around 37.3% removal was observed against expected 94%). 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 4.9: Design analysis calculations for the Final Polishing pond:

Parameter		Feb	March	April	May
HRT (days)		2.28	2.7	5.15	4
Surface loading rate (kg/ha/d)	Design value	200	200	200	200
	Actual value	328	160	104	117
	Maximum allowed at sewage temp.	291 at22°C	355 at24.7°C	294 at28°C	244.4 at30.8°C
Organic matter removal efficiency (%)	Actual value	50	60	32	7
	Efficiency expected	95	95	95	95
Pathogen removal (%)	Actual value	86	34	63	95
	Efficiency expected	89	94	99	99
Nutrient removal (%)	Actual value	-35	41	25	16
	Efficiency expected	30	33	37	40

CONCLUSION

Performance evaluation and design analysis of a Upflow Anaerobic Sludge Blanket Reactor and Polishing Pond 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 treated effluent was found almost complying with the prescribed standards. BOD, COD, TSS and MPN of the treated effluent were 33±11 mg/L, 137±43 mg/L, 75±69 and $3.3x10^6$ respectively. Excepting for MPN, if not for the role played by the algal cells in the treated effluent, the treatment process is satisfactory indicating that plant is working properly.

Average BOD in the effluent is 33mg/l. Reduction of coliform count was not satisfactory as expected and effluent from the polishing pond needs further treatment. Chlorination of the effluent is under commissioning, but higher algal cell concentration may make chlorination not that appropriate. The design analysis indicated that the design of the STP is adequate and appropriate. Hydraulic loading rates have been found frequently going beyond the designed capacity.

1. Concentration for different parameter at different sampling point during month of Feb.

Bhatia STP (FEB) Sampling time: 11:32AM

Parameter	Inlet(P1)	(P2)	(P3)	Outlet(P4)	Sludge(P5)
Flow rate(m^3/hr)	149	-	-	-	-
TSS(mg/l)	510	160	-	140	-
pH	6.92	-	-	7.53	-
Temperature (°C)	22	-	-	18	-
BOD (mg/l)	270	75	-	37	-
COD (mg/l)	750	500	-	200	-
NH4-N (mg/l)	52.08	33.04	30.24	41.72	-
Org-N (mg/l)	7.06	5.83	5.71	6.16	-
Sulphate (mg/l)	41	-	-	-	-
Chloride (mg/l)	166.19	-	-	301.75	-
Total-P (mg/l)	0.081	0.118	-	0.092	-
Total coliform (MPN/100)	8×10^6	1.3×10^7	-	1.7×10^6	-
Nitrate+Nitrite (mg/l)	-	-	-	0.86	-
VSS (%)	-	-	-	-	44.55

2 Concentration for different parameter at different sampling point during month of March.

Bhatia STP (March) Sampling time: 10:15AM

Parameter	Inlet(P1)	(P2)	(P3)	Outlet(P4)	Sludge(P5)
Flow rate(m^3/hr)	122.5	-	-	-	-
TSS(mg/l)	450	116	-	10	-
pH	7.09	-	-	7.26	-
Temperature (°C)	24.7	-	-	23	-
BOD (mg/l)	293.5	44.5	-	17.8	-
COD (mg/l)	1000	600	-	129.6	-
NH4-N (mg/l)	45.36	36.68	34.16	23.24	-
Org-N (mg/l)	2.1	10.36	9.52	1.7	-
Sulphate (mg/l)	699.72	-	-	-	-
Chloride (mg/l)	94.29	-	-	301.75	-
Total-P (mg/l)	0.383	0.094	-	0.076	-
Total coliform (MPN/100)	5×10^6	3.5×10^6	-	2.3×10^6	-
Nitrate+Nirite (mg/l)	-	-	-	0.409	-
VSS (%)	-	-	-		41.4

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

Bhatia STP (April) Sampling time: 10:33AM

Parameter	Inlet(P1)	(P2)	(P3)	Outlet(P4)	Sludge(P5)
Flow rate(m^3/hr)	66	-	-	-	-
TSS(mg/l)	420	80	-	20	-
pH	7.09	-	-	7.26	-
Temperature (°C)	28.4	-	-	27.6	-
BOD (mg/l)	200	53.5	-	36	-
COD (mg/l)	347.5	120	-	100	-
NH4-N (mg/l)	28.84	19.32	23.24	21.84	-
Org-N (mg/l)	5.32	7.50	9.57	1.96	-
Sulphate (mg/l)	353.97	-	-		-
Chloride (mg/l)	186.2	-	-	282.89	-
Total-P (mg/l)	7.037	8.175	-	7.452	-
Total coliform (MPN/100)	13×10^6	6×10^6	-	2.2×10^6	-
NO_2 +NO_3 (mg/l)	-	-	-	0.86	-
VSS (%)	-	-	-	-	39.3
Alkalinity			-	-	-

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

Bhatia STP (May) Sampling time: 10:33AM

Parameter	Inlet(P1)	(P2)	(P3)	Outlet(P4)	Sludge(P5)
Flow rate(MLD)	84	-	-		-
TSS(mg/l)	140	80	-	130	-
pH	7.09	-	-	7.26	-
Temperature (°C)	30.8	-	-	29.4	-
BOD (mg/l)	218.5	47.5	-	44	-
COD (mg/l)	500	160	-	120	-
NH4-N (mg/l)	34.7	35	33.6	26.3	-
Org-N (mg/l)	2.8	2.5	2.7	0.56	-
Sulphate (mg/l)	234	-	-	-	-
Chloride (mg/l)	248	-	-	299	-
Total-P (mg/l)	6.34	6.95	-	8.83	-
Total coliform (MPN/100)	2.4×10^8	1.6×10^8	-	7×10^6	-
NO_2 +NO_3 (mg/l)	-	-	-	3.274	-
VSS (%)	-	-	-	-	39.3
Alkalinity (mg/l)	565	615	-	-	-

5. Metal concentration at different point

Month	metals	P1 (µg/ml)	P2 (µg/ml)	P3 (µg/ml)
Feb	Fe	0.95	0.79	1.86
	Pb	0.23	0.16	0.1
	Cr	**BDL**	**BDL**	**BDL**
	Zn	0.08	0.07	0.07
	Ni	0.5	0.15	0.15
March	Fe	1.25	**BDL**	0.03
	Pb	**BDL**	0.007	0.12
	Cr	**BDL**	**BDL**	**BDL**
	Zn	0.06	0.08	0.01
	Ni	0.12	0.14	0.15
April	Fe	2.6	3.3	2.9
	Pb	0.05	**BDL**	0.1
	Cr	**BDL**	**BDL**	**BDL**
	Zn	1.4	0.4	0.02
	Ni	0.22	0.13	0.13

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