COMPUTER AIDED PROCESS DESIGNING OF SEWAGE TREATMENT PLANT WITH ACTIVATED SLUDGE PROCESS

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By

GUNSAGARDEEP SINGH
(Roll No. 601001011)

UNDER THE GUIDANCE OF

Mr. Amit Dhir
Assistant Professor

Department of Biotechnology and Environmental Sciences
Thapar University
Patiala-147004, Punjab

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DECLARATION CUM CERTIFICATE

I hereby declare that the work embodied in thesis entitled, "**Computer Aided Process Designing of Sewage Treatment Plant with Activated Sludge Process**", is an original piece of work and was conducted in the Department of Biotechnology and Environmental Sciences, Thapar University, Patiala. The matter presented in this thesis has not been submitted in part or full, to this or any other University/Institute for any degree or diploma.

Date: 13/07/2012

Gunsagardeep Singh
(Roll no. 601001011)

It is certified that the above statement made by the student is correct to the best of my knowledge and belief.

13/7/2012
Amit Dhir
Assistant Professor
Department of Biotech & Env. Sciences
Thapar University, Patiala.

Countersigned by:

Head
Department of Biotech & Env. Sciences
Thapar University, Patiala.

Dean, Academic Affairs
Thapar University, Patiala.

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Gunsagardeep Singh

(Roll No. 601001011)

Abstract

Sewage is generated by residential, institutional, commercial and industrial establishments. It includes household waste from toilet, baths, showers, sinks, kitchen and so forth that is disposed off sewers. Sewage treatment is the process of removing contaminants from commercial and household sewage, both runoff (effluents) and domestic. It includes physical, chemical, and biological processes to remove physical, chemical and biological contaminants. There are two fundamental reasons for treatment of sewage viz., prevention of pollution for the protecting of the environment, preventing the spread of water borne diseases for protection of public health. Proper design, construction together with good operation and maintenance are essential for sewage treatment plants (STP), in order to produce effluents which are satisfying the safe disposal standards prescribed by the regulatory authorities. In the present study a computer program in ASP.NET has been developed for comprehensive design of sewage treatment plant which incorporates activated sludge process as biological treatment method. All the units of STP were included in the design by referring various standard procedures and manuals. The validity of the software has been tested by comparing designed values with an existing STP. This program not only helps in sizing the treatment units but also helps in understanding the plant's capacity as well as in deciding the future expansion works needed for increased hydraulic and organic loadings. The program also incorporated sludge management where the sludge is stabilized in anaerobic digester and the biogas produced is used for power generation. In this way, the power requirement of the plant from outside sources can be reduced. The software is unique in itself as it is not needed to be installed in the computer but is available online with URL (www.oberoiinfra.co.in) and is password protected. This software can also work on mobiles through opera mini. The program is user friendly and interactive as it provides ranges to the user in the various stages of the design and it also gives message to the user for any value which is out of range. Some of the units in the program are optional and the user can skip them if he does not wish to design that units. The program along with the treatment units also calculates the capacity on different pumps required, oxygen required for aeration and power required for mixing in the digester. The program in the end provides result sheet showing the sizes of all the units calculated are given alongwith a flow diagram , thus provides complete process design with a facility of printout on A1 and A2 sheets. Hence, this computer aided design provides accurate, precise and time saving means for designing of STP.

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CHAPTER 1
INTRODUCTION

Sewage is essentially the water supply of the community after it has been fouled by a variety of domestic and commercial uses. The water supplied to a community receives a range of chemical substances and microbial flora during its use and the wastewater acquires a polluting potential which makes it health and environmental hazard. Communicable diseases of the intestinal tract such as cholera, typhoid, dysenteries and water borne diseases like infectious hepatitis etc., can be spread from uncontrolled disposal of sewage, and therefore prevention of communicable diseases and protecting public health attracts the primary objective of sanitary wastewater disposal. **[Arceivala, Soli J. Wastewater treatment for Pollution Control, Tata Mc GRAW - Hill, New Delhi, 1986]**

Some of the adverse environmental impact of allowing untreated sewage to be discharged in groundwater or surface water bodies and /or lands is as follow:

1. The decomposition of the organic materials contained in wastewater can lead to the production of large quantities of malodorous gases.
2. Untreated wastewater (sewage) containing a large amount of organic matter, if discharged into a river / stream, will consume the dissolved oxygen (DO) for satisfying the Biochemical Oxygen Demand (BOD) of wastewater and thus deplete the DO level of the stream, thereby causing fish killing and other undesirable effects.
3. Wastewater may also contain nutrients, which can stimulate the growth of aquatic plants and algal blooms, thus leading to eutrophication of the lakes and streams.
4. Untreated wastewater usually contains numerous pathogenic, or disease causing microorganisms and toxic compounds, that dwell in the human intestinal tract or may be present in certain industrial waste. These may contaminate the land or the water body, where such sewage is disposed. For the above-mentioned reasons the treatment and disposal of wastewater, is not only desirable but also necessary.

Given the characteristics of raw wastewater and the requirements of disposal or reuse, the wastewater usually requires some type of preparation or treatment before it is rendered fit for disposal or reuse. Generally, in many situations involving domestic wastewater, the treatment consists of removal of suspended solids and BOD_5, which are the two usual parameters of prime interest. The purpose of sewage treatment plant is to separate inorganic particulates and to stabilize the decomposable organic matter present in waste water so as to produce an effluent and sludge which can be disposed off in the environmentally safe manner. The complete treatment of wastewater is brought by a sequential combination of various physical, chemical and biological unit operations.

In **physical unit operations** change is brought about by the application of physical forces. The various operations involved in physical unit operations are screening, mixing, flocculation, sedimentation, floatation and filtration. In **chemical unit operations** change is brought about by means of or through chemical reactions. The chemical unit operations involve neutralization, coagulation and flocculation, chemical precipitation, oxidation, reduction, ion-exchange, chemical adsorption, reverse osmosis and electrodialysis. **Biological unit operations** involve microbial decomposition of organic matter, aerobically or anaerobically. The two main operations in biological unit operations are Suspended Growth and Attached Growth.

Suspended Growth Processes

In suspended growth processes the microorganism responsible for treatment are maintained in liquid suspension by appropriate mixing methods like using impellers and propellers, aeration etc. Many suspended growth processes are used in domestic and industrial wastewater treatment. The most common example is activated sludge process. (ASP)

Attached Growth Processes

In attached growth processes, the microorganisms responsible for the conversion of organic matter or nutrients are attached on inert packing material. The organic matter and the nutrients are removed from water flowing pass the attached growth known as bio film. Packing material in attached growth processes include rock, gravel, sand, slag, redwood and range of plastics and other synthetic materials. The packing may or may not be submerged completely in liquid, with air or gas space above the biofilm liquid layer. The common example of this process is trickling filter.

Environmental engineers are entrusted with designing wastewater treatment plants that are efficient and at the same time cost effective. Very often the designer has to compare various operations in order to achieve the above said objective, which requires colossal effort if done manually. Computer aided design is not only helpful in sizing the treatment units but also useful in checking the designs of existing plant with relevant input data.

Thesis Objectives

- Process designing of sewage treatment plant by ASP (Activated Sludge Process) Technology.
- Process designing of sludge handling system and production of biogas and power generation from biogas.
- To develop a comprehensive design program for the design of waste water treatment plant Using ASP.NET which is globally available online.

1.1. Computer Aided Process Designing of Sewage Treatment Plant with Activated Sludge Process

Proper design, construction together with good operation and maintenance are essential for sewage treatment plants (STP), in order to produce effluents which are satisfying the safe disposal standards prescribed by the regulatory authorities. In this work a computer program in ASP.NET has been developed for comprehensive design of wastewater treatment plant which incorporates activated sludge process as biological treatment method. All the units of STP are included in the design and the program is developed in a very user friendly manner by referring various standard procedures and manuals. The validity of the software has been verified by test running and comparison with an existing plant data. This program not only helps in sizing the treatment units but also helps in understanding the plant's capacity as well as in deciding the future expansion works needed for increased hydraulic and organic loadings.

1.1.1. Activated Sludge Process

The activated sludge process has already been used in practice for almost eighty years. Originally, it was developed in England by Arden and Lockett in 1914 **[Metcalf and Eddy, 2004]** and since then it has been subjected to many improvements throughout the years. It is rather a unique biotechnological process which consists of an aerated suspension of mixed bacterial cultures which carry out the biological conversion of the contaminants present in the wastewater. The aeration tank, while having many possible configurations, basically retains the influent wastewater for a number of hours (or days) in a well mixed/aerated environment, before forwarding the effluent for further settling to the secondary clarifier. The end products of the clarification process are clarified effluent that is discharged to the open water bodies and sludge. A fraction of the sludge is returned to the aeration tank and is called returned activated sludge. The sludge contains a high density of biomass and an active population of microorganisms is always maintained in the tank. The influent wastewater provides the basic food source for the microorganisms in the aeration tank. This biodegradable organic material is converted into new bacterial cells and other end products include CO_2, NO_3 and SO_4.

Typical features of the activated sludge include diverse substrate in terms of chemical composition and variety of particle sizes, multispecies biological culture, desirably growing in aggregates (flocs), widely fluctuating flows, temperatures, and changes in the influent wastewater concentration and composition, ability to metabolize a vast number of organic compounds and to oxidize/reduce/ polymerize etc. compounds containing nitrogen, phosphorus, sulfur and others, a variety of reactor configurations used e.g. completely stirred tanks, plug-flow, sequencing batch reactors, oxic, anoxic, and anaerobic selectors**[Eckenfelder and Grau, 1992]**

The activated sludge process exists in a large number of modifications and variations. Over last years the most important development in practice can be observed in industrial wastewater treatment, nutrient removal (N and P) and bulking control technologies.

Mechanism of Removal of organic material

When a biodegradable organic food source is supplied to a heterotrophic (utilizing organic matter for energy, as distinct from autotrophic which use CO_2 as the carbon source) microorganism population in a well-aerated environment the following phenomenon occur **[adapted from Kiely, 1997]:**

- The readily soluble biodegradable particulate COD goes through the cell wall and is metabolized quickly;
- The slowly biodegradable particulate COD is adsorbed on to the organisms and stored. This action removes all the particulate and colloidal COD which than over time is broken down and transferred through the cell wall and metabolized;
- Some of the COD metabolized is converted to new cells while the remainder is lost as heat in the energy process required for the new cell synthesis
- At the same time there is a net loss of biomass, termed endogenous mass loss, where some of the organisms utilize as food their own stored food materials and dead cells.

Growing cells utilize external substrate and external additional nutrients as required for growth and energy. The biochemical equation for bacterial cell respiration and synthesis in using organic matter as substrate in the activated sludge process is:

$$\text{Org.matter} + O_2 + \text{nutrients} \longrightarrow CO_2 + NH_3 + \text{new biomass} + \text{end products}$$

Organisms have developed a number of bio mechanisms enabling them to survive rather long periods without external substrate. Several pathways and bio mechanisms enabling them to survive have been recognized; some more may be still unknown **[Eckenfelder and Grau, 1992]**. First, bacteria can store products rich in both energy and nutrients. Second, some of the cell materials can serve several needs. They can be either utilized for cell growth, if the nutritional and environmental conditions are favourable, or they can be utilized as internal sources of energy (endogenous substrate) during starvation periods. These phenomenon are of an increased importance to wastewater treatment since they are employed in several successful modern technologies.

The activated sludge process is governed by the microorganism characteristics and the physical configuration of the aeration tank. As such, the biological kinetics and the process kinetics are closely interconnected. Process kinetics is based on the Monod equation and it has been described by many authors **[Metcalf and Eddy, 2004; Eckenfelder and Grau, 1992].**

Nitrification

The main objective of wastewater treatment is stabilization of the carbonaceous matter. However, the nutrients as nitrogen and phosphorus contribute greatly to eutrophication of the receiving wasters and many countries have passed legislation to remove those compounds from wastewater. Nitrogen in wastewater is generally in the forms of organic N and ammonia N, both in soluble and particulate forms; wastewater contains normally insignificant amounts of nitrite and nitrate N. Organic N and ammonia N are undesirable in wastewater effluents since they both exert a nitrogenous oxygen demand and ammonium N is also toxic to fish life. Nitrification is the biological process where ammonia serves as a substrate for nitrifying bacteria that oxidize ammonium to nitrite and nitrate in two steps as follows:

$NH_4 + 1.5\ O_2 \longrightarrow NO_2^- + H_2O + 2H+\ \ 250\ kJ$ (1)

$NO_2^- + 0.5O_2 \longrightarrow NO_3^- + 75\ kJ$ (2)

All acidity and most energy are produced in the first step, called nitritation. The second stage is called nitratation. Theoretical oxygen demand of nitrification is determined according to the equation:

$TOD_{nit} = 4.57$ g O_2 /g NH_4

Activated Sludge process is designed for BOD removal and Nitrification. The process designing of activated sludge process depends upon factors like Solid Retention Time (SRT), Temperature of the wastewater, Total kjehldahl Nitrogen (TKN), Ammoniacal Nitrogen (NH_4-N), Ultimate BOD entering the aeration tank, desired Mixed Liquor Suspended Solids (MLSS).

1.1.2. Units of Sewage Treatment Plant

The different units included in the design are inlet chamber, coarse screening, raw sewage sump, fine screening, grit chamber, distribution chamber before primary clarifier, primary clarifier, aeration tank, secondary clarifier, primary sludge sump, secondary sludge sump, sludge thickener, thickener overflow sump, thickened sludge sump, gas flaring system, gas generation set and centrifuge. All the units are discussed below:-

1.1.2.1. Inlet Chamber

Inlet chamber is designed to collect all the sewage coming from the different sewerage system. The inlet chamber contains raw sewage and from here the sewage is moved to screens or grit chamber.

1.1.2.2. Coarse Screening

Coarse screens remove large solids, rags, and debris from wastewater, and typically have openings of 6 mm (0.25 in) or larger. Types of coarse screens include mechanically and manually cleaned bar screens, including trash racks. [**Wastewater Technology Fact Sheet, EPA]**

Fig 1.1. Coarse Screen

1.1.2.3. Raw Sewage Sump

Wastewater from the coarse screen is collected in the sump from where it is pumped into the plant for further treatment.

1.1.2.4. Fine screening

Fine screens are typically used to remove material that may create operation and maintenance problems in downstream processes, particularly in systems that lack primary treatment. Typical opening sizes for fine screens are 1.5 to 6 mm (0.06 to 0.25 in). Very fine screens with openings of 0.2 to 1.5 mm (0.01 to 0.06 in) placed after coarse or fine screens can reduce suspended solids to levels near those achieved by primary clarification. Types of fine screens include mechanically and manually cleaned bar screens, including trash rack [**Wastewater Technology Fact Sheet, EPA]**

Fig 1.2. Fine Screen

1.1.2.5. Grit Chamber

Grit includes sand, gravel, cinder, or other heavy solid materials that are “heavier” (higher specific gravity) than the organic biodegradable solids in the wastewater. Grit also includes eggshells, bone chips, seeds, coffee grounds, and large organic particles, such as food waste. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in anaerobic digesters and aeration basins. Grit removal facilities typically precede primary clarification, and follow screening and comminution. This prevents large solids from interfering with grit handling equipment. In secondary treatment plants without primary clarification, grit removal should precede aeration [**Metcalf & Eddy, 2004]**

Many types of grit removal systems exist, including aerated grit chambers, vortex-type (paddle or jet induced vortex) grit removal systems, detritus tanks (short-term sedimentation basins), horizontal flow grit chambers (velocity-controlled channel), and hydro cyclones (cyclonic inertial separation). Various factors must be taken into consideration when selecting a grit removal process, including the quantity and characteristics of grit, potential adverse effects on downstream processes, head loss requirements, space requirements, removal efficiency, organic content, and cost. The type of grit removal system chosen for a specific facility should be the one that best balances these different considerations. The grit chamber designed in the process is square horizontal flow grit chamber.

1.1.2.6. Primary Clarifier

Primary treatment will remove the majority of the suspended solids that are present. This is another physical treatment process. Grit chambers sort out the heavy grit and primary clarifiers remove heavy organic solids in order to reduce the organic loading on the secondary processes. Primary clarifiers are used to slow the velocity of the water to a point where organic solids will settle to the bottom of the tank. This primary sludge is collected and sent to the solids handling processes. Primary clarifiers also contain equipment that is used to remove floating solids and greases from the surface. The solids that are present in the primary influent are classified as settleable and suspended solids. Suspended solids represent all of the remaining particles. Some of the solids are large enough and heavy enough that they will settle out very quickly. These solids are called settleable solids. The settleable solids represent about 30-60% of the total suspended solids. Primary clarifier removal efficiencies vary with changes in flow, temperature or solids loading.

Primary Clarifiers remove:

- 90-95% of the settleable solids
- 40-60% of the suspended solids
- 30-40% of the BOD_5

Fig 1.3. Primary Clarifier

1.1.2.7. Aeration Tank

An aeration tank is a place where a liquid is held in order to increase the amount of air within it. The most common uses of aeration tanks are in wastewater recovery, as the high oxygen levels will increase the speed at which the water is cleaned. The water is mixed with biological agents and then aerated. The increased oxygen promotes the growth of the beneficial biological material. That material will consume unwanted waste products held in the water. Finally, the beneficial material will grow due to the increased oxygen and food, which makes it easier to filter from the clean water.

Fig 1.4.Aeration Tank

1.1.2.8. Secondary Clarifier

The final or secondary clarifier is one of the most important unit processes and often determines the capacity of a treatment plant. The activated sludge system consists of two unit processes, the aeration basin and the final clarifier which is inseparable, with the performance of one closely linked to that of the other. The final clarifier must perform two primary functions namely Clarification and Thickening.

Clarification is the separation of solids from the liquid stream to produce a clarified effluent with low effluent suspended solids (ESS) levels. Thickening is the conveyance of sludge particles to the bottom of the tank, resulting in a slightly concentrated underflow, or return activated sludge (RAS).

Fig 1.5. Secondary Clarifier

1.1.3. Sludge Handling System

In the sewage sludge handling system there are two types of sludges:-

- Primary sludge – It is produced by solids settling the primary clarifier, characterised by high putrescibility and dewater ability when compared to biological sludge. Consistency of primary sludge is 2-6%.**[Turovskiy and mathai,2006]**
- Secondary Sludge – It is also known as biological sludge as it is produced from biological process (Activated Sludge Process) and contains microorganisms grown on organic matter. Consistency of secondary sludge is 0.5-1.5%.**[Turovskiy and mathai,2006]**

1.1.3.1. Sludge Thickener

The sludge is combined (Primary Sludge + Secondary Sludge) in the sludge thickener and it is thickened so that the efficiency of the digester can be increased.

1.1.3.2. Sludge Digester

High rate anaerobic sludge digester is been designed. When activated sludge is kept in an anaerobic environment, specialised bacteria will develop that use the excess sludge as a source of organic matter for fermentative metabolic processes. The end products of the fermentation are mainly methane and carbon dioxide.

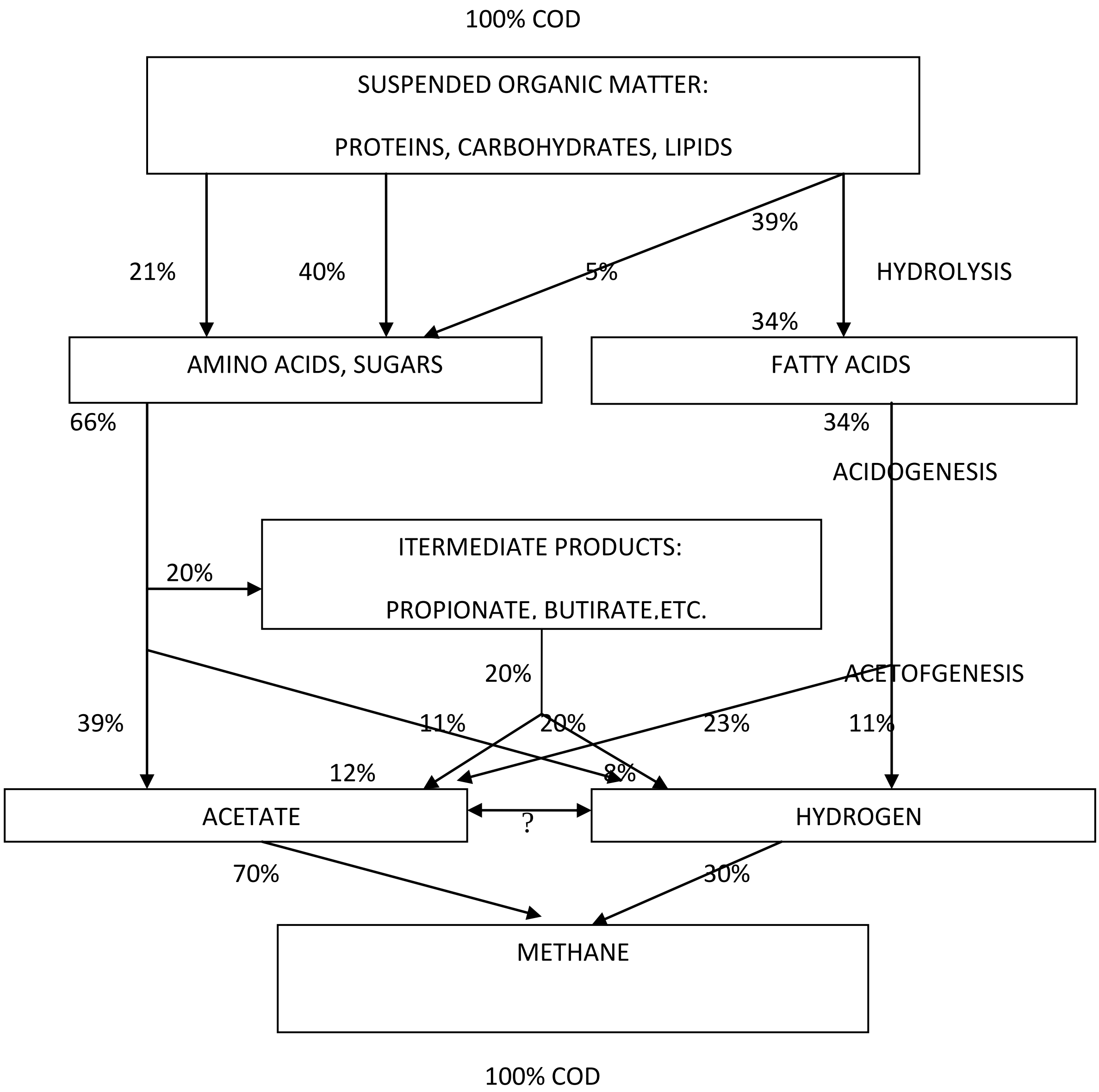

Fig 1.6. Anaerobic Stabilization Process Of Sludge

The overall conversion process of complex organic matter into methane and carbon dioxide can be divided into four steps namely hydrolysis, acidification, acetogenesis, methanogenesis. **[Gujer and Zehnder, 1983]**

Fig 1.7. High Rate Anaerobic Digester

1.1.3.3. Centrifuge Dewatering

A centrifuge is a device for separating particles from a solution according to their size, shape, density, viscosity of the medium and rotor speed. In a solution, particles whose density is higher than that of the solvent sink (sediment), and particles that are lighter than it float to the top. The greater the difference in density, the faster they move. If there is no difference in density (isopyknic conditions), the particles stay steady. To take advantage of even tiny differences in density to separate various particles in a solution, gravity can be replaced with the much more powerful "centrifugal force" provided by a centrifuge.

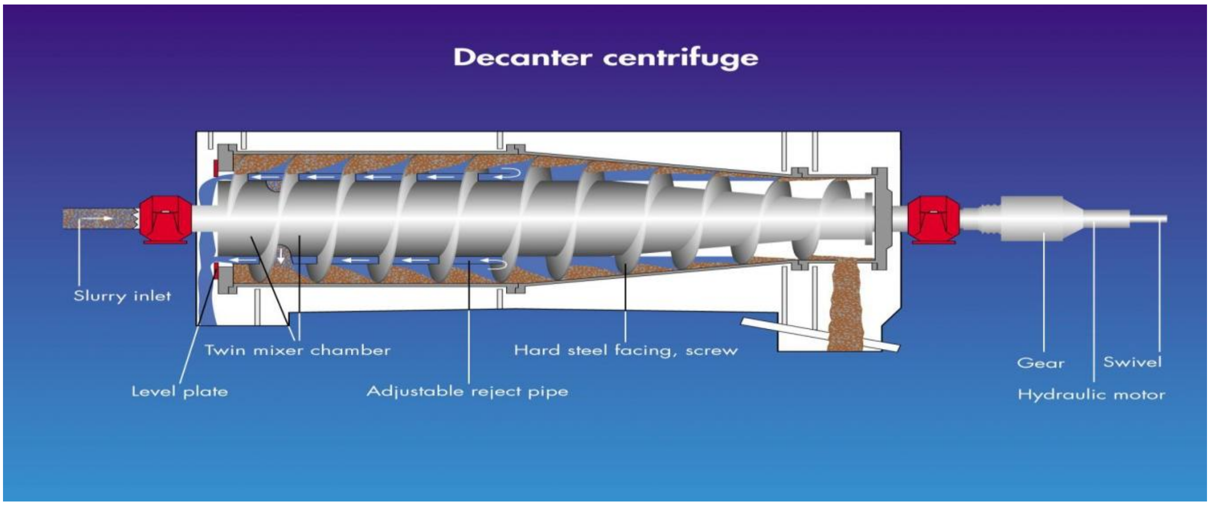

Fig 1.8. Decanter Centrifuge

1.1.4. Electricity Generation From Biogas

Gas generation set is designed to produce electricity from biogas. The main factor on which designing is emphasized is energy content of biogas.

CHAPTER 2

LITERATURE REVIEW

It is proposed in this work to study different units of Sewage treatment plant and understand their properties. This chapter presents a review of literature on various units of sewage treatment plants like clarifier, activated sludge process, sludge management and co generation of electricity from biogas. Literature review on modelling and simulation of sewage treatment plants using different technologies is also presented in this chapter. The idea of study carried in this area upto this stage for last twenty years is given in this chapter.

2.1. Clarifier

Carlos A.V. Costa et al. (2000) worked on conceptual design of industrial wastewater treatment processes: primary treatment. A design methodology was developed to support engineers in the conceptual design of wastewater treatment facilities and help them to improve creativeness and effectiveness. This hierarchical process design package could be described as an expert system coupled to a relational database and external programs integrated as a knowledge-based management system. This study described the proposed hierarchical procedure and the primary treatment selection prototype.

Orris E. Albertson (2008) reported work on solids loading limitations of rectangular secondary clarifiers and concluded that basin configuration and equipment design govern whether rectangular secondary clarifiers will experience problems of inadequate sludge transport capacity. The operating factors to be considered, other than peak flows which may be severe, are the potential for sludge bulking and the higher mixed liquor suspended solids concentrations and solids retention times employed for biological nutrient removal processes.

Athanasia M. Goula et al. (2008) studied on the effect of influent temperature variations in a sedimentation tank for potable water treatment. A computational fluid dynamics (CFD) model was used to assess the effect of influent temperature variation on solids settling in a sedimentation tank for potable water treatment. It is found that a temperature difference of only 1 ^{0}C between influent and tank content is enough to induce a density current. When the influent temperature rises, the tank exhibits a rising buoyant plume that changes the direction of the main circular current. This process keeps the particles in suspension and leads to a higher effluent suspended solids concentration, thus, worse settling. As the warmer water keeps coming in, the temperature differential decreases, the current starts going back to its original position, and, thus, the suspended solids concentration decreases.

2.2. Activated sludge Process

Yakir Hasit et al. (1983) did work on modelling of wastewater and sludge management systems and developed a program known as Optimal Sludge Management Program (OSMP). It is a computer program used to optimize the design and to simulate the steady-state operation of wastewater and sludge management systems. In its development, fundamental and rational process models were used, resorting to empiricism only when no rational models were available. Wastewater and sludge treatment processes were linked so that the impacts of recycle streams on processes were incorporated. The design and operation of water pollution control facilities were investigated by developing mathematical models for the performance of wastewater and sludge management processes in terms of basic design and operational variables, developing equations for the capital, energy, and other operation and maintenance costs in terms of basic design and operational variables, and using optimization and simulation procedures for examining the mathematical models to identify favourable design and operation conditions.

Robert B. Paterson et al. (1983) did work on Computer-aided Design and Control of an Activated Sludge Process The activated sludge process for wastewater treatment was studied by use of steady state and dynamic process models. Process constraints fix the steady state with only three designer-selected parameters: the percentage removal of suspended solids in the primary clarifier, the aeration basin suspended solids and the sludge age.

Sanders et al. (1994) reported work on computer modelling of single sludge systems for computer aided design and control of activated sludge process and presented initial work on the creation of new automatic CAD system for advanced effluent treatment processes. The computer model consisted of two interacting non linear equations which were linearized to give an approximate computer model. Optimal control theory was applied to the design of the controller for this system and a series of experiments examined the system performance using both the new computer model and new controller and concluded the biological process was successfully modelled on the microcomputer as single sludge process.

R. Ducato et al. (1995) studied dynamic simulation of a NDBEPR (nitrification denitrification biological excess phosphorus removal system) activated sludge process that achieved combined biological removal of nitrogen and phosphorus. The model includes the biological reactors and the secondary settler. A computer simulation programme was developed in order to test the model that was verified with reference to an existing urban wastewater treatment plant with a three-stage configuration (anaerobic, anoxic, aerobic reactors). the comparison between the simulation results and the actual plant recorded outputs shows that this model is able to well reproduce the dynamic behaviour of the plant.

Carmen Gabaldon et al. (1997) did work on software for the integrated design of wastewater treatment plants. A user-friendly environment has been implemented to facilitate design tasks, allowing rapid evaluation of different alternatives as well as performing sensitivity analyses. Flexible treatment plant configurations can be established with preliminary, primary, biological and tertiary wastewater treatments, and sludge treatment units. The name of the software is Datar. The software is made in the Microsoft Visual Basic 3.0 programming language to be a flexible and user-friendly tool. The software incorporates the main wastewater and sludge processes including nitrification– denitrification processes. Mathematical models describing treatment processes have been formulated taking into account the variation in waste quality parameters (BOD5, COD, TKN, TN, and TP including soluble fractions, and SS including the biodegradable fraction) for two different climatic conditions (winter and summer seasons) at each treatment element.

F. Nejjari et al. (1998) studied the on-line estimation and optimal control of a biological wastewater treatment process. The objective of the control was to force the residual substrate and the dissolved oxygen concentrations to track a given reference model despite the disturbances and system parameter uncertainties. The control law is based on one step ahead prediction of the controlled variables and minimization of an appropriate quadratic cost function. The estimated variables were used in the explicit design of the control algorithm according to certainty equivalence principle.

Guellil et al. (2001) investigated transfer of organic matter between wastewater and activated sludge flocs. The organic matter of wastewater was fractionated into settleable (i.e., particulate) and non- settleable (i.e., colloidal + soluble) fractions by settling followed by 0.22 μm filtration. Particulate, colloidal and soluble proportions were found to be relatively constant (45, 31 and 24% of the total COD, respectively). Transfer of soluble fraction always occurred from the wastewater to the activated sludge flocs, whereas bidirectional transfer occurred for the colloidal fraction. The transfer of soluble and colloidal matter reached a steady state after 40 min-mixing and 20 min-mixing, respectively.

Sotomayor et al. (2001) worked on Software sensor for on-line estimation of the microbial activity in activated sludge systems and biological activities of a colony of aerobic micro-organisms acting on activated sludge processes, where the carbonaceous waste degradation and nitrification processes are taken into account. These bioactivities are intimately related to the dissolved oxygen concentration. Two factors that affect the dynamics of the dissolved oxygen are the respiration rate or the oxygen uptake rate (OUR) and the oxygen transfer function K_la. In this work, OUR and the oxygen transfer function are estimated through a software sensor, which is based on a modified version of the discrete extended Kalman filter. Numerical simulations are carried out in a predenitrifying activated sludge process benchmark and the obtained results demonstrate the applicability and efficiency of the proposed methodology, which provided a valuable tool to supervise and control activated sludge processes.

Vanrolleghem et al. (2003) reported a comprehensive model calibration procedure for activated sludge models and made a methodology for calibration of the activated sludge plant models on the basis of consolidated engineering experience and a scientific approach. According to the method, the definition of the target(s) plays a crucial role in the selection of the steps incorporated in this so-called 'Biomath - Calibration' protocol. For the activated sludge modellers this protocol tries to combine and link the state of the art methodologies for calibration of different processes in a wastewater treatment plant (hydraulics, biological reactions, sedimentation processes, etc.).

Gernaey et al. (2003) did work on activated sludge wastewater treatment plant modelling and simulation. This paper introduced the nowadays most frequently used white-box models for description of biological nitrogen and phosphorus removal activated sludge processes. These models are mainly applicable to municipal wastewater systems, but could be adapted easily to specific situations such as the presence of industrial wastewater. Some of the main model assumptions are highlighted, and their implications for practical model application were discussed.

Alper Nuhoglu et al. (2004) worked on Mathematical modelling of the activated sludge process in Erzincan city where 124,000 population equivalents, are presented. The dynamic behaviour of simultaneous carbon removal and nitrification process carried out in an Carrousel type aerator and also the clarification-thickening function of final settler were modelled employing activated sludge model 1 (ASM1) and a dynamic model, respectively.

Miyata Jun et.al. (2004) reported wastewater treatment processing simulation technology using “activated sludge model” and that in developing design support software and operation support software for advanced wastewater treatment plants, JFE Engineering uses the “Activated Sludge Model” advocated by International Water Association (IWA).

Clara et al. (2005) investigated the solids retention time which is a suitable design parameter to evaluate the capacity of wastewater treatment plants to remove micropollutants. Critical SRTs were determined for different micro pollutants, indicating that the design criteria based on the sludge age allows an estimation of emissions. The results of the investigations lead to the conclusion that low effluent concentrations can be achieved in WWTPs operating SRTs higher than 10 days (referred to a temperature of 10 ^{0}C).

Holenda et al. (2007) worked on dissolved oxygen control of the activated sludge wastewater treatment process using model predictive control and concluded that activated sludge wastewater treatment processes were difficult to be controlled because of their complex and nonlinear behaviour, however, the control of the dissolved oxygen level in the reactors plays an important role in the operation of the facility. For this reason a new approach is studied in this paper using simulated case-study approach: model predictive control (MPC) had been applied to control the dissolved oxygen concentration in an aerobic reactor of a wastewater treatment plant.

Ujjaini et al. (2009) reported work on dynamic simulation of activated sludge using STOAT based wastewater treatment plant Titagarh, near Kolkata, India. Some alternative schemes were suggested. Different schemes were compared for the removal of Total Suspended Solids (TSS), b-COD, ammonia, nitrates etc. A combination of IAWQ1 module with the Takacs module gave best results for the existing scenarios of the Titagarh Sewage Treatment Plant. Work simulation was done with the help of STOAT, using the existing Scheme of Titagarh and the results were validated with the effluent data collected from the plant. The simulated data go well with the experimental data of the existing plant. Hence this validation was very significant.

Kumar Sundara, (2011) worked on computer aided design of waste water treatment plant with activated sludge process. In this work a computer program in C++ had been developed for comprehensive design of wastewater treatment plant which incorporates activated sludge process as biological treatment method. All the units of WWTP were included in the design and the program is developed in a very user friendly manner by referring various standard procedures and manuals. The validity of the software has been verified by test running and comparison with an existing plant data.

Marcus et al. (2011) reported an online method for estimation of degradable substrate and biomass in an aerated activated sludge process and studied the predenitrifying WWTP in Goteborghaving post nitrification in trickling filters and concluded that estimation does not work for an activated sludge process with aeration in one stirred tank alone, but when the activated sludge process could be described by at least two tanks in series, with oxygen measurements in each tank, the estimates converge.

A review of modelling approaches in activated sludge systems concluded that the feasibility of using models to understand processes, predict and/or simulate, control, monitor and optimize Wastewater Treatment Plants (WWTPs) had been explored by a number of researchers. Mathematical modelling provided a powerful tool for design, operational assistance, forecast future behaviour and control. A good model not only elucidated a better understanding of the complicated biological and chemical fundamentals but is also essential for process design, process start-up, dynamics predictions, process control and process optimization. (**Banadda et al., 2011)**

2.3. Sludge Management and Biogas Generation

Retter et al. (1993) reported work on solid-bowl centrifuges for wastewater sludge treatment and described rapid advances in centrifugal technology for dewatering and thickening of wastewater sludges, against the background of rapidly increasing sludge disposal costs. Examples were given of the key process design factors for wastewater sludge centrifuges, together with performance comparisons with conventional centrifuge technology. As a result of improved digestion, thickening of waste activated sludge can produce savings of 20-30% in the costs of disposal of dewatered digested sludge. Dewatering with advanced centrifuges were shown to have total operating costs for chemicals, personnel, power, capital servicing, transportation and landfill of the sludge at least 20% lower than with conventional technology.

Petrides et al. (1995) worked on EnviroCAD (A computer tool for analysis and evaluation of waste recovery, treatment and disposal processes). EnviroCAD runs on personal computers and assists scientists and engineers to simulate efficiently and analyze new integrated environmental processes and improve the performance of existing ones. In short, it helps in process optimization from an environmental standpoint. An important feature of the program is its ability to carry out material balances on individual compounds and track the fate of hazardous chemicals (e.g., chlorinated organic solvents, heavy metals, etc.) in integrated environmental process. This paper describes the architecture of EnviroCAD, its process modelling, economic evaluation, graphical user interface, and environmental assessment capabilities.

Maharaj et al. (2000) investigated the role of HRT and low temperature on the acid-phase anaerobic digestion of municipal and industrial wastewaters. Two identical completely mixed reactors with solids recycling capabilities were used to investigate the effects of hydraulic retention time (HRT) and low temperatures on volatile fatty acid (VFA) production. One reactor was fed with a 1:1 ratio of diluted primary sludge and a starch-rich industrial wastewater, while the other was fed with diluted primary sludge alone. The VFA and soluble COD concentrations and specific production rates reached their highest values at 30 h HRT and at 25°C. Further increase in HRT (at 25°C) or decrease in temperature (at an HRT of 30 h) resulted in lower amounts of VFA and COD produced.

Edgar Fernando Castillo M. et al. (2005) studied the operational conditions for anaerobic digestion of urban solid wastes and described an experimental evaluation of anaerobic digestion technology as an option for the management of organic solid waste in developing countries. As raw material, a real and heterogeneous organic waste from urban solid wastes was used. In the experimental phase, seed selection was achieved through an evaluation of three different anaerobic sludges coming from wastewater treatment plants. The methanization potential of these sludges was assessed in three different batch digesters of 500 mL, at two temperature levels. The results showed that by increasing the temperature to 15 ^{0}C above room temperature, the methane production increases to three times. So, the best results were obtained in the digester fed with a mixed sludge, working at mesophilic conditions (38–40 ^{0}C).

Hilkiah Igoni et al. (2007) investigated designs of anaerobic digesters for producing biogas from municipal solid-waste and concluded that the production of biogas was of growing interest as fossil-fuel reserves decline. However, there exists a dearth of literature on the design considerations that would lead to process optimization in the development of anaerobic digesters aimed at creating useful commodities from the ever-abundant municipal solid-waste. Consequently the study provided a synthesis of the key issues and analyses concerning the design of a high-performance anaerobic digester.

A review of the literature suggests that recent developments in sludge preconditioning technologies have substantially reduced the sludge residence time requirement to the order of 7 days. Also, the preconditioned sludges have been reported to hold potential for higher methane recovery with reduced excess sludge production requiring disposal. Such advantages, coupled with escalating fuel prices and the introduction of carbon credits under the Kyoto Accord, have significantly improved the economics of anaerobic digestion. As the cost of sludge management varies from one mill to another, mill-specific economic assessment of anaerobic digestion could identify cost-saving opportunities. **Allan Elliott et al.,(2007)**

Lise Appels et al. (2008) reviewed the principles of anaerobic digestion, the process parameters and their interaction, the design methods, the biogas utilisation, the possible problems and potential pro-active cures, and the recent developments to reduce the impact of the problems. After having reviewed the basic principles and techniques of the anaerobic digestion process, modelling concepts were assessed to delineate the dominant parameters.

Isil Toreci et al. (2008) investigated effect of microwave pretreatment (MW) high temperature (175 ^{0}C) and MW intensity to waste activated sludge digested with acclimatized inoculum in single- and dual-stage semi-continuous mesophilic anaerobic digesters at different sludge retention times (SRTs) (20, 10 and 5 days)**.** MW pretreatment led to similar sludge stabilization at low SRTs (5 and 10 days). Although lowering MW intensity slightly improved sludge solubilization, it had a negative effect on digestion at low SRTs. Single-stage digesters with MW pretreatment surpass dual-stage digesters performances.

A general overview of anaerobic digestion and the current status of biomethane technology on livestock farms in the United States was provided in this literature. It is part of the Bioenergy Engineering Education Program (BEEP) of the Biological Systems Engineering Department at Virginia Tech. Most of the discussion uses dairy manure as an example of feedstock for an anaerobic digester. **Jactone Arogo Ogejo et al., (2009)**

Puig et al. (2010) demonstrated that mass balance evaluation approach helps the WWTP engineers to distinguish and quantify between different strategies, where others could not. In the studied case, by-passing raw wastewater (27% of the influent flow) directly to the biological reactor did not improve the effluent quality and the nutrient removal efficiency of the WWTP. The increase of the influent C/N and C/P ratios was associated to particulate compounds with low COD/VSS ratio and a high non-biodegradable COD fraction.

Ivo Achu Nges et al. (2010) investigated effects of solid retention time on anaerobic digestion of dewatered-sewage sludge in mesophilic and thermophilic conditions and evaluated Anaerobic digestion of dewatered-sewage sludge using continuous stirred tank reactors (CSTRs) in duplicate under thermophilic (50 ^{0}C) and mesophilic (37 ^{0}C) conditions over a range of nine solid retention times (SRTs) and found that biogas production rate could be tripled when the SRT was shortened from 30 to 12 days and more than doubled from 35- to 15-day SRT because of a concomitant increase in OLR.

A review on the process of anaerobic digestion which is considered to be one of the most viable options for recycling the organic fraction of solid waste is provided. This manuscript provides a broad overview of the digestibility and energy production (biogas) yield of a range of substrates and the digester configurations that achieve these yields. The involvement of a diverse array of microorganisms and effects of co-substrates and environmental factors on the efficiency of the process has been comprehensively addressed. The recent literature indicates that anaerobic digestion could be an appealing option for converting raw solid organic wastes into useful products such as biogas and other energy-rich compounds, which may play a critical role in meeting the world's ever-increasing energy requirements in the future. **Khalid Azeem et al.,(2011)**

Nuno Miguel Gabriel Coelho et al. (2011) did work on evaluation of continuous mesophilic, thermophilic and temperature phased anaerobic digestion of microwaved activated sludge and studied the effects of microwave (MW) pretreatment, staging and digestion temperature on anaerobic digestion were investigated in a setup of ten reactors and found that the association of MW pretreatment and thermophilic operation improves dewaterability of digested sludge.

Joshua L. Rapport et al. (2011) worked on modelling the performance of the anaerobic phased solids digester system for biogas energy production. A process model was developed to predict the mass and energy balance for a full-scale (115 td^{-1}) high-solids anaerobic digester using research data from lab and pilot scale (3000 kg d^{-1} wet waste) systems and it was found that the digester system was financially viable whether producing electricity or CNG for discount rates of up to 13% y^{-1} without considering debt (all capital was considered equity), heat sales, feed-in tariffs or tax credits.

2.4. Power generation

M. Komiyama et al. (2006) investigated biogas as a reproducible energy source. Its steam reforming for electricity generation and for farm machine fuel. The chemical uniqueness of the biogas as a feedstock for steam reforming was identified, along with the difficulties to be overcome for its practical implementation were discussed.

Martina Poschl et al. (2010) worked on evaluation the energy efficiency of different biogas systems, including single and co-digestion of multiple feedstock, different biogas utilization pathways, and waste-stream management strategies concluded that the upgrading of biogas to biomethane for injection into natural gas network potentially increased the primary energy input for biogas utilization by up to 100%; the energy efficiency of the biogas system improved by up to 65% when natural gas was substituted instead of electricity.

Mikael Lantz (2012) investigated the economic performance of combined heat and power from biogas produced from manure in Sweden and evaluated the economic feasibility of various technologies, also on different scales, for the production of combined heat and power from manure-based biogas in Sweden is presented. The overall conclusion was that such production was not profitable under current conditions.

Caceres et al. (2012) did work on Biogas production from grape pomace. A thermodynamic model of the process and dynamic model of the power generation system was developed. The aim of this research was to develop the thermodynamic equilibrium analysis of grape pomace anaerobic digestion based on the equilibrium constants for predicting the potential production of biogas and its composition. In addition, a dynamic model of a biogas-fuelled microturbine system for distributed generation applications was derived.

CHAPTER 3

METHODOLOGY

Sewage treatment plants are designed to convert a raw sewage into an acceptable final effluent and to dispose of the solids removed in the process. Activated sludge process is the most commonly used biological treatment method for treating municipal wastewaters of large cities. The entire treatment process depends on physical as well as biological principles and no chemical additions are provided to protect the ecosystems that receive the treated effluents. In the present study a comprehensive ASP.NET program has been developed for the design of the following units as they are commonly used in the field of wastewater treatment:-

1. Screen Chamber
2. Grit Chamber
3. Primary Clarifier
4. Activated Sludge Process (Aeration Tank)
5. Secondary Clarifier
6. Sludge Handling System
7. Electricity Generation from Biogas

Other units designed are primary sludge sump, secondary sludge sump, overflow from thickener sump, sludge sump after thickener, centrifuge feed sump, biogas holder and gas flaring system. Pumps designed are return sludge pump, thickener overflow pump, sludge digester feed pump, biogas feed air blowers and centrifuge feed pump.

The various principles and rational, scientific as well as empirical formulae used in the design of the above treatment units are derived from standard references, hand books and manuals. To improve the suitability of the software for various field conditions and limitations, various constraints and compatibility range values are incorporated. The various parameters used in process designing are BOD_5, TSS (Total Suspended Solids), VSS (Volatile Suspended Solids), NH_4-N (Ammonical Nitrogen), NO_3-N (Nitrate Nitrogen), temperature and elevation from mean sea level.

3.1. Methodology of Process Designing of different Units

The methodology of process designing of different units is discussed below:-

3.1.1. Inlet Chamber

The main factor affecting its design is hydraulic retention time (HRT). The hydraulic retention time is the time for which the wastewater stays in the chamber. The inlet chamber designed is a rectangular inlet chamber.

3.1.2. Coarse Screening

The designing of coarse screening depends upon design flow passing through the screens and maximum allowable velocity of the wastewater passing through the screens. The factors like thickness of the bars and space between the bars are considered to find out number of bars in the screen.[**Wastewater Technology Fact Sheet, EPA]**

3.1.3. Raw Sewage Sump

The main factor affecting its design is hydraulic retention time (HRT). The sump designed is circular in shape.

3.1.4. Fine Screening

The designing of fine screening depends upon design flow passing through the screens and maximum allowable velocity of the wastewater passing through the screens. The factors like thickness of the bars and space between the bars are considered to find out number of bars in the screen. [**Wastewater Technology Fact Sheet, EPA]**

3.1.5. Grit Chamber

The factors on which the process designing of grit chamber is done are design flow, size of the grit to be removed, and specific gravity of the grit to be removed. Reynolds number is also used to find the type of the flow and stokes law is used.

3.1.6. Primary Clarifier

The primary clarifier used in the design is circular clarifier. The design mainly depends upon factors like design flow and solid loading rate. The BOD removal efficiency and TSS removal efficiency depends upon HRT (Hydraulic Retention Time) of the wastewater. The removal efficiency of VSS (Volatile Suspended Solids) is equal to BOD removal efficiency.

3.1.7. Aeration Tank

The aeration tank designed is rectangular in shape. Aeration is provided in the tank so that activated sludge process takes place properly. The oxygen is required both for removal of organic matter and nitrification process. The oxygen is provided from the air and so it depends on efficiency of the aerator to supply oxygen.

3.1.8. Secondary Clarifier

The secondary clarifier designed is circular in shape. It is designed on the factors like TSS concentration in the sludge, RAS (Return Activated Sludge), Surface overflow rate, Solid loading rate.

3.1.9. Sludge Thickener

The Sludge thickener designed is circular in shape. The designing is based on factors like specific gravity of the solids, consistency of sludge, solid loading rate and the combined sludge flow.

3.1.10. Sludge Digester

The process designing depends upon factors like SRT, VSS in the digesters, operating temperature of the digester, degree of stabilization of VSS. Biogas is produced in the process and its production depends upon volatile suspended solids reduced. The designing of the digester depends upon the flow of sludge entering the digester from the thickener.

3.1.11. Centrifuge Dewatering

The process designing depends upon factors like SRT, VSS in the digesters, operating temperature of the digester, degree of stabilization of VSS. Biogas is produced in the process and its production depends upon volatile suspended solids reduced. The designing of the digester depends upon the flow of sludge entering the digester from the thickener.

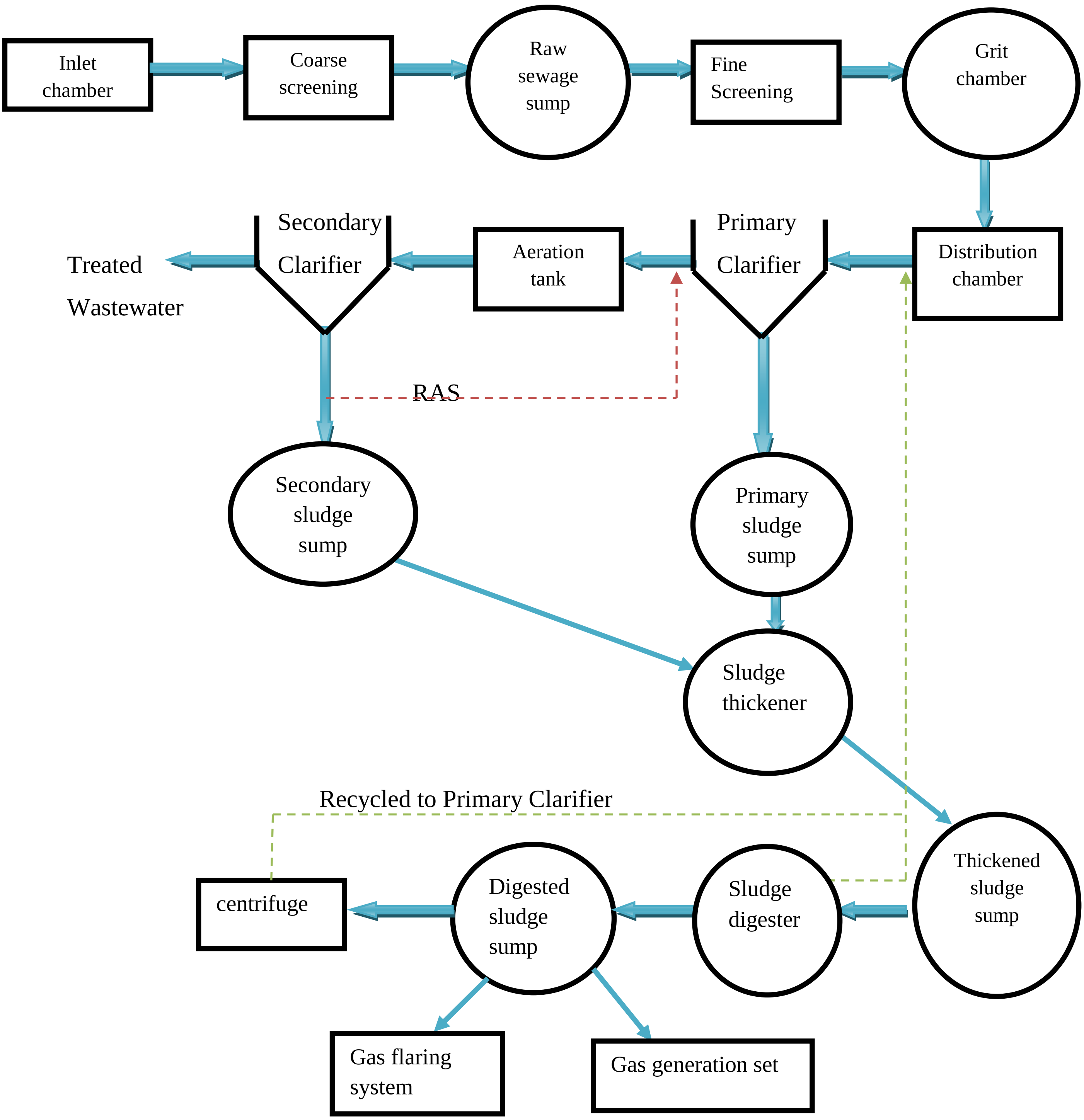

Fig 3.1. Flow diagram of the units designed

3.2. Program Development

The advantage of computer aided design of sewage treatment plant is an easy way to repeat the design calculations with different sets of input data and optimal size of the system may be obtained. Using ASP.NET language, a computer program has been developed for the comprehensive design of wastewater treatment plant. The flow chart of the program is shown in **Fig 3.2.** The entire program has been written comprising of functions, with object oriented programming (OOPs) concept. Object oriented programming allows one to write programs in a much more rational manner than procedural oriented programming.**ASP.NET 3.5 and Visual Studio 2008** bring great new functionality around web development and design that makes building standards based, next generation web sites easier than ever. From the inclusion of ASP.NET AJAX into the runtime, to new controls, the new LINQ data capabilities, to improved support for CSS, JavaScript and others, Web development has taken a significant step forward.

3.2.1. New Features in ASP.NET 3.5

ASP.NET AJAX

With ASP.NET AJAX, developers can quickly create pages with sophisticated, responsive user interfaces and more efficient client-server communication by simply adding a few server controls to their pages. Previously an extension to the ASP.NET runtime, ASP.NET AJAX is now built into the platform and makes the complicated task of building cross-platform, standards based AJAX applications easy.

New List View and Data Pager Controls

The new List View control gives unprecedented flexibility in display of data, by allowing to have complete control over the HTML mark up generated. List View‘s template approach to representing data is designed to easily work with CSS styles, which comes in handy with the new Visual Studio 2008 designer view. In addition, one can use the Data Pager control to handle all the work of allowing users to page through large numbers of records.

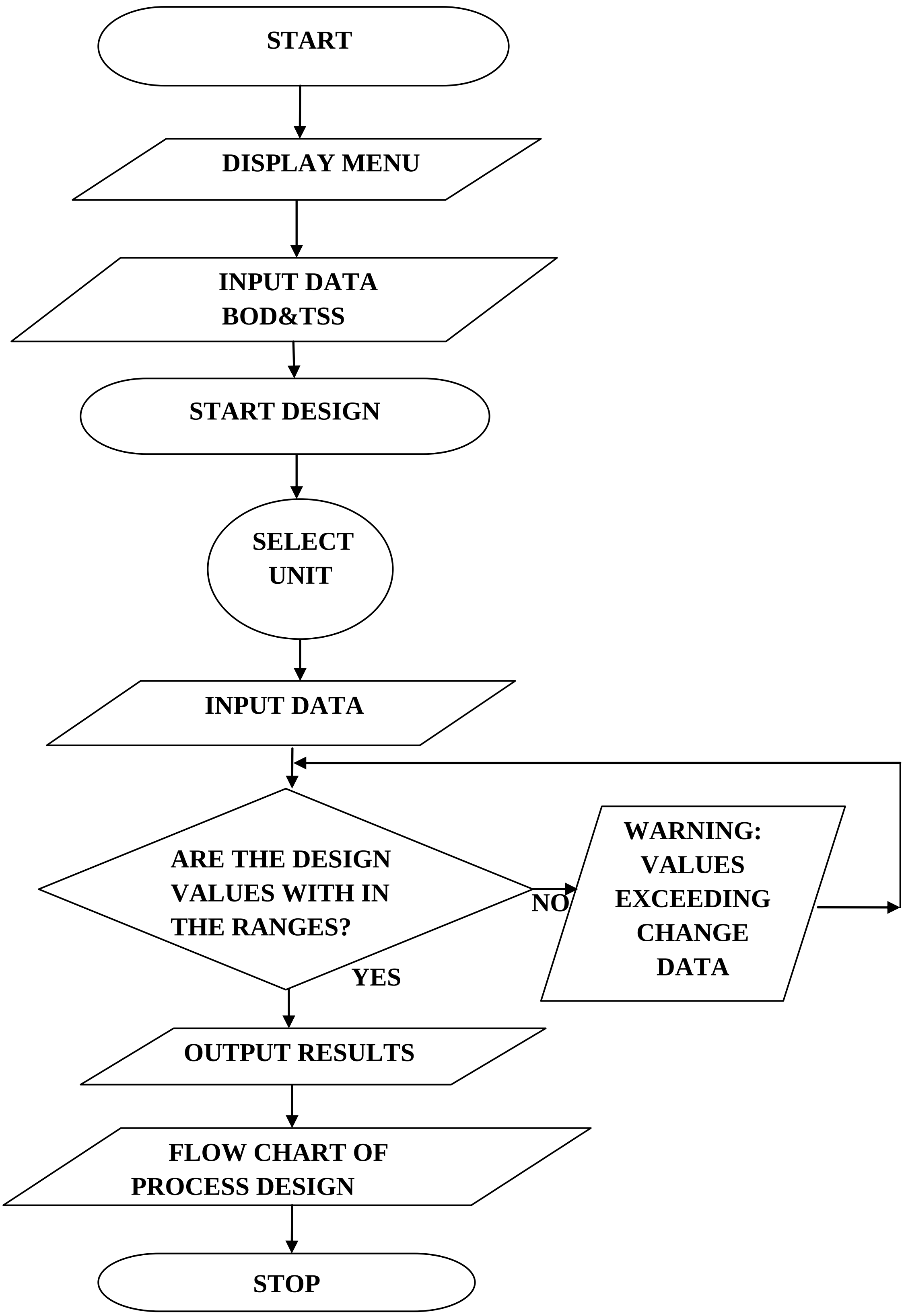

Fig 3.2. Flow Chart of the Program Developed for Design Of STP

3.2.2. LINQ and other .NET Framework 3.5 Improvements

With the addition of Language Integrated Query (LINQ) in .NET Framework 3.5, the process of building SQL queries using error-prone string manipulation is a thing of the past. LINQ makes relational data queries a first-class language construct in C++ and Visual Basic, complete with compiler and Intel license support. For web applications, the ASP.NET LINQ Data Source control allows to easily use LINQ to filter, order and group data that can then be bound to any of the data visualization controls like the List View and Grid View controls. In addition, all the other improvements to .NET Framework 3.5, including the new Hash Set collection, Date Time offset support, diagnostics, garbage collection, better thread lock support, and more, are all available in ASP.NET applications.

The main features of the software are as follows:-

- The software is completely user friendly.
- The software works online so is accessible anywhere in the world without any installations in the computer.
- Menu is displayed to select a particular unit for design.
- Design procedures followed are according to standard practice and field oriented.
- Permissible ranges of the parameters are provided to guide the user for entering the input data.
- A warning message is displayed when value of any parameter entered yields a design value, which exceeds or falls short of the expected range usually practiced and, also an option to modify that particular parameter until a satisfactory design is obtained.
- The program is written in a user friendly environment, and supports necessary information for design of the units.
- The software will not allow entering any data which is incompatible and prevents from obtaining erroneous results.

CHAPTER 4
CALCULATIONS

Overview

The treatment of the sewage depends upon the influent entering the plant and effluent going out of the plant and these factors also decide the degree of treatment required by the sewage to achieve the effluent quality levels. The influent and effluent quality levels are measured by certain parameters such as pH, TSS, BOD_5, COD, VSS, TKN, NH_4 – N, Influent Alkalinity as $CaCO_3$. The treatment also depends upon many factors such as temperature of sewage, elevation from mean sea depth, Average flow rate , peak flow rate, HRT, SRT, etc.

4.1. Raw Sewage characteristics

For the Process designing of sewage treatment plant by ASP Technology it is necessary to find out influent characteristics of influent wastewater such as pH, TSS, BOD_5, COD, Temperature, VSS, TKN, NH_4 – N, Influent Alkalinity as $CaCO_3$, Elevation from mean sea depth, Average flowrate. The different parameters alongwith their units are shown **Table 4.1** below:-

Table 4.1 Raw sewage characteristics

PARAMETERS	UNITS
pH	
TSS	mg/l
BOD_5	mg/l
COD	mg/l
TOTAL COLIFORM	MPN/100 ml
FECAL COLIFORM	MPN/100 ml
TEMPERATURE	^0C

VSS	mg/l
TKN	mg/l
NH_4-N	mg/l
INFLUENT ALKANITY AS $CaCO_3$	mg/l
ELEVATION FROM MEAN SEA DEPTH	M
AVERAGE FLOW RATE	Mld

4.2. Treated Sewage Characteristics

Treated sewage quality parameters alongwith its units is shown in **Table 4.2**:-

Table 4.2 Treated sewage characteristics

PARAMETERS	UNITS
pH	
BOD_5	mg/l
TSS	mg/l
TKN	mg/l
NH_4-N	mg/l

After deciding the degree of treatment and effluent characteristics process designing of sewage treatment plant is carried out unit wise. The designing of various sequential units is discussed here under:-

4.3. Process Designing of Inlet Chamber

The first unit in the designing is **Inlet Chamber**. Inlet Chamber is the unit where the sewage coming from the sewerage system is first collected or stored. First of all ratio between peak flow and average flow is decided and on the basis of which peak flow is calculated.

Peak Flow = $(\text{Average Flow} \times \text{Ratio between Peak Flow and Average Flow})$ Mld

The designing of inlet chamber is done on the basis of peak flow. Then to calculate the size of the inlet chamber retention time of the sewage at peak flow in the inlet chamber is decided. Let the units of retention time be in seconds.

Then volume of inlet chamber is calculated by following formula:

Volume of inlet Chamber = $\left(\frac{\text{Peak Flow x Retention time of sewage at peak flow}}{24 \times 3600}\right)$ m^2

Then depth of the inlet chamber is decided. Let the units of depth of inlet chamber be in metres.

Area of inlet chamber is calculated = $\left(\frac{\text{Volume of Inlet Chamber}}{\text{Depth of Inlet Chamber}}\right)$ m^2

Then width of inlet chamber is decided. Let the width be in metres.

Length of inlet chamber is calculated = $\left(\frac{\text{Area of Inlet Chamber}}{\text{Width of Inlet Chamber}}\right)$ m

Length provided is little higher than the calculated length to provide all allowances.

Total volume of inlet chamber provided =

(Length of inlet chamber provided x width of inlet chamber x depth of inlet chamber) m^3

Free Board is also to be provided.

4.4. Process Designing of Coarse Screens

The coarse screening is of two types, mechanical screening and manual screening. Now days mechanical screens are mostly used and manual screens are used as standby. The calculations of both screens are discussed respectively:-

4.4.1. Process Designing of Mechanical screening

First step is to decide **number of screens** to be provided depending on the flow rate.

Peak Flow through each screen = $\left(\frac{\text{Peak Flow}}{\text{No of screens} \times 24 \times 3600}\right)$ m^3/sec

Maximum velocity through the screen at peak flow should be 1 m/s (assumed) and minimum velocity through the screen at average flow be 0.5 m/s (assumed).**[Metcalf Eddy, 2004]**

Clear area of opening through the screen at peak flow =

$$\left(\frac{\text{Peak Flow through each screen}}{\text{Maximum velocity through the screen at peak flow}}\right) m^2 \quad \text{(i)}$$

Average Flow through each screen = $\left(\frac{\text{Average Flow}}{\text{No of screens} \times 24 \times 3600}\right)$ m^3/sec

Clear area of opening through the screen at average flow

$$= \left(\frac{\text{Average Flow through each screen}}{\text{Minimum velocity through the screen at average flow}}\right) m^2 \quad \text{(ii)}$$

Clear area of opening through the screen considered = greater value among Eq (i) and Eq (ii)

Clear spacing between the bars ranges between 20 – 50 mm. **[Metcalf Eddy, 2004]**

Thickness of bars is usually taken 10 mm.

Gross area of screen (m^2) =

$$\left(\frac{\text{Clear area of opening through the screen considered} \times (\text{clear spacing between the bars} + \text{thickness of bars})}{\text{clear spacing between the bars}}\right)$$

Depth of the screen is usually kept 2/3rd of width of the screen

Width of the screen = $\left(\sqrt{\text{Gross area of screen} \times 1.5}\right)$ m

Depth of the screen = $\left(\frac{\text{Gross area of screen}}{\text{width of the screen}}\right)$ m

No. of bars = $\left(\frac{\text{width of the screen} \times 1000}{\text{Clear spacing between the bars} + \text{thickness of bars}}\right)$ (iii)

Free board as decided is also provided. Let it also be in metres. Usually its value is taken 0.3-0.8 m.

4.4.2. Process Designing of Manual Screening (Standby)

Usually 1 standby manual screen is provided along with mechanical screening.

Peak Flow through screen = Peak Flow through each screen in mechanical screening

Average Flow through screen = Average Flow through each screen in mechanical screening

Maximum velocity through the screen at peak flow should be 1 m/s (assumed) and minimum velocity through the screen at average flow be 0.5 m/s (assumed).

Clear area of opening through the screen at peak flow

$$= \left(\frac{\text{Peak Flow through each screen}}{\text{Maximum velocity through the screen at peak flow}}\right) m^2 \quad \text{(iv)}$$

Clear area of opening through the screen at average flow

$$= \left(\frac{\text{Average Flow through each screen}}{\text{Minimum velocity through the screen at average flow}}\right) m^2 \quad \text{(v)}$$

Clear area of opening through the screen considered = greater value among Eq (iv) and Eq (v).

Clear spacing between the bar ranges between 20 – 50 mm.

Thickness of bars is usually taken 10 mm.

Gross area of screen (m^2) =

$$\left(\frac{\text{Clear area of opening through the screen considered} \times (\text{clear spacing between the bars} + \text{thickness of bars})}{\text{clear spacing between the bars}}\right)$$

Depth of the screen is usually kept 2/3rd of the width of the screen

Width of the screen = $\left(\sqrt{\text{Gross area of screen} \times 1.5}\right)$ m

Depth of the screen = $\left(\frac{\text{Gross area of screen}}{\text{width of the screen}}\right)$m

No.of bars = $\left(\frac{\text{width of the screen} \times 1000}{\text{Clear spacing between the bars} + \text{thickness of bars}}\right)$ (vi)

Free board as decided is also provided. Let it also be in metres. Usually its value is taken 0.3-0.8 m.

4.5. Process Designing of Raw Sewage Sump

Decide the number of units of raw sewage sump to be provided. Hydraulic retention time (HRT in minutes) of sewage at peak flow in raw sewage sump should be decided.

Volume of each Raw Sewage Sump

$$= \left(\frac{\text{Peak Flow} \times 1000 \times \text{HRT}}{\text{No.of units of Raw Sewage Sump} \times 24 \times 60}\right) m^3$$

Provide side water depth (SWD in metres).

Area of each Raw Sewage Sump required

$$= \left(\frac{\text{Volume of each Raw Sewage Sump}}{\text{Side Water Depth}}\right) m^2$$

Diameter of each sump required

$$= \left(\sqrt{\frac{\text{Area of each Raw Sewage Sump required} \times 4}{\pi}}\right) m$$

Decide diameter of each sump to be provided. Usually value little greater than diameter of each sump required is provided.

Volume of each Sump provided $= \left(\frac{\pi \times (\text{Diameter of each Sump provided})^2}{4}\right) m^3$

Free board as decided is also provided.

4.6. Process Designing of Fine Screens

The fine screening is of two types, mechanical screening and manual screening. Nowdays mechanical screens are mostly used and manual screens are used as standby. The calculations of both screens are discussed respectively:-

4.6.1. Process Designing of Mechanical screening

First step is to decide **number of screens** to be provided depending on the flow rate.

Peak Flow through each screen = $\left(\frac{\text{Peak Flow}}{\text{No of screens} \times 24 \times 3600}\right)$ m^3/sec

Maximum velocity through the screen at peak flow should be 1 m/s (assumed) and minimum velocity through the screen at average flow be 0.5 m/s (assumed).

Clear area of opening through the screen at peak flow

= $\left(\frac{\text{Peak Flow through each screen}}{\text{Maximum velocity through the screen at peak flow}}\right)$ m^2 (vii)

Average Flow through each screen = $\left(\frac{\text{Average Flow}}{\text{No of screens} \times 24 \times 3600}\right)$ m^3/sec

Clear area of opening through the screen at average flow

= $\left(\frac{\text{Average Flow through each screen}}{\text{Minimum velocity through the screen at average flow}}\right)$ m^2 (viii)

Clear area of opening through the screen considered = greater value among Eq (vii) and Eq (viii)

Clear spacing between the bars ranges between 6 – 20 mm.

Thickness of bars usually considered 2 mm.

Gross area of screen =

$\left(\frac{\text{Clear area of opening through the screen considered} \times (\text{clear spacing between the bars} + \text{thickness of bars})}{\text{clear spacing between the bars}}\right)$ m^2

Depth of the screen = 2/3 of width of the screen **(assumption)**

Width of the screen = $\left(\sqrt{\text{Gross area of screen} \times 1.5}\right)$ m

Depth of the screen = $\left(\frac{\text{Gross area of screen}}{\text{width of the screen}}\right)$ m

No of bars = $\left(\frac{\text{width of the screen} \times 1000}{\text{Clear spacing between the bars} + \text{thickness of bars}}\right)$ (ix)

Free board as decided is also provided. Let it also be in metres. Usually its value is taken 0.3-0.8 m.

4.6.2. Process Designing of Manual Screening (Standby)

Usually 1 standby manual screen is provided along with mechanical screening.

Peak Flow through screen = Peak flow through each screen in mechanical screening

Average Flow through screen = Average Flow through each screen in mechanical screening

Maximum velocity through the screen at peak flow should be 1 m/s (assumed) and minimum velocity through the screen at average flow be 0.5 m/s (assumed).

Clear area of opening through the screen at peak flow

$$= \left(\frac{\text{Peak Flow through each screen}}{\text{Maximum velocity through the screen at peak flow}}\right) \text{ m}^2 \qquad \text{(x)}$$

Clear area of opening through the screen at average flow

$$= \left(\frac{\text{Average Flow through each screen}}{\text{Minimum velocity through the screen at average flow}}\right) \text{ m}^2 \qquad \text{(xi)}$$

Clear area of opening through the screen considered = greater value among Eq (x) and Eq (xi)

Clear spacing between the bars ranges between 6 – 20 mm.

Thickness of bars is usually taken 2 mm.

Gross area of screen =

$$\left(\frac{\text{Clear area of opening through the screen considered } \times (\text{clear spacing between the bars} + \text{thickness of bars})}{\text{clear spacing between the bars}}\right)$$

Depth of the screen is usually kept 2/3rd of width of the screen

Width of the screen = $\left(\sqrt{\text{Gross area of screen} \times 1.5}\right)$ m

Depth of the screen = $\left(\frac{\text{Gross area of screen}}{\text{width of the screen}}\right)$ m

No. of bars = $\left(\frac{\text{width of the screen} \times 1000}{\text{Clear spacing between the bars} + \text{thickness of bars}}\right)$ (xii)

Free board as decided is also provided.

4.7. Process Designing of Grit Chamber

Decide the number of units of grit chamber to be provided. This depends on the flow rate of the sewage.

Design Flow through each Grit Chamber =

$$\left(\frac{\text{Peak Flow}}{\text{No.of units of Grit Chamber Provided}}\right)\text{Mld}$$

Decide the size of the grit to be removed. Let it be in millimetres.

Specific Gravity of the grit (S_g).generally its value is taken 2.65.

Applying Stokes Law

Viscosity (ν) = ν

$$\textbf{Settling Velocity} = \left(\frac{9.81 \times (\text{Sp.Gravity of the grit} - 1)\left(\frac{\text{size of the grit to be removed}}{1000}\right)^2}{18 \times \nu}\right) \text{m/s}$$

$$\textbf{Reynolds Number (R)} \quad = \quad \left(\frac{\text{Settling Velocity} \times \text{Sp.Gravity of the grit}}{1000 \times \nu}\right)$$

$$\textbf{Settling Velocity}_{\mathbf{p}} \quad = \left(\frac{4 \times 9.81 \times \text{size of the grit to be removed} \times (Sg - 1)}{\sqrt{\left(3 \times \left(\left(\frac{24}{R}\right) + \left(\frac{3}{R}\right) + 0.34\right)\right)}}\right) \text{m/s}$$

Decide the desired efficiency of removal of grit particles (η). It is expressed in %.

Select basin performance.

Surface Overflow rate =

$$\left(\frac{\text{Settling Velocity p} \times \text{basin performance} \times 24 \times 3600}{\left(1 - \left(\frac{\eta}{100}\right)\right)^{-0.125} - 1}\right) \text{m}^3/\text{m}^2/\text{day}$$

$$\textbf{Area of each grit chamber} = \left(\frac{\text{Design Flow through each Grit Chamber} \times 1000}{\text{Surface Overflow rate}}\right) \text{m}^2$$

Size of each square basin = $\left(\sqrt{\text{Area of each grit chamber}}\right)$ m

Provide side water depth (SWD in metres).

Free board as decided is also provided.

4.8. Process Designing of Distribution Chamber before Primary Clarifier

Decide retention time (in seconds) at peak Flow.

Volume required = $\left(\frac{\text{Peak Flow} \times \text{Retention time} \times 1000}{24 \times 3600}\right)$ m^3

Decide Side Water Depth (SWD). Let it be in metres.

Area required = $\left(\frac{\text{Volume Required}}{\text{SWD}}\right)$ m^2

Decide Width (W) to be provided. Let it be in metres.

Length required = $\frac{\text{Area Required}}{\text{Width}}$ m

4.9. Process Designing of Primary Clarifier

Decide Number of Clarifiers to be provided.

Average Flow in each Clarifier = $\frac{\text{Average Flow}}{\text{Number of Clarifiers}}$ Mld

Peak Flow in each Clarifier = $\frac{\text{Peak Flow}}{\text{Number of Clarifiers}}$ Mld

Table 4.3 Type of settling, overflow rate, depth and detention time

Type of settling	Overflow rate, $m^3/m^2/d$		Depth , m	Detention time , hr
	average	peak		

A. Primary settling				
Primary settling only	25 – 30	50 – 60	2.5 – 3.5	2 – 2.5
Primary settling followed by secondary treatment	35 – 50	80 – 120	2.5 – 3.5	
B. Secondary Settling				
Secondary settling for trickling filter	15 – 25	40 – 50	2.5 – 3.5	1.5 – 2
Secondary settling for activated sludge aeration	15 – 35	40 – 50	3.5 – 4.5	
Secondary settling for extended aeration	8 – 15	25 – 35	3.5 – 4.5	

[**Source :Manual on Sewerage And Sewage Treatment CPHEEO]**

Decide Surface loading at peak flow that ranges from 80 – 120 m^3/m^2/day **(Table 4.3)**

Decide Surface loading at average flow that ranges from 35– 50 m^3/m^2/day **(Table 4.3)**

Area of each Clarifier required at average flow

$$= \left(\frac{\text{Average Flow in each Clarifier} \times 1000}{\text{Surface loading at average flow}}\right) m^2 \qquad \text{(xiii)}$$

Area of each Clarifier required at Peak flow

$$= \left(\frac{\text{Peak Flow in each Clarifier} \times 1000}{\text{Surface loading at Peak flow}}\right) m^2 \qquad \text{(xiv)}$$

Area of each Clarifier Required = Greater Value between (xiii) and (xiv)

Diameter of each Clarifier required = $\left(\sqrt{\frac{\text{Area of each Clarifier Required} \times 4}{\pi}}\right)$ m

Diameter of each Clarifier to be provided (D) is decided. It is usually kept little more than required Diameter of the clarifier.

Provide side water depth (SWD in metres).

Volume Provided = $\left(\frac{\pi \times (D)^2 \times SWD}{4}\right)$ m^3

Hydraulic Retention Time (HRT) = $\left(\frac{\text{Volume Provided} \times 24}{\text{Average Flow in each Clarifier} \times 1000}\right)$ hrs

BOD removal efficiency = $\left(\frac{HRT}{0.018 + (0.02 \times HRT)}\right)$ %.

It ranges from 30 – 45 % **[Manual on Sewerage And Sewage Treatment CPHEEO].**

TSS removal efficiency = $\left(\frac{HRT}{0.0075 + (0.014 \times HRT)}\right)$ %.

It ranges from 45 – 60 % **[Manual On Sewerage And Sewage Treatment CPHEEO].**

Effluent BOD concentration from Primary Clarifier

$$= \left(\text{Influent BOD} \times \left(\frac{100 - \text{BOD removal efficiency}}{100}\right)\right) \text{mg/l}$$

Effluent TSS concentration from Primary Clarifier

$$= \left(\text{Influent TSS} \times \left(\frac{100 - \text{TSS removal efficiency}}{100}\right)\right) \text{mg/l}$$

4.10 Process Designing of Aeration Tank

Table 4.4 gives the following values of $\mu_{N,m}$, K_n, k_{dn}, K_o:-

$\mu_{N,m} = (0.75 \times (1.07)^{T-20})$ g/g.d.

$K_n = (0.74 \times (1.053)^{T-20})$ g/m^3.

$k_{dn} = (0.08 \times (1.04)^{T-20})$ g/g.d.

DO = 2 g/m^3

K_o = 0.5 g/m^3.

Let Treated NH_4-N = N mg/l

$$\mu_n = \left(\left(\left(\frac{(\mu N,m \times N)}{Kn + N}\right) \times \left(\frac{DO}{DO + Ko}\right)\right) - kdn\right) \text{ g/g.d}$$

Table 4.4 Typical Values of constants used in activated sludge process

Coefficient	Unit	Range	Typical value
$\mu_{N,m}$	g VSS/ g VSS.d	0.20 – 0.90	0.75
K_n	g NH_4-N/m^3	0.5 – 1.0	0.74
Y_n	g VSS/g NH_4	0.1 – 0.15	0.12
k_{dn}	g VSS/ g VSS.d	0.05 – 0.15	0.08
K_o	g/m^3	0.40 – 0.60	0.50
μ_m	g VSS/ g VSS.d	3.13.2	6
K_s	g bCOD/m^3	5 – 40	20
Y	g VSS/ g bCOD	0.30 – 0.50	0.40
k_d	g VSS/ g VSS.d	0.06 – 0.20	0.12
f_d	Unitless	0.08 – 0.20	0.15

[Source: Metcalf Eddy, 2004]

Solid Retention Time (SRT) = $\frac{1}{\mu n}$ days

Design SRT = $(1.5 \times SRT)$ days

Determining the Biomass Production

Table 4.4 gives values of following values Y, $K_{d,}$ $\mu_{m,}$ $K_{s,}$ $Y_{n,}$ f_d :-

Y = 0.4 gVSS/g bCOD.

S_0 = $(1.6 \times \text{Effluent BOD concentration from Primary Clarifier})$ mg/l

$K_d = 0.12 \times (1.04)^{T-20}$ g/g.d.

$\mu_m = 6 \times (1.07)^{T-20}$ g/g.d.

$K_s = 20$ g/m^3

$$S = \left(\frac{Ks \times (1 + (Kd \times \text{Design SRT})}{(\text{Design SRT} \times (\mu m - Kd)) - 1}\right) \text{ mg/l}$$

$Y_n = 0.12$ g VSS/ g NO_x.

$NO_x = (0.8 \times \text{TKN})$ mg/l (assuming 80 % of TKN)

$f_d = 0.15$ g/g.

$P_{x,bio}$=

$$\left(\left(\frac{\text{Average Flow Rate} \times Y \times (S0 - S)}{1 + (Kd \times \text{Design SRT})}\right)\left(\frac{fd \times Kd \times \text{Average Flow Rate} \times Y \times (S0 - S) \times \text{Design SRT}}{1 + (Kd \times \text{Design SRT})}\right) \cdot \left(\frac{\text{Average Flow Rate} \times Yn \times NOx}{1 + (kdn \times \text{Design SRT})}\right)\right)$$

kg/d

Amount of NO_x oxidised to nitrate

$$= \left(\text{Influent TKN} - N - \left(\frac{0.12 \times Px,bio}{\text{Average Flow Rate}}\right)\right) \text{ mg/l}$$

VSS $= \text{Influent VSS} \times \left(\frac{100 - \text{BOD removal efficiency}}{100}\right)$ mg/l

nbVSS $= (0.25 \times \text{VSS})$ mg/l. Usually 25 – 35 % of VSS

$P_{x,vss}$ $= (Px, bio \times (nbVSS \times \text{Average Flow Rate}))$ kg/d

iTSS $= (\text{Effluent TSS concentration from Primary Clarifier} - \text{VSS})$ mg/l

$P_{x,TSS}$ $= \left(\left(\frac{Px,vss}{0.85}\right) + (iTSS \times \text{Average Flow Rate})\right)$ kg/d

Mass of VSS $= (Px, vss \times \text{Desired SRT})$ kg

Mass of TSS $= (Px, tss \times \text{Desired SRT})$ kg

Decide Desired MLSS (in g/m^3).

Volume of Aeration Tank = $\left(\frac{1000\times \text{Mass of TSS}}{\text{Desired MLSS}}\right)$ m^3

HRT = $\left(\frac{24 \times \text{Volume of Aeration Tank}}{\text{Average Flow Rate}\times 1000}\right)$ hr

MLVSS/MLSS = $\frac{\text{Mass of VSS}}{\text{Mass of TSS}}$

MLVSS concentration in aeration tank = $\left(\frac{\text{MLVSS}}{\text{MLSS}}\times \text{Desired MLSS}\right)$ g/m^3

F/M = $\left(\frac{\text{Average Flow Rate}\times\text{Effluent BOD from primary Clarifier}\times 1000}{\text{MLVSS}\times\text{Volume of Aeration Tank}}\right)$ kg/kg.day

BOD loading = $\left(\frac{\text{Average Flow Rate}\times\text{Effluent BOD from primary Clarifier}}{\text{Volume of Aeration Tank}}\right)$ kg/m^3.day

Alkalinity needed as $CaCO_3$ = $(7.14 \times \text{Amount of Nox oxidized to nitrate})$ mg/l

Decide Residual alkalinity as $CaCO_3$ to keep pH in range 6.8-7.0. It is expressed in mg/l.

Alkalinity needed to be added ($CaCO_3$)

$$= \left(\begin{array}{c}\text{Residual alkanity} + \text{Residual alkanity} \\ - \text{Influent Alkanity}\end{array}\right) \text{mg/l}$$

Total Alkalinity needed as $CaCO_3$

$$= (\text{Alkanity needed to be added (CaCO3)} \times \text{Average Flow Rate}) \text{ mg/l}$$

Alkalinity is to be provided in terms of $Na(HCO_3)$ then amount needed

$$= \left(\text{Total Alkanity needed as CaCO3} \times \frac{84}{50}\right) \text{mg/l}$$

Oxygen to be supplied in aeration tank

Theoretical oxygen required in aeration tank

$$= \left(\left(\text{Average flow} \times 1000 \times \frac{(So-S)}{1000}\right) - (1.42 \times Px, bio) + \left(\frac{(4.33 \times NOx)}{1000}\right)\right) \text{kg/d}$$

AOTR is equal to theoretical oxygen required in aeration tank

AOTR in kg/hr = $\left(\frac{\text{AOTR}}{24}\right)$ kg/hr

P_b/P_a = $\left(\exp\frac{9.81\times 28.97\times\text{Elevation from mean sea depth}}{8314\times 273.15+\text{Temperature}}\right)$

C_T value is taken from the table 4.5 and expressed in mg/l:

Table 4.5 Temperature, Barometric Pressure and Dissolved Oxygen concentration , mg/l

	Barometric pressure, millimetres of mercury									
Temp ^{0}C	735	740	745	750	755	760	765	770	775	780
0	14.12	14.22	14.31	14.41	14.51	14.60	14.70	14.80	14.89	14.99
1	13.73	13.82	13.92	14.01	14.10	14.20	14.29	14.39	14.48	14.57
2	13.36	13.45	13.54	13.63	13.72	13.81	13.90	14	14.09	14.18
3	13	11.09	13.18	13.27	13.36	11.45	13.53	13.62	13.71	13.80
4	12.66	12.75	12.83	12.92	13.01	13.09	13.18	13.27	13.35	13.44
5	12.33	12.42	12.50	12.59	12.67	12.76	12.84	12.93	13.01	13.10
6	12.02	12.11	12.19	12.27	12.35	12.44	12.52	12.60	12.68	12.77
7	11.72	11.80	11.89	11.97	12.05	12.13	12.21	12.29	12.37	12.45
8	11.44	11.52	11.60	11.67	11.75	11.83	11.91	11.99	12.07	12.15
9	11.16	11.24	11.32	11.40	11.47	11.55	11.63	11.70	11.78	11.86
10	10.90	10.98	11.05	11.13	11.20	11.28	11.35	11.43	11.50	11.58
11	10.65	10.72	10.80	10.87	10.94	11.02	11.09	11.16	11.24	11.31
12	10.41	10.48	10.55	10.62	10.69	10.77	10.84	10.91	10.98	11.05

13	10.17	10.24	10.31	10.38	10.46	10.53	10.60	10.67	10.74	10.81
14	9.95	10.02	10.09	10.16	10.23	10.29	10.36	10.43	10.50	10.57
15	9.73	9.80	9.87	9.94	10	10.07	10.14	10.21	10.27	10.34
16	9.53	9.59	9.66	9.73	9.79	9.86	9.92	9.99	10.06	10.12
17	9.33	9.39	9.46	9.52	9.59	9.65	9.72	9.78	9.85	9.91
18	9.14	9.20	9.26	9.33	9.39	9.45	9.52	9.58	9.64	9.71
19	8.95	9.01	9.07	9.14	9.20	9.26	9.32	9.39	9.45	9.51
20	8.77	8.83	8.89	8.95	9.02	9.08	9.14	9.20	9.26	9.32
21	8.60	8.66	8.72	8.78	8.84	8.90	8.96	9.02	9.08	9.14
22	8.43	8.49	8.55	8.61	8.67	8.73	8.79	8.84	8.90	8.96
23	8.27	8.33	8.39	8.44	8.50	8.56	8.62	8.68	8.73	8.79
24	8.11	8.17	8.23	8.29	8.34	8.40	8.46	8.51	8.57	8.63
25	7.96	8.02	8.08	8.13	8.19	8.24	8.30	8.36	8.41	8.47
26	7.82	7.87	7.93	7.98	8.04	8.09	8.15	8.20	8.26	8.31
27	7.68	7.73	7.79	7.84	7.89	7.95	8.0	8.06	8.11	8.17
28	7.54	7.59	7.65	7.70	7.75	7.81	7.86	7.91	7.97	8.02
29	7.41	7.46	7.51	7.57	7.62	7.67	7.72	7.78	7.83	7.88
30	7.28	7.33	7.38	7.44	7.49	7.54	7.59	7.64	7.69	7.75

31	7.16	7.21	7.26	7.31	7.36	7.41	7.46	7.51	7.46	7.62
32	7.04	7.09	7.14	7.19	7.24	7.29	7.34	7.39	7.44	7.49
33	6.92	6.97	7.02	7.07	7.12	7.17	7.22	7.27	7.31	7.36
34	6.80	6.85	6.90	6.95	7.0	7.05	7.10	7.15	7.20	7.24
35	6.69	6.74	6.79	6.84	6.89	6.93	6.98	7.03	7.08	7.13
36	6.59	6.63	6.68	6.73	6.78	6.82	6.87	6.92	6.97	7.01
37	6.48	6.53	6.57	6.62	6.67	6.72	6.76	6.81	6.86	6.90
38	6.38	6.43	6.47	6.52	6.56	6.61	6.66	6.70	6.75	6.80
39	6.28	6.33	6.37	6.42	6.46	6.51	6.56	6.60	6.65	6.69
40	6.18	6.23	6.27	6.32	6.36	6.41	6.46	6.50	6.55	6.59

[Source : Metcalf Eddy, 2004]

$\mathbf{C_{S,T,H}} = \left(\frac{Pb}{Pa} \times C_T\right)$ mg/l

$\mathbf{C_{20}}$ = 9.08 mg/l

$\mathbf{P_{atm,H}} = \left(\frac{\left(\frac{P_b}{P_a}\right) \times 101.325}{C_{20}}\right)$ m

$\mathbf{C_{s\_,T,H}} = \left[C_{S,T,H} \times 0.5 \times \left(\left(P_{atm,H} + \frac{SWD - 0.5}{P_{atm,H}}\right) + \frac{19}{21}\right)\right]$ mg/l

F (Fouling factor) ranges from 0.65 – 0.9

β ranges from 0.95 – 0.98

α ranges from 0.3 – 1.2

[Source : Metcalf Eddy, 2004]

C = 2 mg/l

SOTR = $\left[\frac{AOTR \times \left(C_{20} \times (1.024)^{20-T}\right)}{\alpha \times F \times \left(\left(\beta \times C_{S,T,H}\right) - C\right)}\right]$ kg/hr

4.11. Process Designing of Secondary Clarifier

Decide TSS in Sludge. It is expressed in g/m^3. It ranges from **4000 – 12000** g/m^3.

[Source: Metcalf Eddy, 2004]

R (Return Activated Sludge) = $\left(\frac{\text{Desired MLSS}}{\text{TSS in Sludge} - \text{Desired MLSS}}\right)$

This value is a ratio so no units.

Total Flow (including R) = $\left((1 + R) \times \text{Average Flow Rate}\right)$ Mld

Decide No of Secondary Clarifier units to be provided.

The surface overflow rate to be provided as decided ranges from 15 – 35 m^3/m^2/day.

(Table 4.3)

Area of each Clarifier required

$$= \left(\frac{\text{Total Flow} \times 1000}{\text{Surface Overflow Rate provided} \times \text{No of tanks provided}}\right) \text{ m}^2$$

Diameter per Clarifier = $\sqrt{\frac{\text{Area of each Clarifier required} \times 4}{\pi}}$ m

Decide Diameter per Clarifier provided. Let it be in metres.

Surface area of each Clarifier = $\left[\frac{\pi \times (\text{Diameter per Clarifier provided})^2}{4}\right]$ m^2

Decide SWD to be provided. Let it be in metres.

Volume of each clarifier provided

$$= (\text{Surface area of each Clarifier} \times \text{SWD}) \text{ m}^3$$

HRT = $\left[\frac{\text{Volume of each Clarifier Provided} \times \text{No of Secondary Clarifier units} \times 24}{\text{sTotal Flow} \times 1000}\right]$ hr

Solid Loading at avg. flow = $\left[\frac{\text{Total Flow} \times \text{Desired MLSS}}{\text{Area} \times 24}\right]$ kg/hr/m^2

Table 4.6 Type of settling and Solid Loading rate

Type of treatment	Solid Loading (kg/m^2.h)	
	Average	**Peak**
Settling followed by air-activated sludge (excluding extended aeration)	4 – 6	8
Selectors, biological nutrient removal	5 – 8	9
Settling following oxygen – activated sludge	5 – 7	9
Settling following extended aeration	1 – 5	7

[Source: Metcalf Eddy, 2004]

Solid Loading ranges from 4 – 6 kg/ m^2.hr **(Table 4.6)**

Length of weir = $[\pi \times \text{ Diameter per Clarifier provided}]$ m

One Way Avg. Weir loading

$$= \left[\frac{\text{Total Flow} \times 1000}{\text{Length of weir} \times \text{No of Secondary Clarifier units}}\right] \text{m}^3\text{/m/day}$$

Its value should come below 185 m^3/m/day [**Manual on Sewerage And Sewage Treatment CPHEEO]** otherwise two way Average Weir loading is to be provided.

Two Way Avg Weir loading = $\left[\frac{\text{One Way Avg Weir loading}}{2}\right]$ m^3/m/day

Free board as decided is also provided.

TSS concentration in secondary clarifier effluent =

$$\left(\frac{(\text{Influent TSS} \times \text{Average flow rate}) - (\text{Total Solids removed in both primary and secondary treatment})}{((\text{Average Flowrate in Mld} \times 1000) - \text{Combined Sludge Flowrate})} \times 1000\right)\text{mg/l}$$

TSS concentration in secondary clarifier effluent =

$(3 + (\text{Influent VSS})/\text{Influent TSS}\ ) \times (\text{TSS concentration in secondary clarifier effluent}/1.42)$

mg/l

4.12 Process Designing of Secondary Sludge Recycle Pumps

Let Return Sludge Ratio to be designed be 0.5

Design Flow = $\left[\frac{\text{Return Sludge Ratio} \times \text{Average Flow} \times 1000}{24}\right]$ m^3/hr

No of working pumps to be provided is decided based upon the return sludge flowrate.

Capacity of each pump required = $\left[\frac{\text{Design Flow}}{\text{No of working pumps}}\right]$ m^3/hr

Decide Capacity of each pump to be provided. It is expressed in m^3/hr.

Decide No of standby units to be provided.

4.13. Process Designing of Sludge Handling System

In the municipal/domestic wastewater sludge handling system there are two types of sludges:-

- Primary sludge – It is produced by solids settling the primary clarifier, characterised by high putrescibility and dewater ability when compared to biological sludge. Consistency of primary sludge is 2-6%.**[Turovskiy and mathai,2006]**
- Secondary Sludge – It is also known as biological sludge as it is produced from biological process (Activated Sludge Process) and contains microorganisms grown on organic matter. Consistency of secondary sludge is 0.5-1.5%.**[Turovskiy and mathai,2006]**

4.13.1 Primary Sludge

Total Solids removed in Primary Clarifier

= $\left[\frac{\text{TSS removal efficiency} \times \text{Influent TSS} \times \text{Avg Flow}}{100}\right]$ kg/d

Table 4.7 Typical data for the physical characteristics and quantities of sludge produced from various wastewater treatment processes

Treatment operations or process	Specific gravity of solids	Specific gravity of sludge	Dry solids lb/10^3 gal		Dry solids kg/10^3 m^3	
			Range	Typical	Range	Typical
Primary sedimentation	1.4	1.02	0.9-1.4	1.25	110-170	150
Activated sludge (waste biosolids)	1.25	1.005	0.6-0.8	0.7	70-100	80
Trickling filter (waste biosolids)	1.45	1.025	0.5-0.8	0.6	60-100	70
Extended areation (waste biosolids)	1.30	1.015	0.7-1.0	0.8	80-120	100
Areated lagoon (waste biosolids)	1.30	1.01	0.7-1.0	0.8	80-120	100
Filtration	1.20	1.005	0.1-0.2	0.15	12-24	20
Algae removal	1.20	1,005	0.1-0.2	0.15	12-24	20
Chemical addition to primary tanks for phosohorus removal						
Low lime (350-500 mg/L)	1.9	1.04	2.0-3.3	2.5	240-400	300
High lime (800-1600 mg/L)	2.2	1.05	5.0-11.0	6.6	600-1300	800
Suspended growth nitrification	–	–	–	–	–	–
Susupended growth denitrification	1.20	1.005	0.1-0.25	0.15	12-30	18

Roughing filters	1.28	–	–	–	–	–

[Source: Metcalf Eddy, 2004]

Table 4.8 Expected solids concentrations from various treatment operations and processes

Operation or process application	Solids concentration % dry solids	
	Range	Typical
Primary settling tank:		
Primary sludge	5-9	6
Primary sludge to a cyclone degritter	0.5-3	1.5
Primary sludge and waste activated sludge	3-8	4
Primary sludge and trickling –filter humus	4-10	5
Primary sludge with iron salt addition for phosphorus removal	0.5-3	2
Primary sludge with low lime addition for phosphorus removal	2-8	4
Primary sludge with high lime addition for phosphorus removal	4-16	10
Scum	3-10	5
Secondary settling tank:		
Waste activated sludge with primary settling	0.5-1.5	0.8
Waste activated sludge without primary settling	0.8-2.5	1.3
High –purity oxygen with primary settling	1.3-3	2
High –purity oxygen without primary settling	1.4-4	2.5
Trickling – filter humus	1-3	1.5
Rotating biological contactor waste sludge	1-3	1.5
Gravity thickener:		
Primary sludge	5-10	8
Primary sludge and waste activated sludge	2-8	4

Primary sludge and trickling –filter humus	4-9	5
Dissolved air flotation thickner : Waste activated sludge with polymer addition Waste activated sludge without polymer addition Centrifuge thickner (waste activated sludge only) Gravity –belt thickner (waste activated sludge with ploymer addition)	 4-6 3-5 4-8 4-8	 5 4 5 5
Anaerobic digester : Primary sludge Primary sludge and waste activated sludge Primary sludge and trickling –filter humus	 2-5 1.5-4 2-4	 4 2.5 3
Aerobic digester: Primary sludge Primary sludge and waste activated sludge Primary sludge and trickling –filter humus	 2-5 -7 1.5-4 0.8-2.5	 3.5 2.5 1.3

[Source: Metcalf Eddy, 2004]

Consistency of the solid is considered. It ranges from 2 – 6 %. **(Table 4.7)**

Specific Gravity of Sludge is considered. It is 1.02 for Primary Sludge**.(Table 4.8)**

Flowrate of Primary Sludge

$$= \left[\frac{\text{Total Solids removed} \times 100}{\text{Consistency of the solid} \times \text{Specific Gravity of Sludge}}\right] m^3/day$$

4.13.2. Process Designing of Primary Sludge Sump

Volume of primary Sludge = Flowrate of Primary Sludge. It is expressed in m^3.

Decide HRT. It is expressed in hrs.

Volume of Sump $= \left[\frac{\text{Volume of primary Sludge} \times \text{HRT}}{24}\right] m^3$

4.13.3. Process Designing of Primary Sludge Pump

Volume of sludge to be fed to sludge thickener = Volume of primary Sludge.

It is expressed in m^3.

Decide Running time of sludge thickener considered. Let it be in Hrs.

Sludge flowrate = $\left[\frac{\text{Volume of sludge to be fed to sludge thickner}}{\text{Running time of sludge thickner considered}}\right]$ m^3/hr

Decide No of working pumps provided.

Capacity of each pump required = $\left[\frac{\text{Sludge flowrate}}{\text{No of working pumps provided}}\right]$ m^3/hr

Decide Capacity of each pump to be provided.

Decide No of standby pumps to be provided.

4.13.4. Secondary Sludge

Total solids removed in Secondary Clarifier = $P_{X,TSS}$ (in kg/d).

Consider Consistency of solids. It ranges from 0.5 – 1.5. **(Table 4.7)**

Consider Specific Gravity. It is 1.005 for Secondary Sludge. **(Table 4.8)**

Flow rate of Secondary Sludge

$$= \left[\frac{\text{Total solids removed} \times 100}{\text{Total solids removed} \times \text{Specific Gravity of Sludge} \times 1000}\right] m^3/day$$

4.13.5. Process designing of Secondary Sludge Sump

Volume of excess sludge = Flowrate of Secondary Sludge.

Decide no. of sludge withdrawal cycle. It is expressed in No/Shift.

Decide Duration of Shift.

No of Shifts per day = $\left[\frac{24}{\text{Duration of Shift}}\right]$

No of sludge withdrawal cycles per day

$$= \text{No. of sludge withdrawal cycle per Shift} \times \text{Duration of Shift}$$

Sludge withdrawal per cycle = $\left[\frac{\text{Volume of excess sludge}}{\text{No of sludge withdrawal cycles per day}}\right]$ m^3

Decide HRT. It is expressed in hrs.

Volume required = $[\text{Sludge withdrawal per cycle} \times \text{HRT}]$ m^3

4.13.6. Process Designing of Secondary Sludge Pump

Volume of excess sludge to be fed to the Sludge thickener = Flow rate of Secondary Sludge

Secondary sludge flow rate

$$= \left[\frac{\text{Volume of excess sludge to be fed to the Sludge thickner}}{24}\right] \text{m}^3/\text{hr}$$

Decide **No of working Pumps provided**.

Capacity of each pump required = $\left[\frac{\text{Secondary sludge flowrate}}{\text{No of working Pumps provided}}\right]$ m^3/hr

Decide capacity of each pump provided.

Decide no. of standby pumps to be provided.

4.13.7. Process Designing of Sludge Thickener

Combined Sludge flow rate =

$$[\text{Primary Sludge Flowrate} + \text{Secondary sludge flowrate}] \text{ m}^3/\text{hr}$$

Combined Sludge Quantity (kg/day)

$$= [\text{Total Solids removed in Primary Clarifier} + \text{Total solids removed in Secondary Clarifier}]$$

Specific Gravity of combined Sludge is considered. It value is usually taken 1.04. **(Table 4.7)**

Consistency of solids = $\left[\frac{\text{Combined Sludge Quantity} \times 100}{\text{Combined Sludge flowrate} \times \text{Specific Gravity} \times 1000}\right]$ %

Table 4.9 Typical concentrations of unthickened and thickened sludges and solids loadings for gravity thickeners

Type of sludge or Biosolids	Solids concentration,%		Solids loading	
	Unthickened	Thickened	lb/ft^2.d	kg/m^2.d
Separate:				
Primary sludge	2-6	5-10	20-30	100-150
Trickling –filter humus sludge	1-4	3-6	8-10	40-50
Rotating biological contactor	1-3.5	2-5	7-10	35-50
Air-activated sludge	0.5-1.5	2-3	4-8	20-40
High –purity oxygen –activated sludge	0.5-1.5	2.3	4-8	20-40
Extended aeration-activated sludge	0.2-1.0	2.3	5-8	20-40
Anaerobically digested primary sludge from primary digester	8	12	25	120
Combined:				
Primary and trickling –filter humus sludge	2-6	5-9	12-20	60-100
Primary and rotating biological contactor	2-6	5-8	10-18	50-90
Primary and waste –activated sludge	0.5-1.5	4-6	5-14	25-70
	2.5-4.0	4-7	8-16	40-80
Waste –activated sludge and trickling-filter humus sludge	0.5 – 2.5	2-4	4-8	20-40
Chemical (tertiary)sludge :				
High lime	3-4.5	12-15	24-61	120-300

Low lime Iron	3-4.5 0.5-1.5	10-12 3-4	10-30 2-10	50-150 10-50

[Source: Metcalf Eddy, 2004]

Consider Solid loading Rate. It is expressed in kg/m^2.d. It ranges from 25 – 70 kg/m^2.d, when consistency of solids ranges from 0.5 – 1.5 % and Solid loading rate ranges from 40 – 80 kg/m^2.d when consistency of solids ranges from 2 – 4 %.**(Table 4.9)**

Area of thickener = $\left[\frac{\text{Combined Sludge Quantity}}{\text{Solid loading Rate}}\right]$ m^2

Decide no. of thickeners to be provided.

Area of each thickner = $\left[\frac{\text{Area of thickener}}{\text{No of Thickeners provided}}\right]$ m^2

Diameter of each thickner required = $\left(\sqrt{\frac{4 \times \text{Area of each thickner}}{\pi}}\right)$ m

Decide diameter of each thickner provided.

Total area of Thickener provided

$$= \left[\frac{\pi \times \text{No of Thickeners provided} \times (\text{diameter of each thickner provided})^2}{4}\right] \text{m}$$

Decide SWD to be provided in metres.

Table 4.10 Typical solids concentrations and capture values for various solids – processing methods

Operation	Solid concentration,%		Solid capture, %	
	Range	**Typical**	**Range**	**Typical**
Gravity thickeners: Primary sludge only Primary and waste – activated	 4 – 10 2 – 6	 6 4	 85 – 92 80 – 90	 90 85

Flotation thickener: With chemicals Without chemicals	 4 – 6 3 – 5	 5 4	 90 – 98 80 – 90	 95 90
Centrifuge thickeners: With chemicals Without chemicals	 4 – 8 3 – 6	 5 4	 90 – 98 80 – 90	 95 85
Belt – filter press: With chemicals	 15 – 30	 22	 85 – 98	 93
Filter press: With chemicals	 20 – 50	 36	 90 – 98	 92
Centrifuge dewatering: With chemicals Without chemicals	 10 – 35 10 – 30	 22 18	 85 – 98 55 – 90	 92 80

[Source : Metcalf Eddy,2004]

Consider solid capture in thickener. It ranges between 80 – 90 %. **(Table 4.10)**

Consider consistency of thickened Sludge solids. It ranges between 2– 6 % for thickened sludge. **(Table 4.10)**

Total solids withdrawn

$$= \left[\frac{\text{Solid capture in Thickener} \times \text{Combined Sludge Quantity}}{100}\right] \text{ kg/day}$$

Total volume of thickened sludge withdrawn

$$= \left[\frac{\text{Total solids withdrawn} \times 100}{\text{Consistency of Thickened Sludge solids} \times \text{Specific Gravity} \times 1000}\right] \text{ m}^3\text{/day}$$

Decide hydraulics loading desired. It ranges between 6 –12 m^3/m^2/day.**[Metcalf Eddy, 2004]**

Dilution water

$$= \left[\begin{matrix}(\text{Hydarulic loading desired} \times \text{Total area of Thickener provided}) \\ - \text{Combined Sludge flowrate}\end{matrix}\right] \text{m}^3/\text{day}$$

Total Flow rate of thickener overflow

$$= \left[\begin{matrix}(\text{Hydarulic loading desired} \times \text{ Total area of Thickener provided}) \\ -\text{Total volume of thickened sludge withdrawn}\end{matrix}\right] \text{m}^3/\text{day}$$

Total solids in overflow water

$$= \left[\begin{matrix}\left(\frac{\text{Average Design Flow} \times \text{Dilution water}}{1000}\right) + \\ \left(1 - \frac{\text{Solid capture in Thickener}}{1000} \times \text{ Combined Sludge Quantity }\right)\end{matrix}\right] \text{kg/day}$$

BOD_c (BOD concentration) = $UBOD \times 0.68$ mg/l

BOD in overflow water = $(\text{Total solids in overflow water} \times 0.6 \times 1.42 \times 0.68\,)$ kg/day.

Assuming biological solids that are biodegradable = 0.6

4.13.8. Process Designing of Thickener Overflow Sump

Total Flowrate of thickener overflow = $\left[\frac{\text{Total volume of thickner overflow}}{24}\right]$ m^3/hr

Decide HRT. Let it be in minutes.

Volume required = $\left[\frac{\text{Total volume of thickener overflow} \times \text{HRT}}{60}\right]$ m^3

4.13.9. Process Designing of Thickener Overflow Pump

Total Flowrate of thickener overflow = $\left[\frac{\text{Total volume of thickner overflow}}{24}\right]$ m^3/hr

Decide no. of pumps to be provided.

Capacity of each pump required = $\left[\frac{\text{Total Flowrate of thickener overflow}}{\text{No of pumps provided}}\right]$ m^3/hr

Decide capacity of each pump to be provided.

Decide no .of standby pumps to be provided.

4.13.10. Process Designing of Sludge Sump after Thickener

Flow rate of Thickened Sludge = Total volume of thickened sludge withdrawn.

Decide No of sludge withdrawal cycle. It is expressed in No/Shift.

Decide Duration of Shift.

No of Shifts per day = $\left[\frac{24}{\text{Duration of Shift}}\right]$

No of sludge withdrawal cycles per day

$$= \text{No. of sludge withdrawal cycle per Shift} \times \text{Duration of Shift}$$

Sludge withdrawal per cycle = $\left[\frac{\text{Volume of excess sludge}}{\text{No of sludge withdrawal cycles per day}}\right]$ m^3

Decide HRT. It is expressed in hrs.

Volume required = $[\text{Sludge withdrawal per cycle} \times \text{HRT}]$ m^3

4.13.11. Process Designing of Sludge Digester Feed Pump

Volume of thickened sludge to be fed to sludge digesters = Total volume of thickened sludge withdrawn.

Decide Running time of Sludge digesters in a day considered.

Thickened sludge flow rate

$$= \left[\frac{\text{Volume of thickened sludge to be fed to sludge digesters}}{\text{Running time of Sludge digesters in a day}}\right] m^3/\text{day}$$

Decide no. of working pumps provided.

Capacity of each pump required = $\left[\frac{\text{Thickened sludge flow rate}}{\text{No of working pumps provided}}\right]$ m^3/day

Decide Capacity of each pump provided.

Decide No of standby pumps to be provided.

4.13.12. High rate Anaerobic Digester

Design Flow coming in Digester = Total volume of thickened sludge withdrawn.

Total solid in Sludge = Total Solids withdrawn by the thickener from the combined sludge. It is expressed in kg/day.

Mass flow in digester = $\left[\frac{\text{Total solid in Sludge} \times 100}{\text{Consistency of thickened Sludge}}\right]$ kg/day

Ratio of VSS in Sludge.

Total VSS in the sludge

$$=[\text{Ratio of VSS in Sludge} \times \text{Total solid in Sludge}] \text{ kg/day}$$

Decide operating temperature. It is expressed in ^{o}C.Decide SRT from the table given below:

Table 4.11 Suggested solids retention times for use in the design of complete – mix anaerobic digester

Operating temperature, ^{o}C	**SRT (minimum)**	**SRT_{des}**
18	11	28
24	8	20
30	6	14
35	4	10
40	4	10

[Source: Metcalf Eddy, 2004]

Degree of stabilization = $[13.7 \times \text{LN}(\text{SRT}) + 18.9]$ %

Volatile Solids Reduced = $\left[\frac{\text{Total VSS in the sludge} \times \text{Degree of stabilization}}{100}\right]$ kg/day

Gas Production per kg of VSS reduced ranges from 0.75 to 1.12 m^3/day. **[Metcalf Eddy, 2004]**

Gas Produced

$$=[\text{Volatile Solids Reduced} \times \text{Gas Production per kg of VSS reduced}] \text{ m}^3\text{/day}$$

Ratio of density of gas to air =0.86. It is standard value.

Solids used up in gas production

$$= \left[\begin{array}{c}\text{Volatile Solids Reduced} \times \text{Gas Production per kg of VSS reduced} \\ \times \text{ Ratio of density of gas to air} \times \text{Density of air}\end{array}\right] \text{kg/day}$$

Solids in Digested Sludge =

$$[\text{Total solid in Sludge} - \text{ Solids used up in gas production}] \text{ kg/d}$$

Consider consistency of digested solids. It is expressed in %.

TSS in supernatant = 5000mg/l. **(Table 4.14)**

Solids in Supernatant

$$= \left[\frac{\left(\frac{\text{Mass flow in digester} \times \text{Consitency of Digested Solids}}{100}\right) - \text{Solids in Digested Sludge}}{\left(\frac{\text{consistency of Digested Solids}}{100 \times \frac{\text{TSS in supernatant}}{100000}} - 1\right)}\right] \text{kg/d}$$

Digested Solids = (Solids in Digested Sludge - Solids in Supernatant) kg/day

Supernatant Flow rate $= \left[\frac{\text{Solids in Supernatant} \times 100}{\text{TSS in supernatant}}\right]$ m^3/day

Digested Sludge Flow rate $= \left[\frac{\text{Digested Solids} \times 100}{\text{Consistency of Digested Solids} \times 1000}\right]$ m^3/day

BOD concentration in supernatant = 1000mg/l. **(Table 4.14)**

BOD in supernatant flow = $(\text{Supernatant Flow rate} \times \frac{1000}{1000})$ kg/d

Dimension of Digester

Decide No of digesters to be provided.

Total Flow in Digester =

$$(\text{Design flow coming in the digester} \times \text{SRT}) \text{ m}^3\text{/day}$$

Actual volume of each digester required $= \frac{\text{Total Flow in Digester}}{\text{No of digesters provided}}$ m^3/day

Decide Ratio of Diameter to SWD provided. It ranges from 1.5 – 4 **[Manual on Sewerage And Sewage Treatment CPHEEO].**

Decide SWD to be provided. It ranges from 6 – 12 metres [**Manual on Sewerage And Sewage Treatment CPHEEO].**

Area of each Digester required = $\frac{\text{Actual volume of each digester required}}{\text{SWD}}$ m^2

Diameter of each digester required = $\sqrt{\frac{4 \times \text{Area of each Digester required}}{\pi}}$ m

Decide Grit storage capacity. Let it be in metres.

Decide Scum Blanket . Let it be in meters (m).

Decide Free Board .Let it be in meters (m).

Power Requirement for Mixing

Decide Mean Velocity Gradient (G) .Let it be in per second. It ranges between 30 – 85 /sec. (Table 4.12)

Assume Dynamic Viscosity (μ) to be 0.00146 Ns/m^2.

Table 4.12 Typical design parameters for anaerobic digester mixing system

Parameter	Type of mixing system	Typical values
Unit power	Mechanical systems	0.005 -0.008 KW/m^3 of digested volume
Unit gas flow	Gas mixing: unconfined Confined	 0.0045 – 0.005 m^3/m^3.min 0.005 – 0.007 m^3/m^3.min
Velocity gradient G	All	50 – 80 s^{-1}
Turnover time of tank contents	Confined gas mixing and mechanical systems	20 – 30 min

[Source : Metcalf Eddy, 2004]

Volume of digester = $\left(\frac{\text{Total flow in digester}}{\text{No. of digester provided}}\right)$ m^3

Power required in (Nm/s)

$$= ((\text{Dynamic viscosity} \times \text{volume of digester}) \times (\text{Mean Velocity Gradient})^2)$$

Power required in (KW) = $\left(\frac{(\text{Power required in Nm/s})}{1000}\right)$

Decide **no. of mixers** to be provided.

Capacity of mixers required = $\left(\frac{(\text{Power required in (Kw)})}{\text{No.of mixers provided}}\right)$ KW

Decide capacity **of mixers** to be provided.

Biogas Holder

Biogas generated = $\left(\frac{(\text{Gas Production per kg of VSS reduced})}{\text{Votalite Solids Reduced}}\right)m^3/d$

Gas Holder Volume required = (0.25 x Biogas generated) m^3

Decide **no. of gas holder** to be provided.

Volume of each gas holder required = $\left(\frac{\text{Gas Holder Volume required}}{\text{No.of gas holder provided}}\right)$ m^3

Decide height of Biogas Holder.

Area of each gas holder = $\left(\frac{\text{Volume of each gas holder required}}{\text{height of gas holder}}\right)m^2$

Diameter of gas holder required = $\left(\frac{(\text{Area of each gas holder x 4})}{3.14}\right)^{0.5}$ m

4.14. Process Designing of Gas Flaring System

Biogas generated m^3per day = $\left(\frac{(\text{Gas Production per kg of VSS reduced})}{\text{Votalite Solids Reduced}}\right)$

Biogas generated m^3 per hour = $\left(\frac{\text{Biogas genrated } m^3/d}{24}\right)$

Decide burner capacity required to be 1.5 times of design gas generation.

Burner capacity required = (Biogas generated x 1.5) m3/hr

Decide **no. of working burners** to be provided.

Capacity of burner required = $\left(\frac{(\text{Burner capacity required m3/hr})}{\text{No.of working burners provided}}\right)$ m^3/hr

Decide **capacity of burner** to be provided.

Decide **no. of stand by burners** to be provided.

4.15. Process Designing of Gas Engine Generation set

Biogas generated m^{3}per day = $\left(\frac{\text{(Gas Production per kg of VSS reduced)}}{\text{Votalite Solids Reduced}}\right)$

Decide energy content of biogas to be **6.4 KW.hr/d**

Decide electric efficiency of gas motor.

Energy produced per day

= (Biogas generated x energy content of biogas x electric efficiency of gas motor) KW.hr/d

Decide **no. of gas engine set** to be provided.

Decide **no. of running hours of gas engine set**.

Capacity of gas engine set required

$$= \left(\frac{\text{(energy produced per day x no.of running hours of gas engine set)}}{\text{No of running hours of gas engine set}}\right)\text{KWe}$$

Decide **capacity of gas engine** set to be provided.

4.16. Process Designing of Centrifuge Feed Sump

Quantity of digested sludge generated

= (Solids in Digested Sludge - Solids in Supernatant) kg/day

Flow rate of digested sludge = $\left[\frac{\text{Digested Solids} \times 100}{\text{Consistency of Digested Solids} \times 1000}\right]$ m^3/day

Decide **no. of sludge with drawl cycles** to be provided/shift

Decide **duration of 1 shift**.

No. of shifts per day = $\left(\frac{24}{\text{Duration of 1 shift}}\right)$

No of sludge withdrawal cycles per day

= (No. of shifts per day x No. of sludge withdrawal cycles)

Sludge withdrawal per cycle = $\left(\frac{\text{Flow rate of digested sludge}}{\text{No of sludge withdrawal cycles per day}}\right)$ m^3

Assume HRT to be 1 hr.

Volume required = $\left(\frac{\text{Sludge withdrawal per cycle}}{\text{HRT}}\right)$m^3

4.17. Process designing of Centrifuge Feed Pump

Volume of digested sludge to be fed to centrifuge in a day

$$= \left[\frac{\text{Digested Solids} \times 100}{\text{Consistency of Digested Solids} \times 1000}\right] \text{ m}^3/\text{day}$$

Decide **no. of operating hours of centrifuge** to be provided.

Capacity of centrifuge required

$$= \left(\frac{\text{Volume of digested sludge to be fed to centrifuge in a day}}{\text{No. of operating hours of centrifuge}}\right) \text{m}^3/\text{day}$$

Decide **no. of working centrifuge pumps** to be provided.

Capacity of each pump required

$$= \left(\frac{\text{Capacity of centrifuge required}}{\text{no.of working centrifuge pumps provided}}\right) \text{ m}^3/\text{hr}$$

Decide **no. of working centrifuge pumps** to be provided.

Capacity of each pump required

$$= \left(\frac{\text{Capacity of each pump required}}{\text{No. of working centrifuge pumps provided}}\right) \text{ m}^3/\text{hr}$$

Decide **capacity of each pump** to be provided.

Decide **no of standby pump** to be provided.

4.18. Process designing of Centrifuge

Table 4.13 Solid Capture and solids in solid cake in different types of sludge

Type of Sludge	Cake solids	Solid capture, %	
		Without chemicals	With chemicals

Untreated:			
Primary	25 – 35	75 – 90	95+
Primary and trickling filter	20 – 25	60 – 80	95+
Primary and air - activated	12 – 20	55 – 65	92+
Waste sludge:			
Trickling filter	10 – 20	60 – 80	92+
Air activated	5 – 15	60 – 80	92+
Oxygen activated	10 – 20	60 – 80	92+
Anaerobically digested:			
Primary	25 – 35	65 – 80	92+
Primary and trickling filter	18 – 25	60 – 75	90+
Primary and air activated	15 – 20	50 – 65	90+
Aerobically digested:			
Waste activated	8 – 10	60 – 75	90+

[Source: Metcalf Eddy, 2004]

Table 4.14 Typical BOD and TSS concentrations in the return flows for various processes

Operation	**BOD,(mg/L)**		**Suspended solids, (mg/L)**	
	Range	**Typical**	**Range**	**Typical**
Gravity thickening supernatant	100-400	250	80 -350	200

Primary sludge Primary sludge+ waste activated sludge	60-400	300	100 -350	250
Flotation thickening supernatant	50-1200	250	100-2500	300
Centrifuge thickening centrate	170-3000	1000	500-3000	1000
Aerobic digestion supernatant	100-1700	500	100-10,000	3400
Anaerobic digestion (two stage high rate supernatant	500-5000	1000	1000-11,500	4500
Centrifuge dewatering centrate	100-2000	1000	200-20,000	5000

Belt filter press filtrate	50-5000	300	100-2000	1000
Recessed plate filter press filtrate	50-250		50-1000	
Sludge lagoon supernatant	100-200		5-200	
Sludge drying bed underdrainage	20-500		20-500	
Composting leache		2000		
Incinerator scrubber water	30-80		600-8000	
Depth filter washwater	50-500		100-1000	
Microscreen washwater	100-500		240-1000	
Carbon absorber washwater	50-400		100-1000	

[Source: Metcalf Eddy, 2004]

Percentage of Solids in Sludge cake ranges between 15 – 20 %**. (Table 4.13)**

Specific Gravity of Sludge is constant, its value is **1.06**.

Total Solids in Sludge cake

$$= \left(\frac{((\text{Solids in Digested Sludge} - \text{Solids in Supernatant})\ x\text{Solid capture})}{100}\right) \text{kg/d}$$

Flow rate of Sludge cake

$$= \left(\frac{(\text{Total Solids in Sludge cake x 100x Specific Gravity of Sludge})}{\text{Percentage of Solids in Sludge cake x 1000}}\right) \text{m}^3\text{/d}$$

TSS in centrate

$$=\left((\text{Solids in Digested Sludge} - \text{Solids in Supernatant})\text{x}\left(1 - \frac{\text{solid capture}}{100}\right)\right) \text{kg/d}$$

Flow rate of centrate

$$= \left[\frac{\text{Digested Solids} \times 100}{\text{Consistency of Digested Solids} \times 1000}\right] - \left(\frac{(\text{Total Solids in Sludge cake x 100x Specific Gravity of Sludge})}{\text{Percentage of Solids in Sludge cake x 1000}}\right) \text{m}^3\text{/d}$$

BOD in centrate $= \left(\frac{(\text{Flow rate of centrate x 2000})}{1000}\right)$ Kg/d

BOD concentration as **2000g/m^{3}**. (**Table 4.14**)

Total BOD recycled to primary clarifier

= (BOD in centrate + BOD in supernatant flow + BOD in thickener overflow) kg/d

Total TSS recycled to primary clarifier

= (TSS in centrate + TSS in supernatant flow + TSS in thickener overflow) kg/d

CHAPTER 5

SIMULATION & VALIDATION

The computer program for the design of wastewater treatment plant was test-run and the results were compared with those of the existing plant. The program is interactive. The program has been fed with relevant input data for each unit of Nangal sewage treatment plant and executed. Data are entered as and when necessary as per the guidance obtained from the program.

5.1. Simulation

On the execution of the program first name of the project, company and department is asked and all other succeeding steps of the process designing are shown below in the snapshots:-

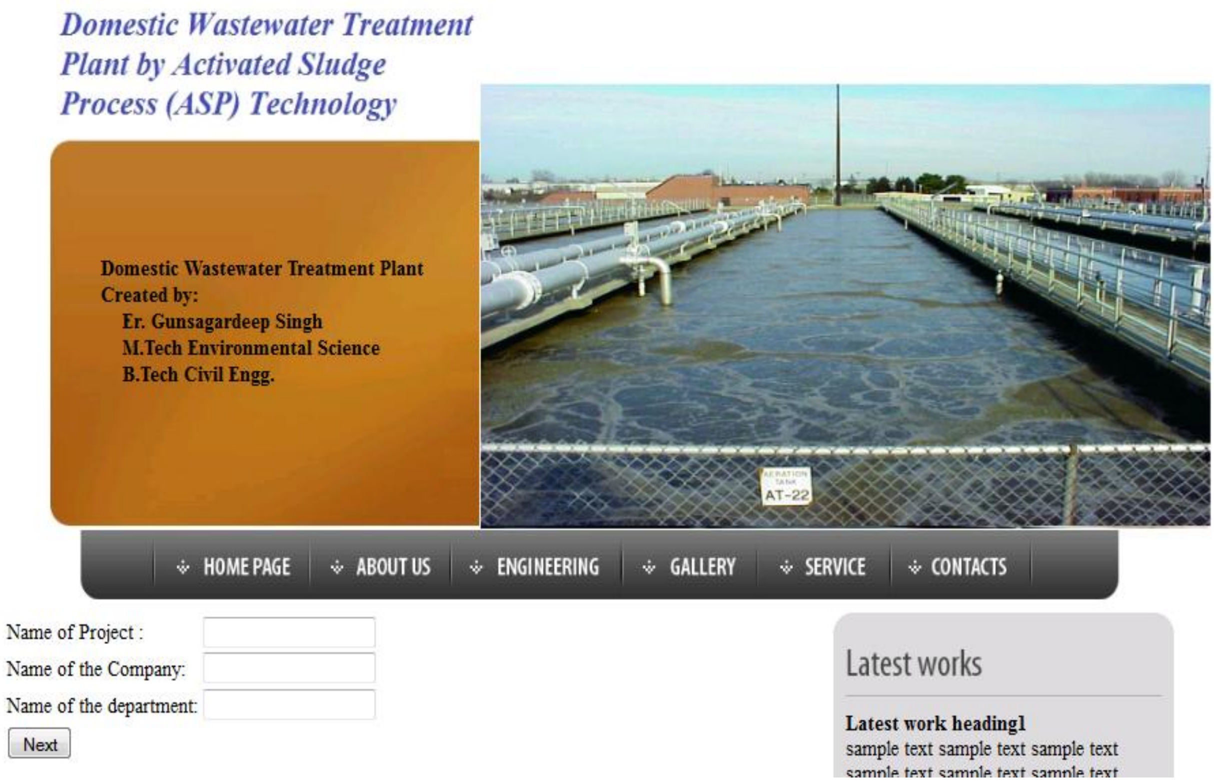

Fig 5.1 General Information

Raw sewage chararcterstics

pH (0-14):	7	
TSS:	400	mg/l
BOD_5:	300	mg/l
COD:	500	mg/l
Total Coliform:	10000000	MPN/100ml
Faecal Coliform:	1000000	MPN/100ml
Temperature:	15	^0C
VSS:	240	mg/l
TKN:	30	mg/l
NH_4-N :	25	mg/l
Alkanity as $CaCO_3$	140	mg/l
Elevation From Mean Sea Depth:	220	m

Next

Fig 5.2. Raw sewage characteristics

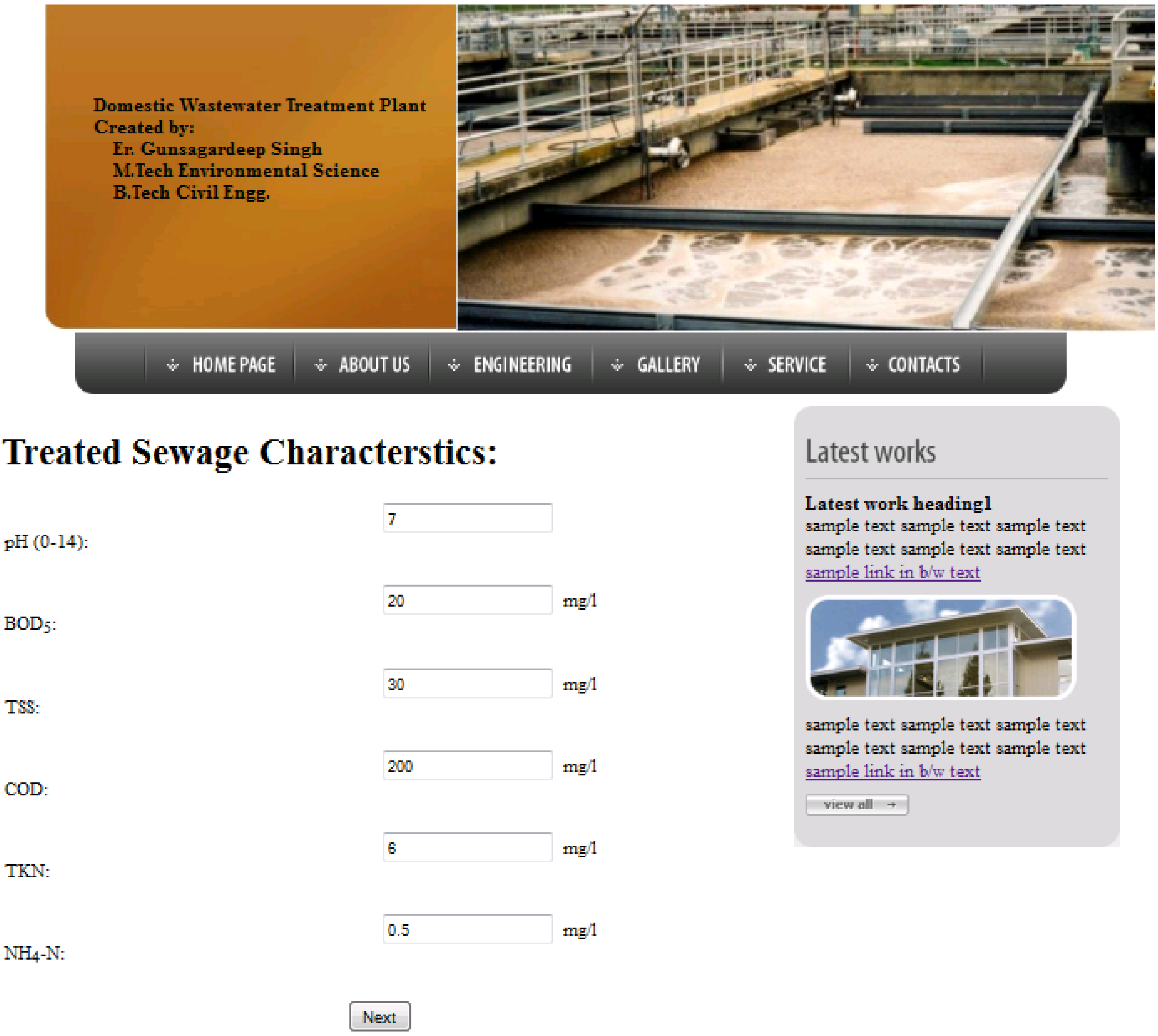

Fig 5.3 Treated wastewater characteristics

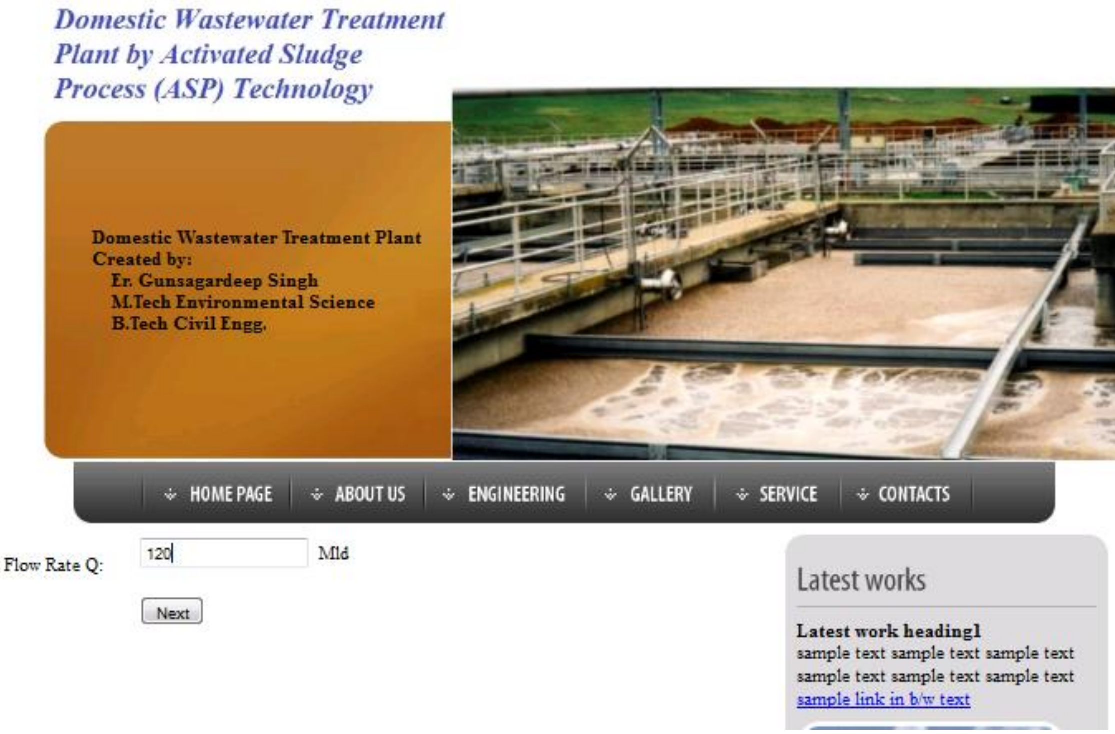

Fig 5.4 Flowrate

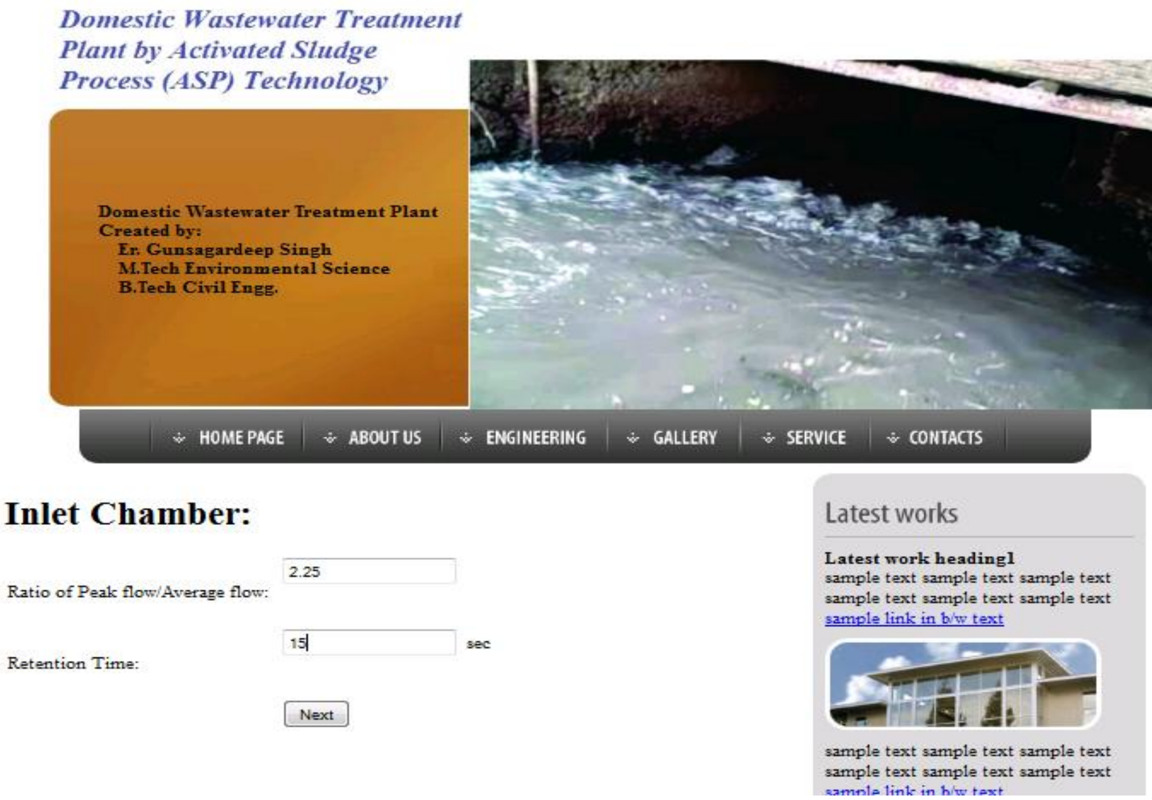

Fig 5.5.(a) Inlet Chamber

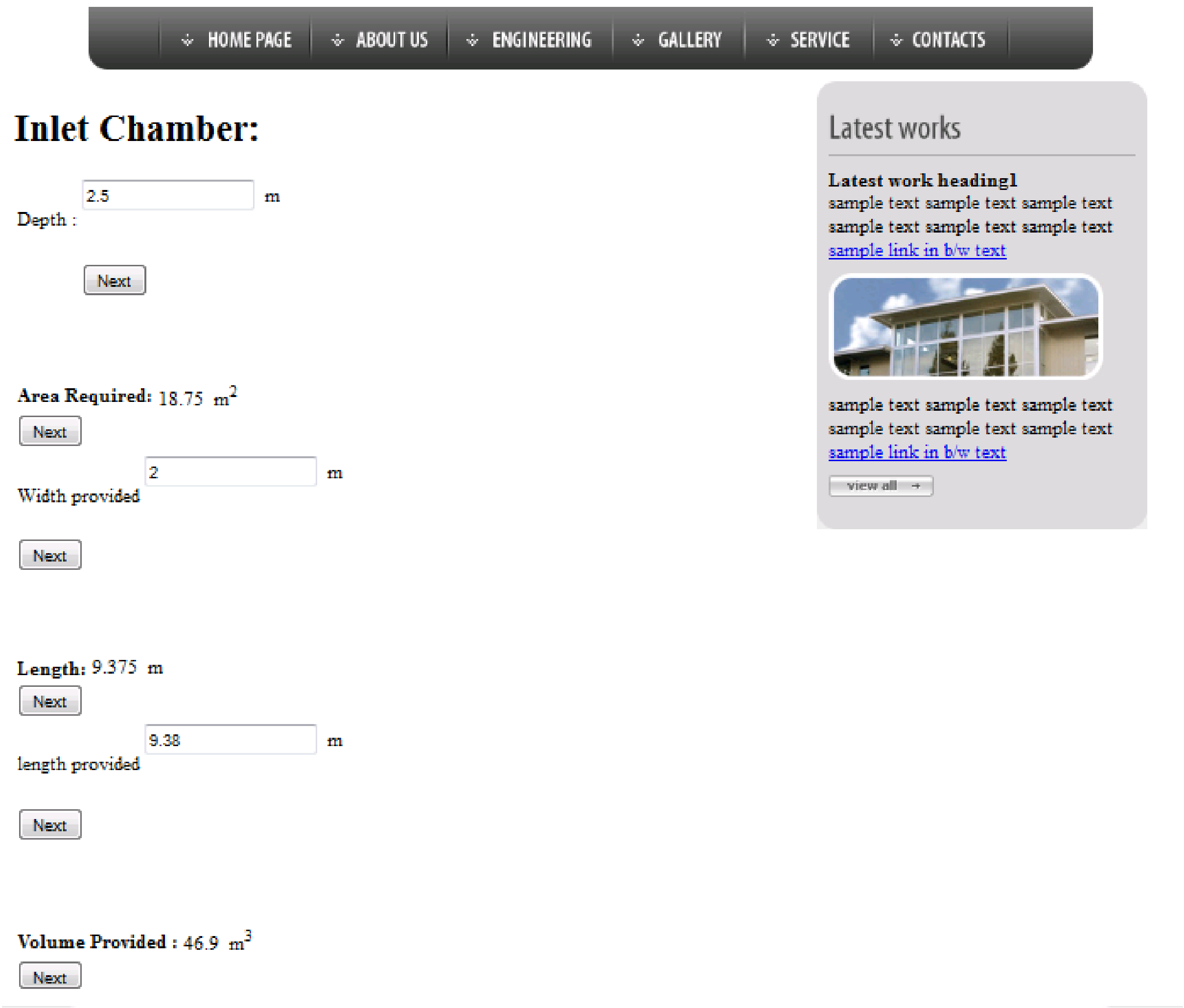

Fig 5.5 (b) Inlet Chamber

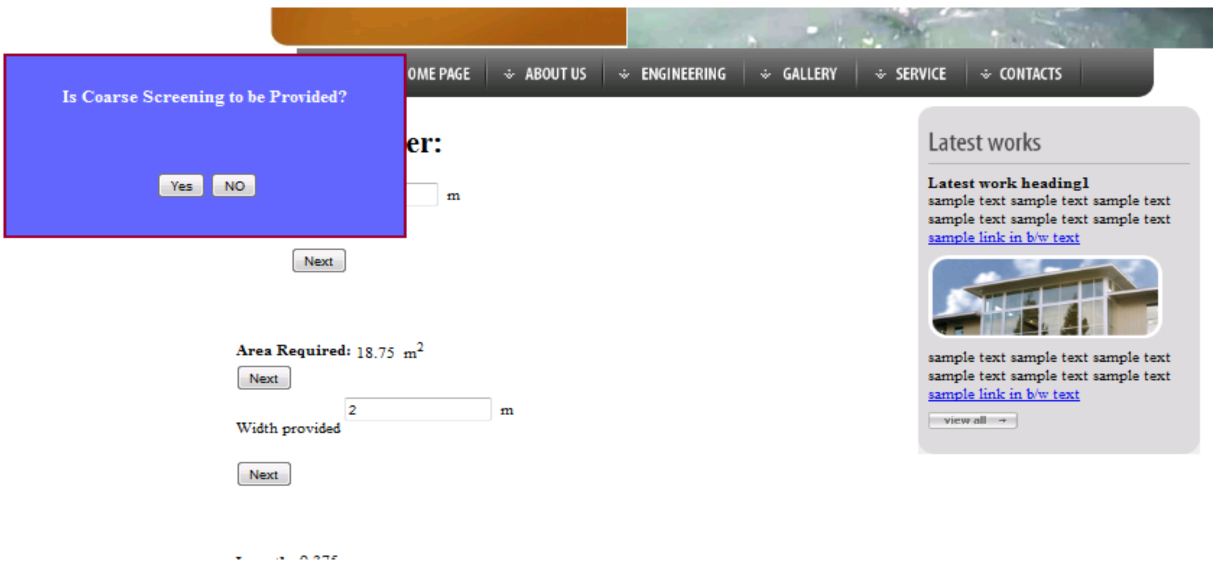

Fig 5.6 (a) Coarse Screening

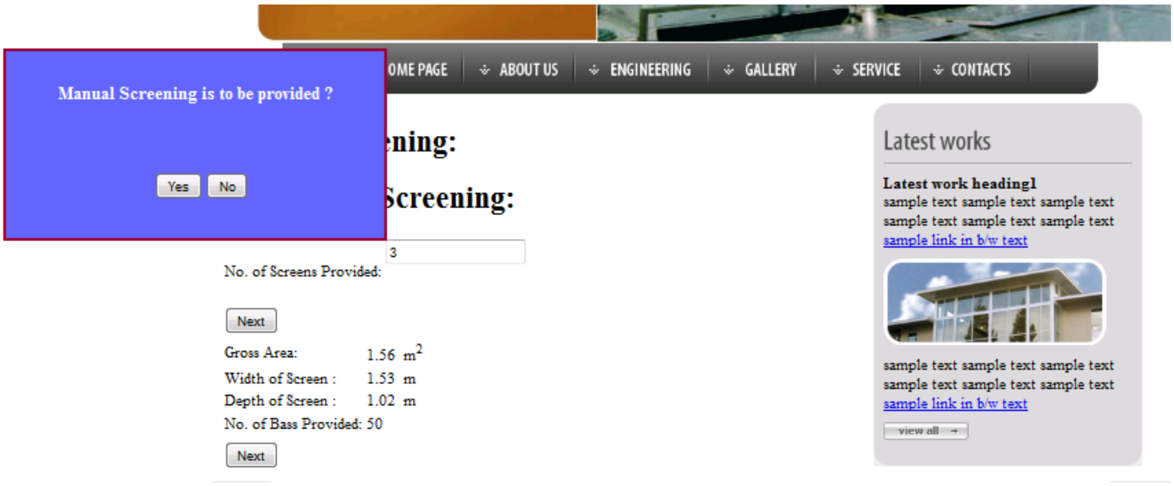

Fig 5.6 (b) Coarse Screening

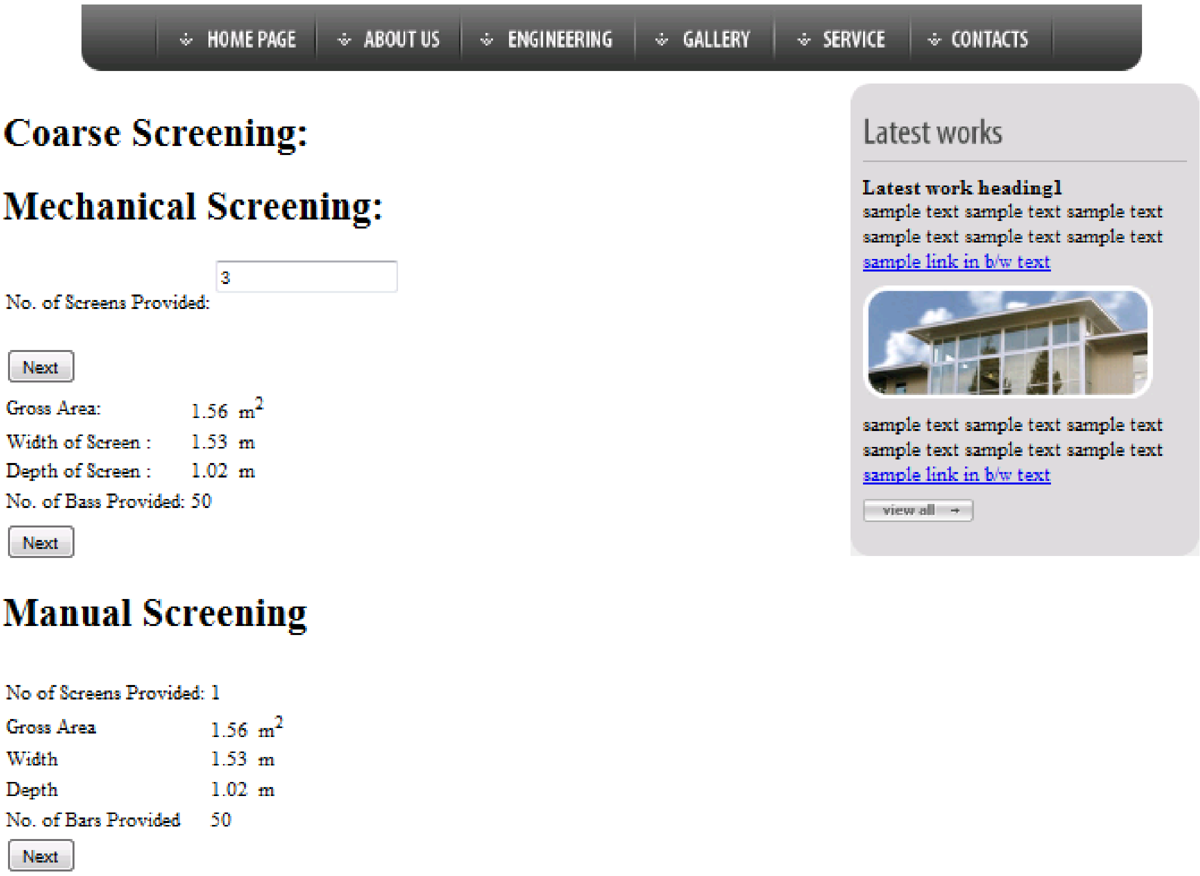

Fig 5.6 (c) Coarse Screening

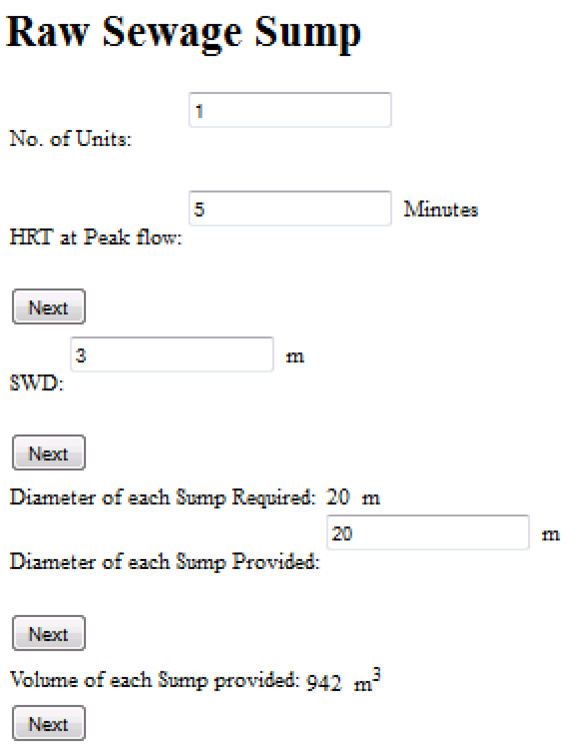

Fig 5.7 Raw Sewage Sump

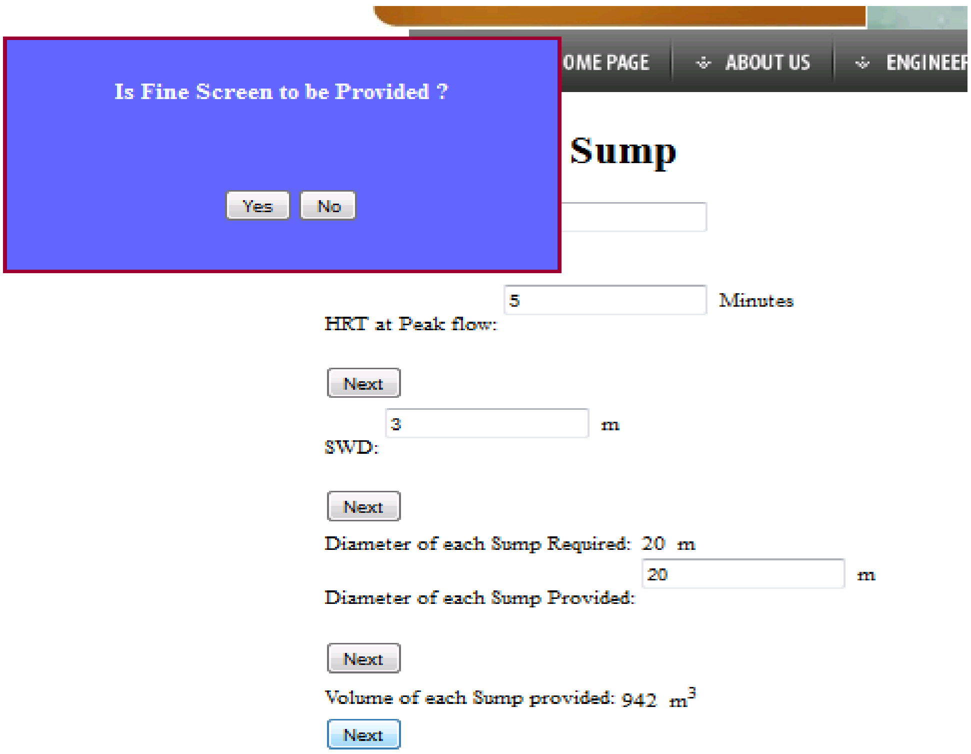

Fig 5.8 (a) Fine Screening

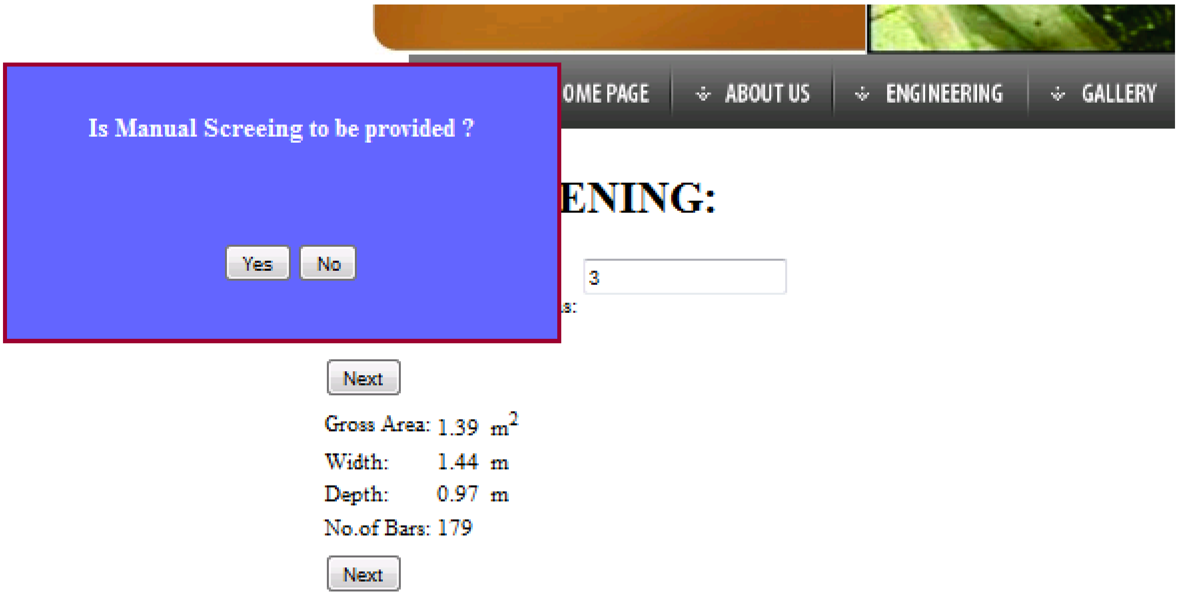

Fig 5.8 (b) Fine Screening

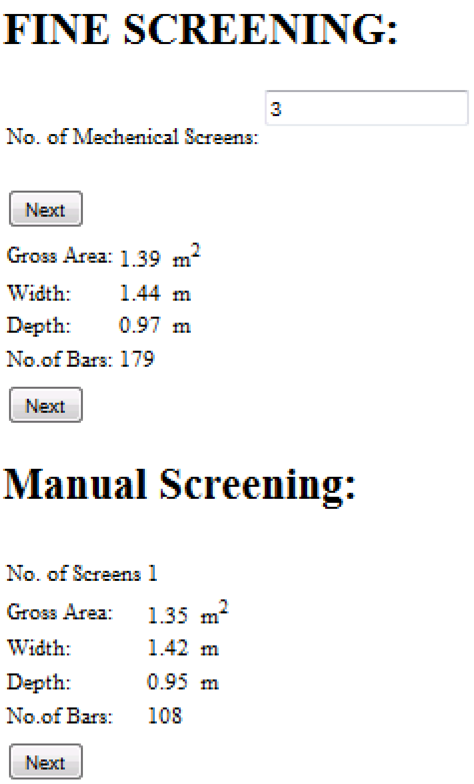

Fig 5.8 (c) Fine Screening

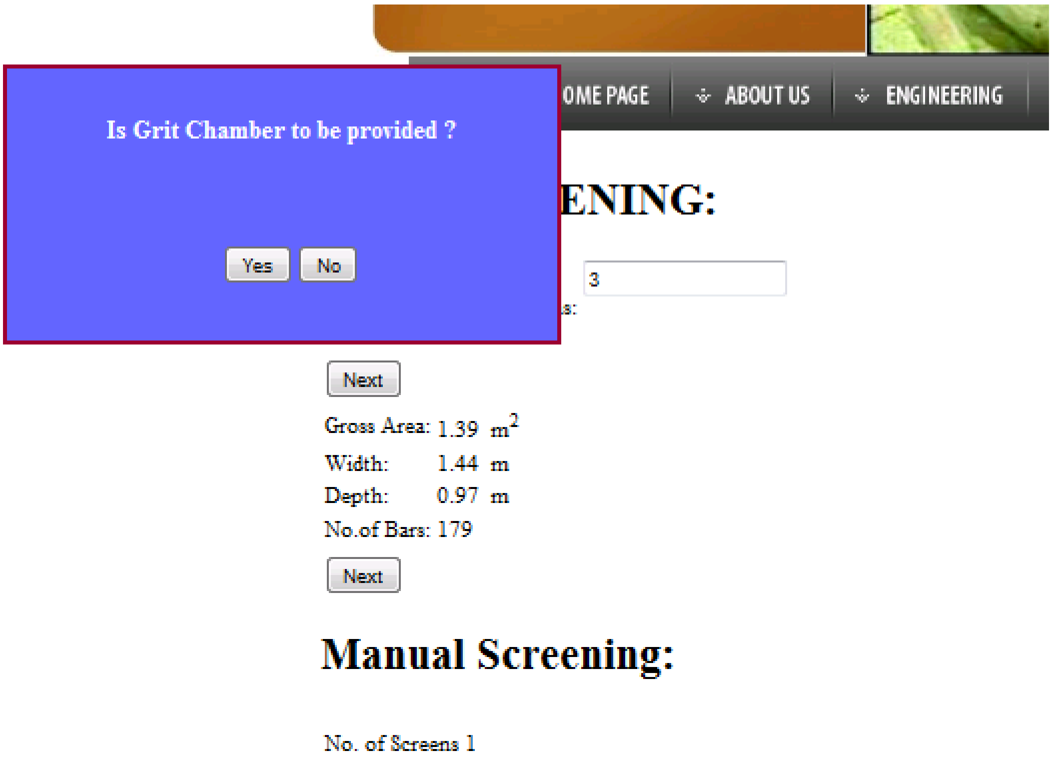

Fig 5.9 (a) Grit Chamber

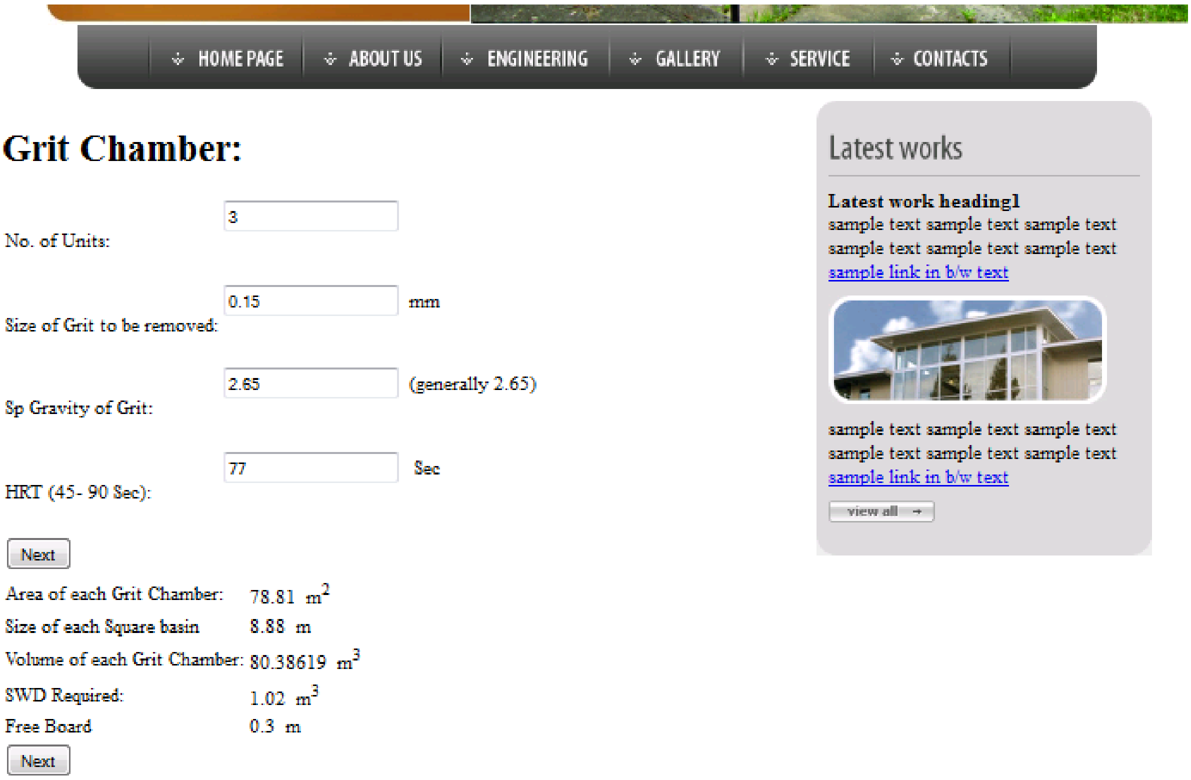

Fig 5.9.(b) Grit Chamber

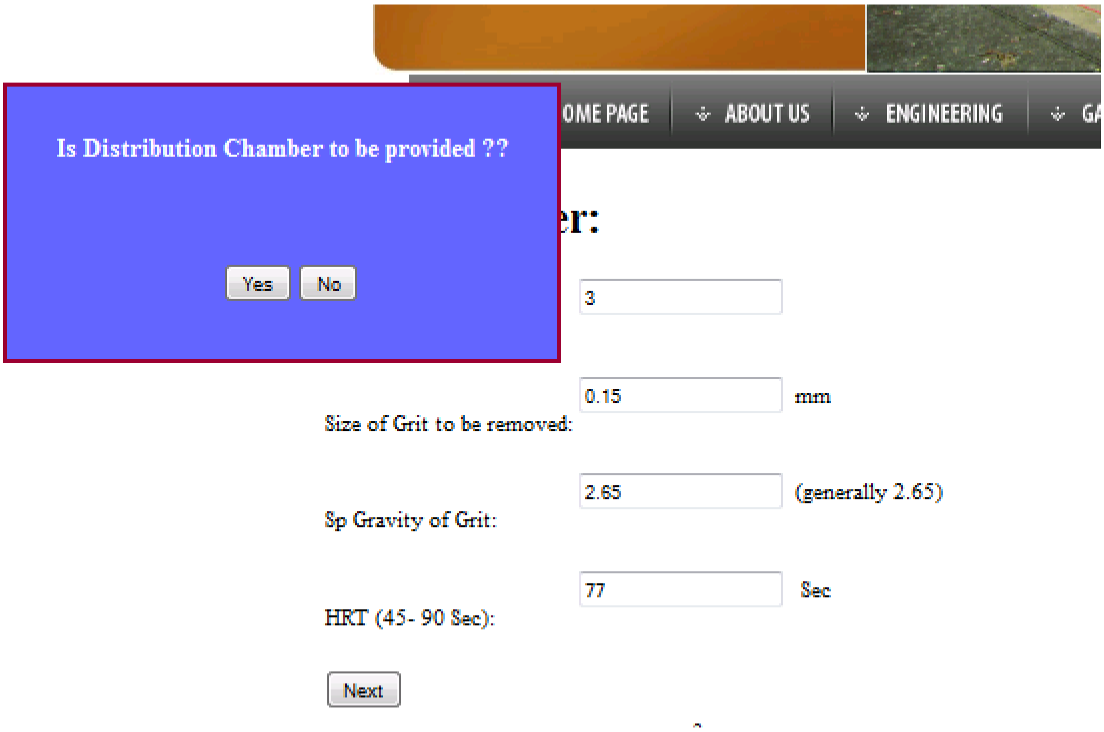

Fig 5.10 (a) Distribution Chamber

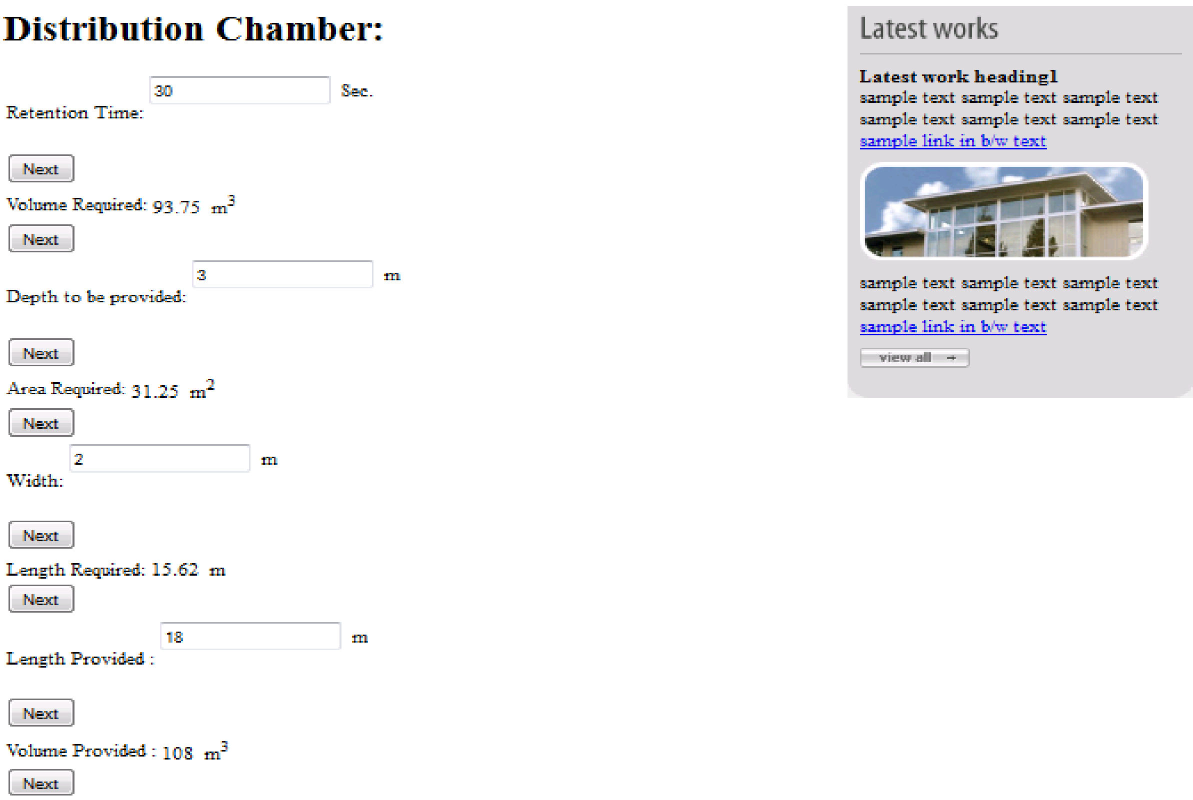

Fig 5.10 (b) Distribution Chamber

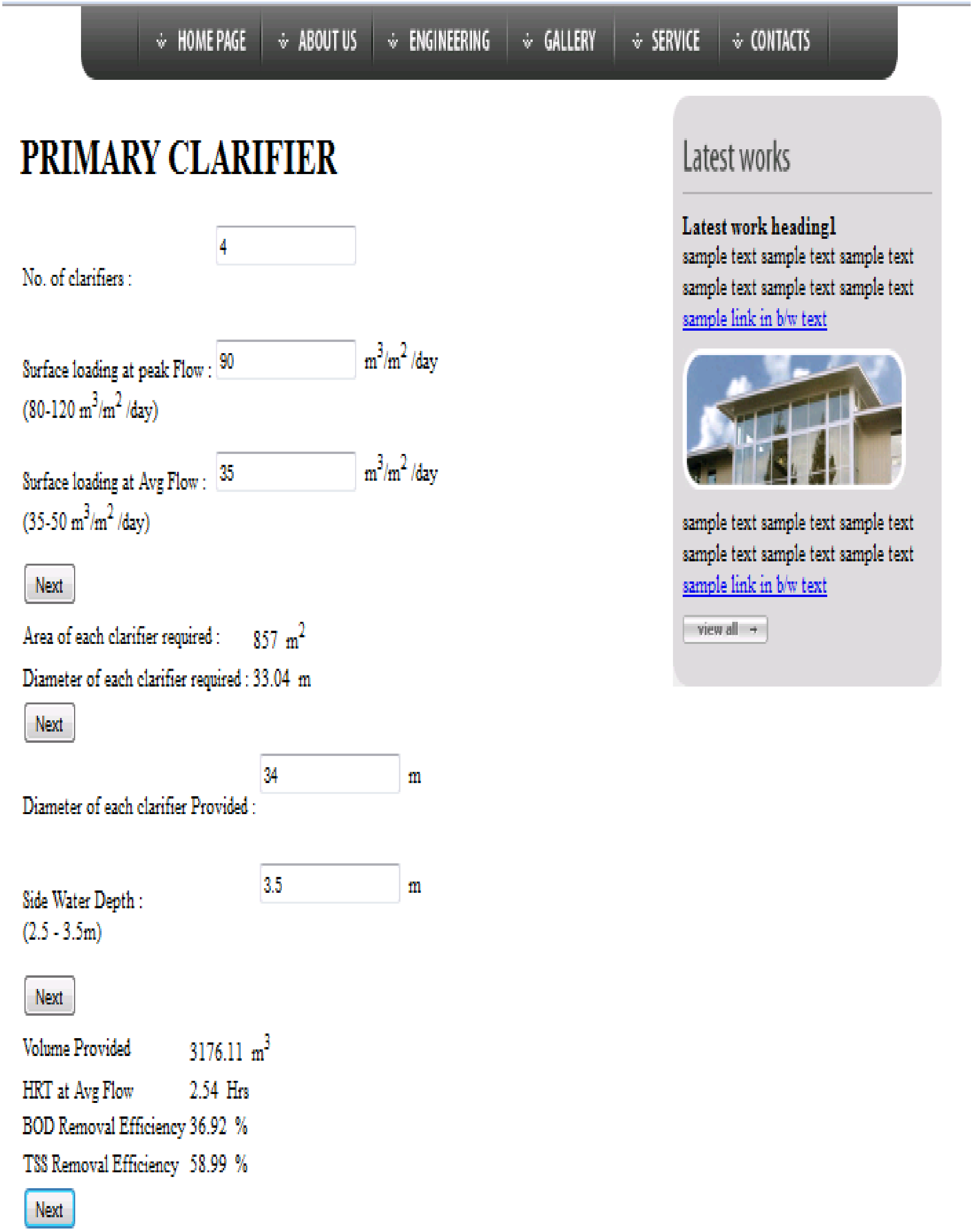

Fig 5.11 Primary Clarifier

AERATION TANK :

SRT : 11.53 Days

Desired MLSS : 2500 g/m^3

Next

Volume of aeration Tank : 72860 m^3

HRT : 14.57 Hrs

F/M : 0.16 Kg/Kg.d

BOD Loading : 0.31 $Kg/m^3.d$

Next

Alkalinity used for nitrification as $CaCO_3$: 151.7964 mg/l

Total Alkalinity needed as $CaCO_3$: 11015.57 kg/d

If Alkalinity is to be provided in terms of $Na(HCO_3)$ then amount needed : 18506.15 kg/d

Next

Side water Depth (maximun: 7.5m) : 7.5 m

Next

Area : 9714.667 m^2

No. of Tanks to be Provided: 6

No - Flow Rate

2 - 20-40 Mld

4 - 40-80 Mld

6 - 80-150 Mld

>6 - for above

Next

Area of each tank : 1619 m^2

Width of each tank : 22.5 m

Length of each tank : 72 m

Length of each tank provided 75 m

Next

Total volume provided 75938 m^3

Next

Latest works

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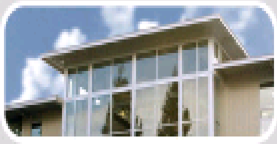

sample text sample text sample text sample text sample text sample text

sample link in b/w text

view all →

Fig 5.12 Aeration Tank

Fig 5.13 Oxygen to be supplied for aeration

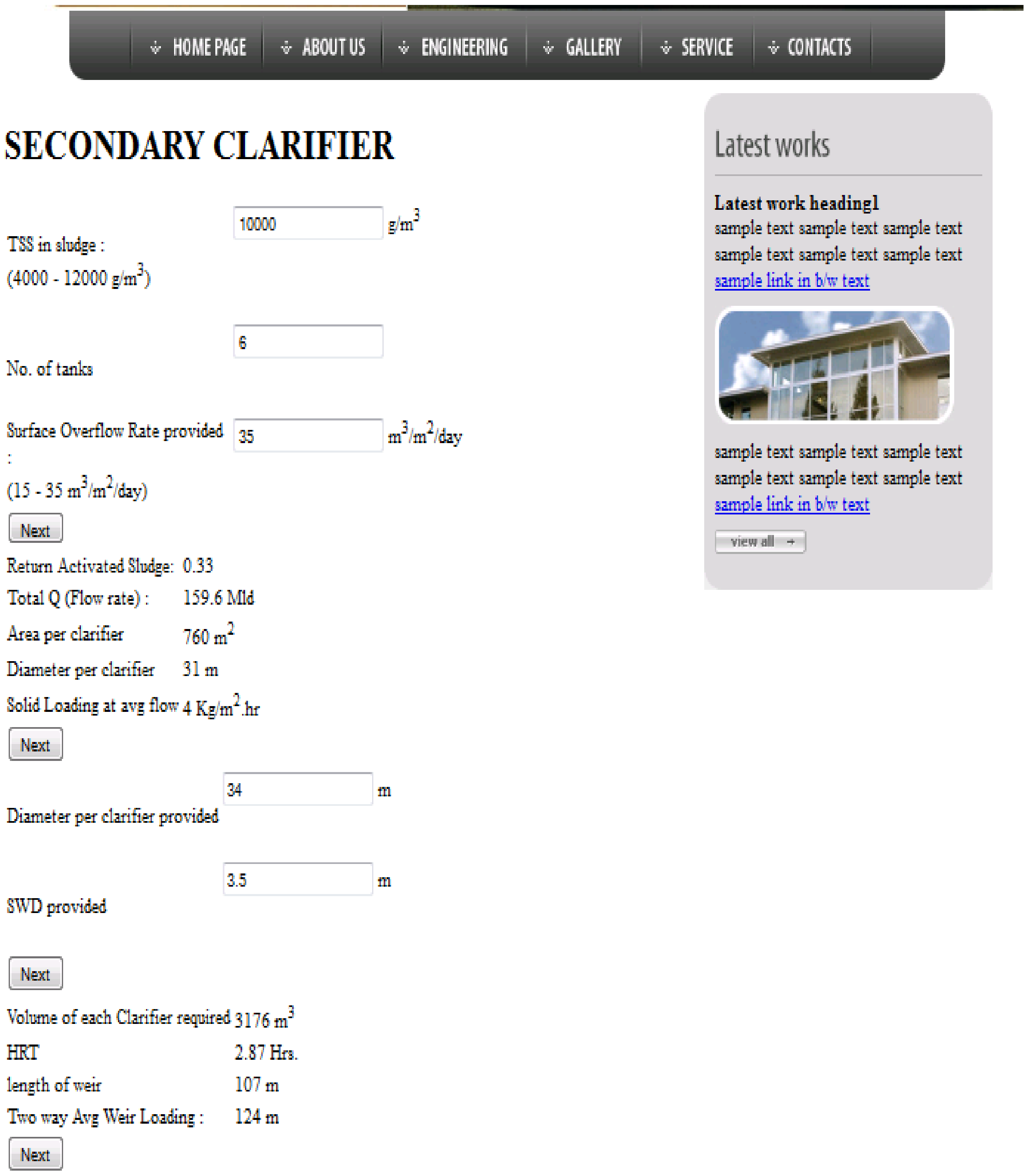

Fig 5.14 Secondary Clarifier

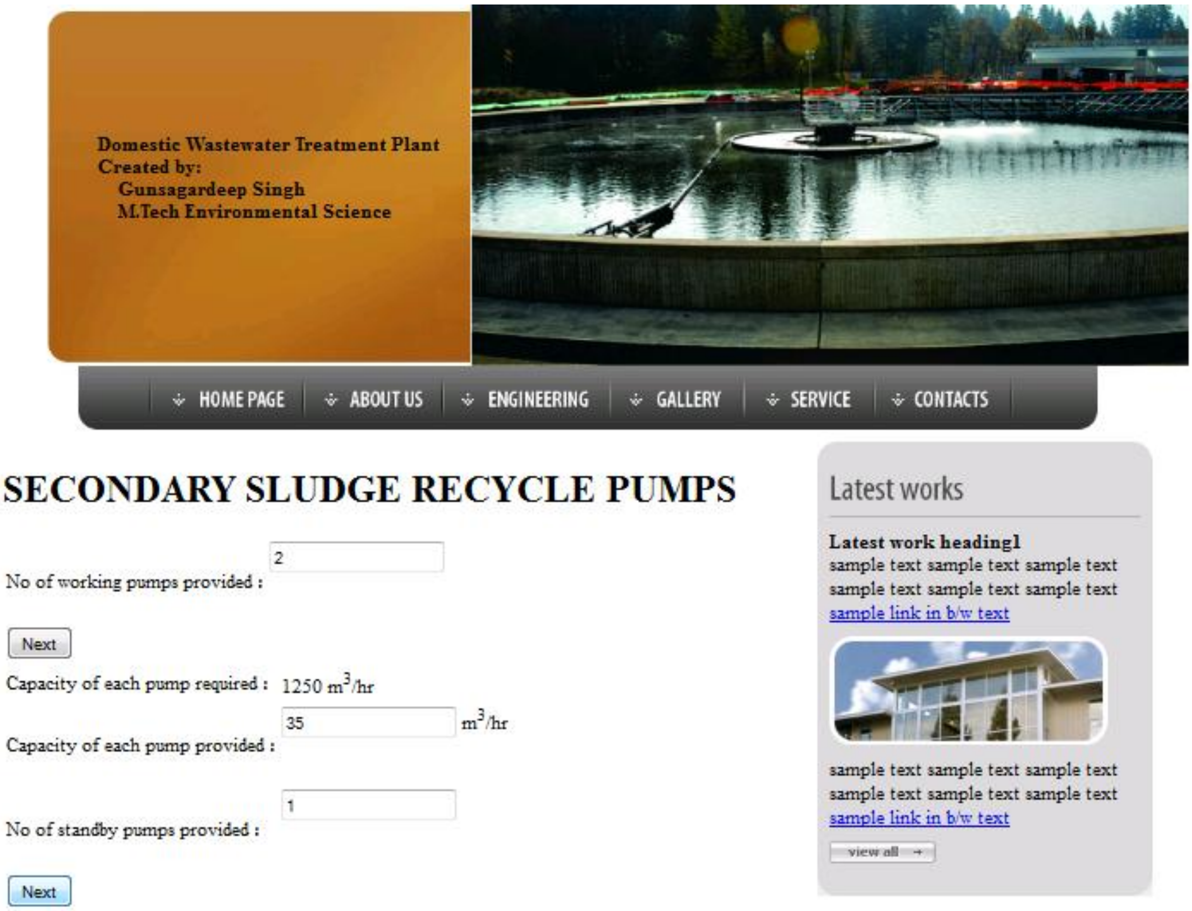

Fig 5.15 Secondary Sludge Recycle Pumps

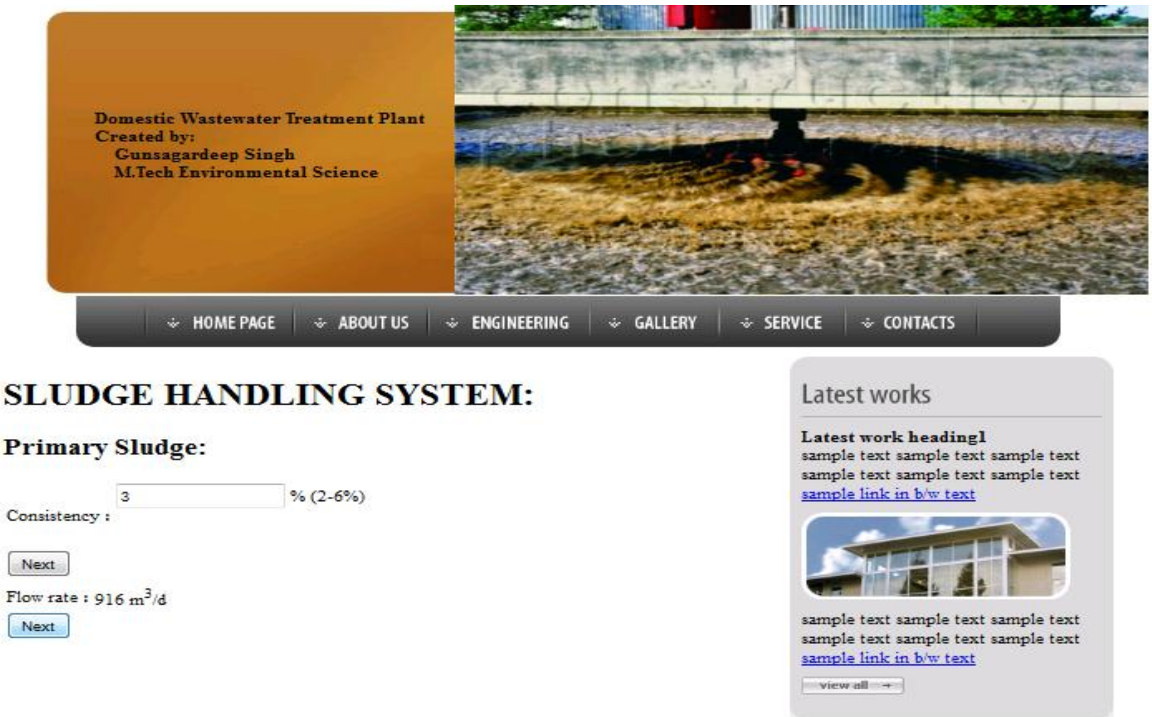

Fig 5.16 Sludge Handling System

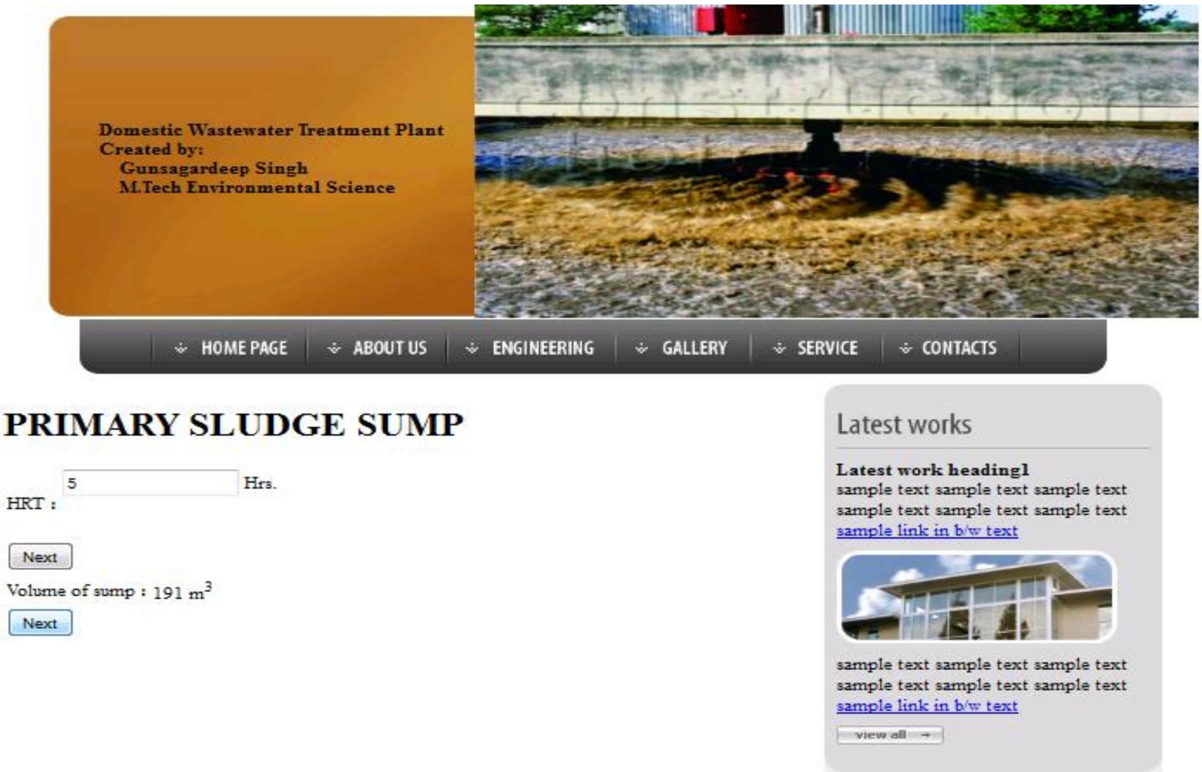

Fig 5.17 Primary Sludge Sump

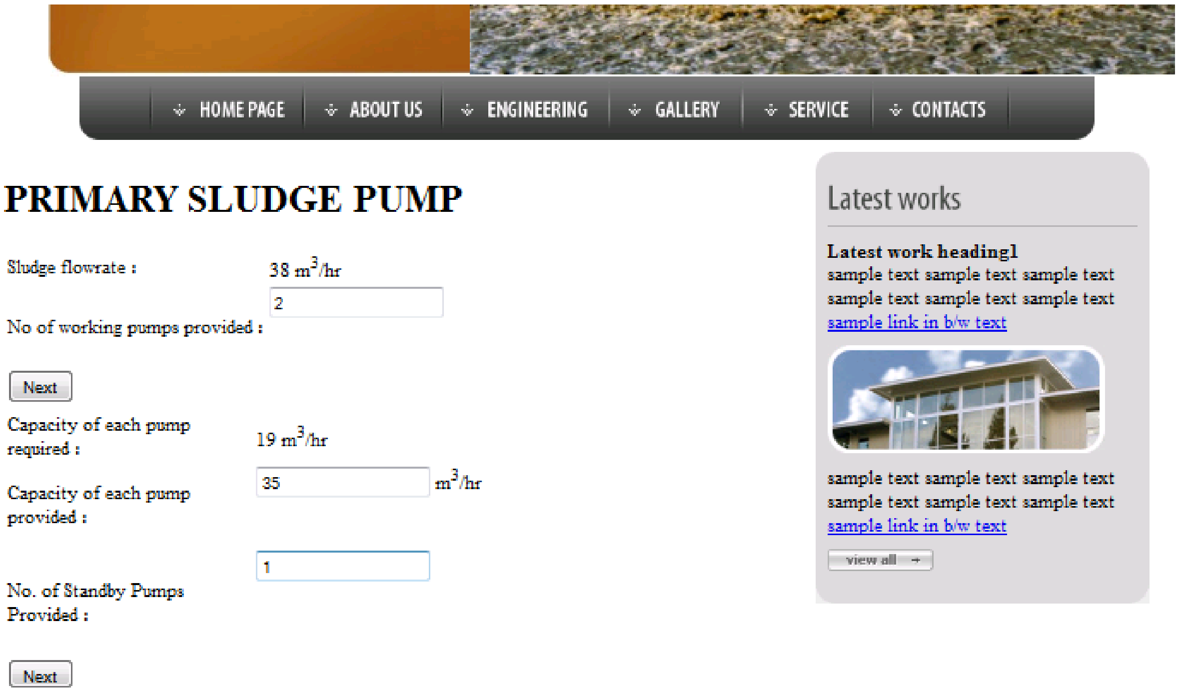

Fig 5.18 Primary Sludge Pump

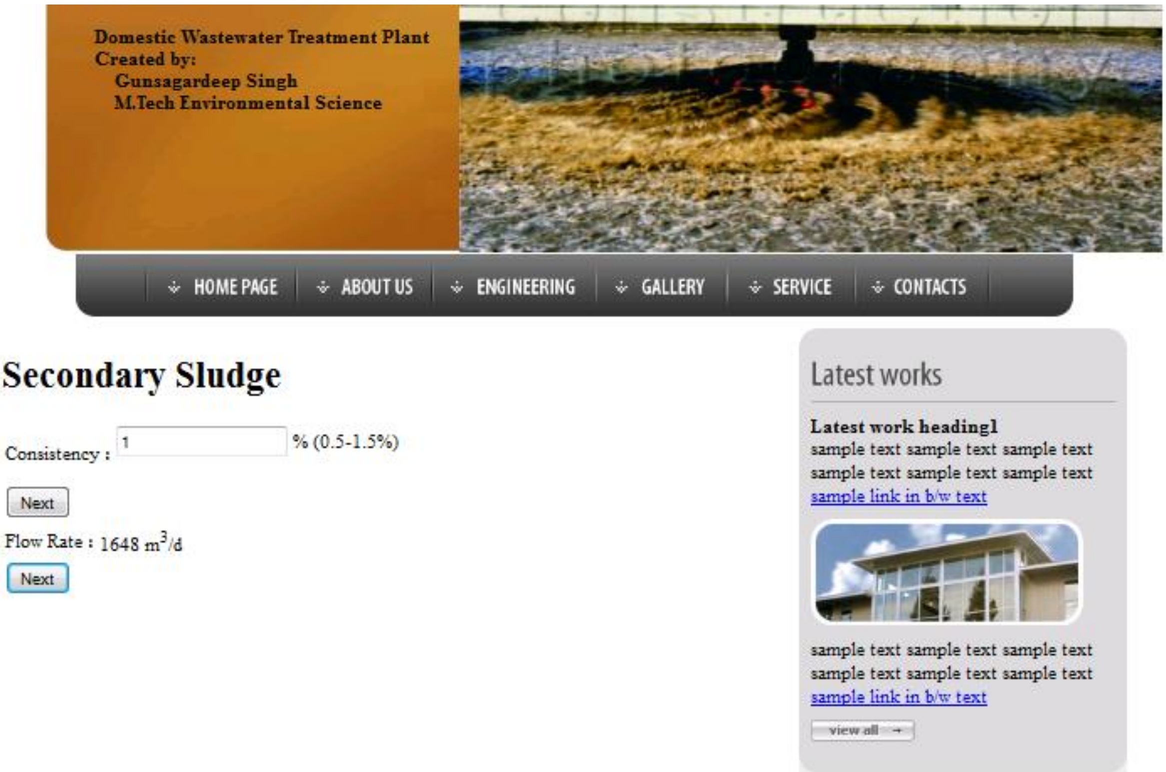

Fig 5.19 Secondary Sludge Flow rate

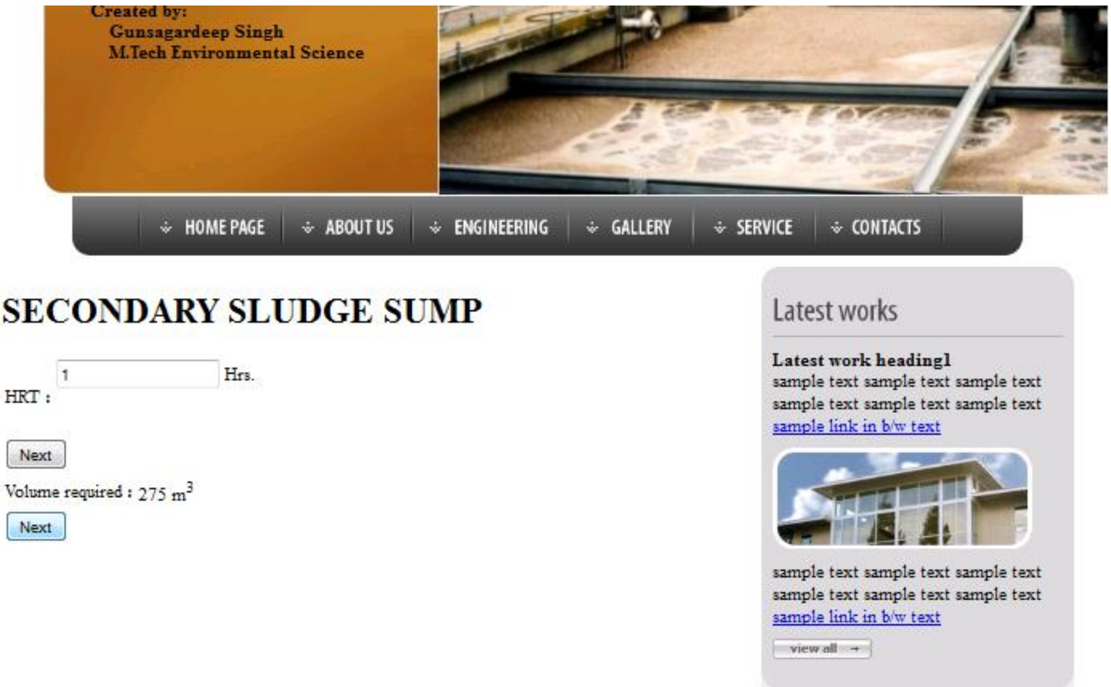

Fig 5.20 Secondary Sludge Sump

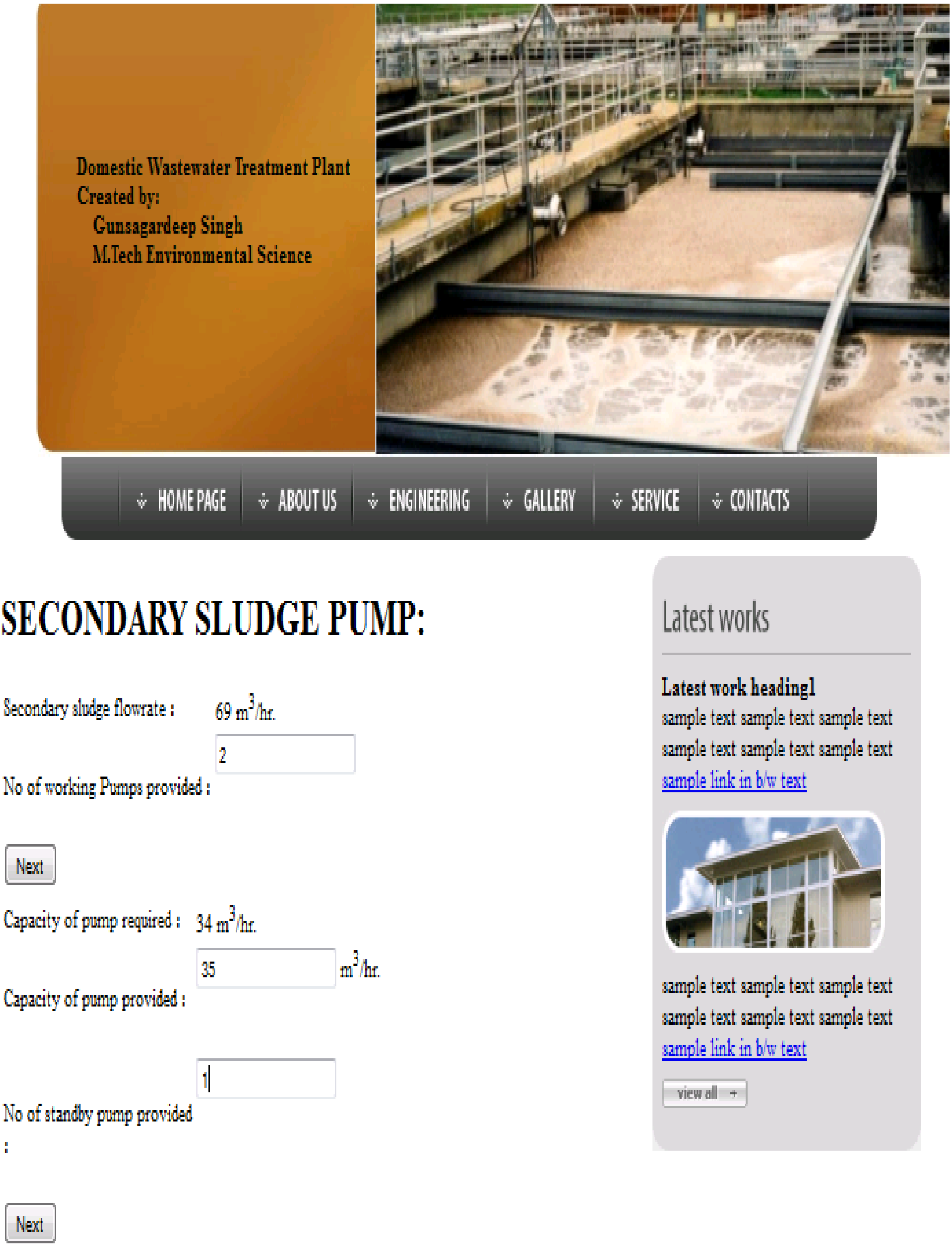

Fig 5.21 Secondary Sludge Pump

On combining both the sludges

Combined Sludge flowrate : 2564 m^3/d
Combined Sludge Quantity : 44874 kg/d
Consistency : 1.68 %
Solid loading Rate : 50 kg/m^2d

Consistency	Solid loading Rate
0.5-1.5%	25-70 kg/m^2d
2-4%	40-80 kg/m^2d

Next

Thickener :

Area of thickener : 897 m^2
Number of thickeners : 2

Next

Area of each thickner : 448 m^2
diameter of each thickner required : 24 m
Diameter of each thickner provided : 25 m

Next

Total area provided : 981 m^2
SWD provided : 3 m
Solid Capture : 90 % (90-98%)
Consistency of thickened Sludge : 5 % (4-6%)

Next

Total solids withdrawn : 40387 kg/d
Total volume of thickened sludge withdrawn : 777 m^3/d
Hydarulic loading desired : 12 $m^3/m^2/d$ (6-12 $m^3/m^2/d$)

Next

Dilution Water : 8200 m^3/d
Total volume of thickner overflow : 9987 m^3/d
Total solids in overflow water : 4733 kg/d
BOD in overflow water : 2742 kg/d

Next

Fig 5.22 Thickener

Fig 5.23 Thickener Overflow Sump

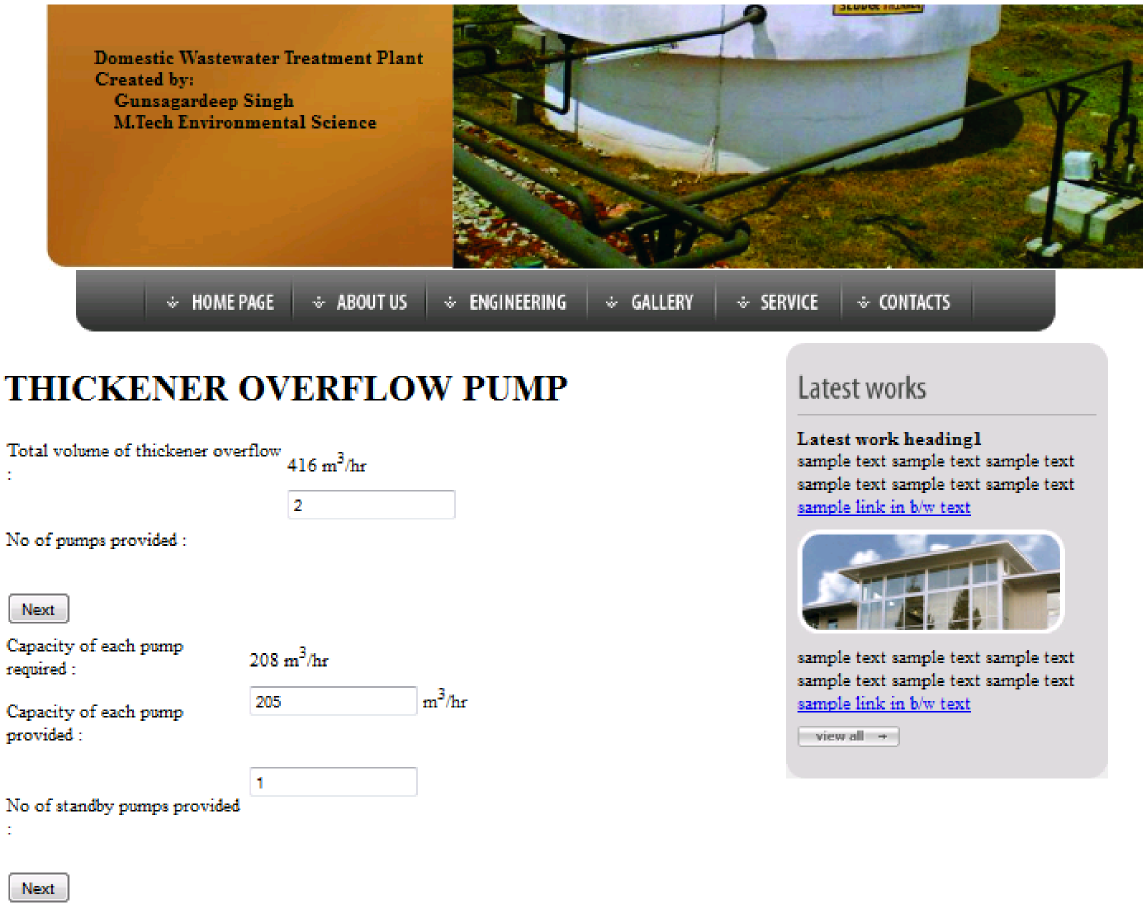

Fig 5.24 Thickener Overflow Pump

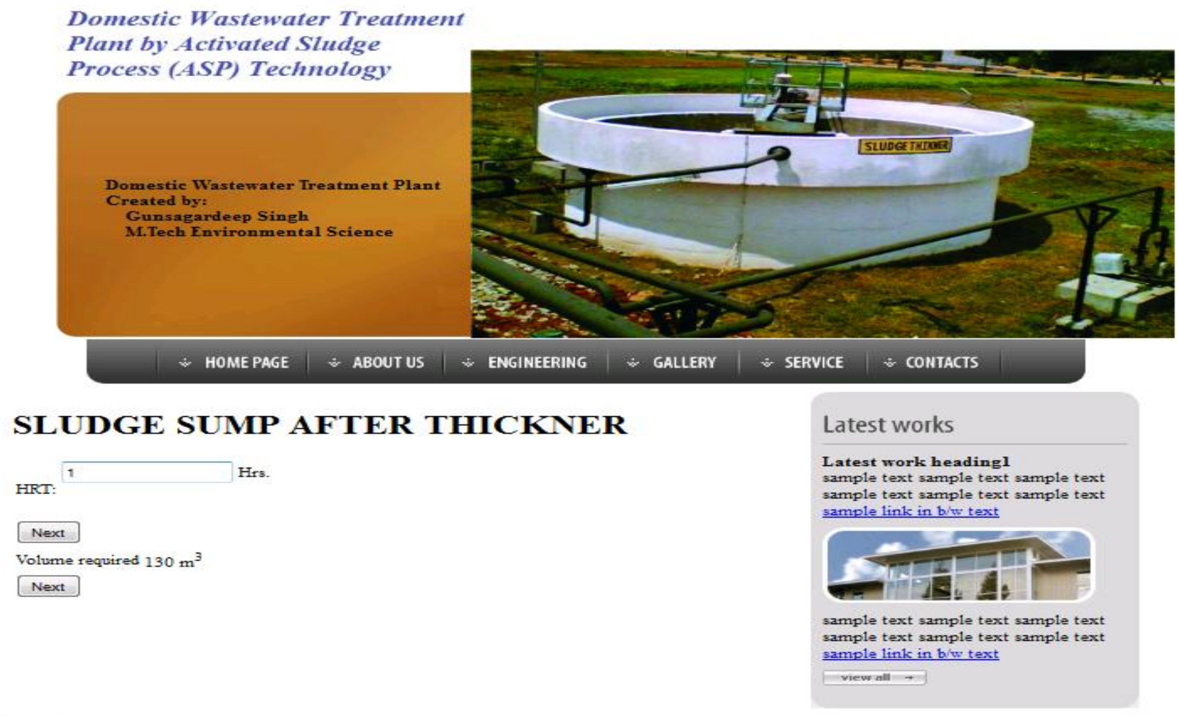

Fig 5.25. Sludge Sump After Thickener

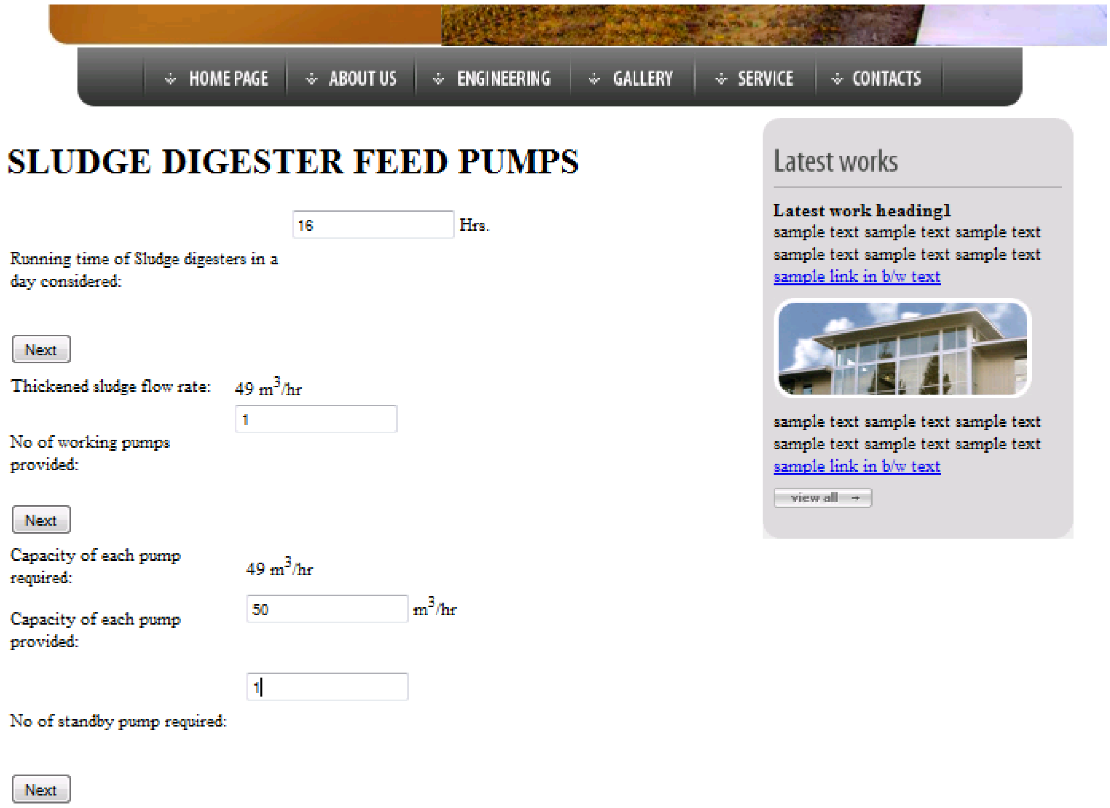

Fig 5.26 Sludge Digester Feed pump

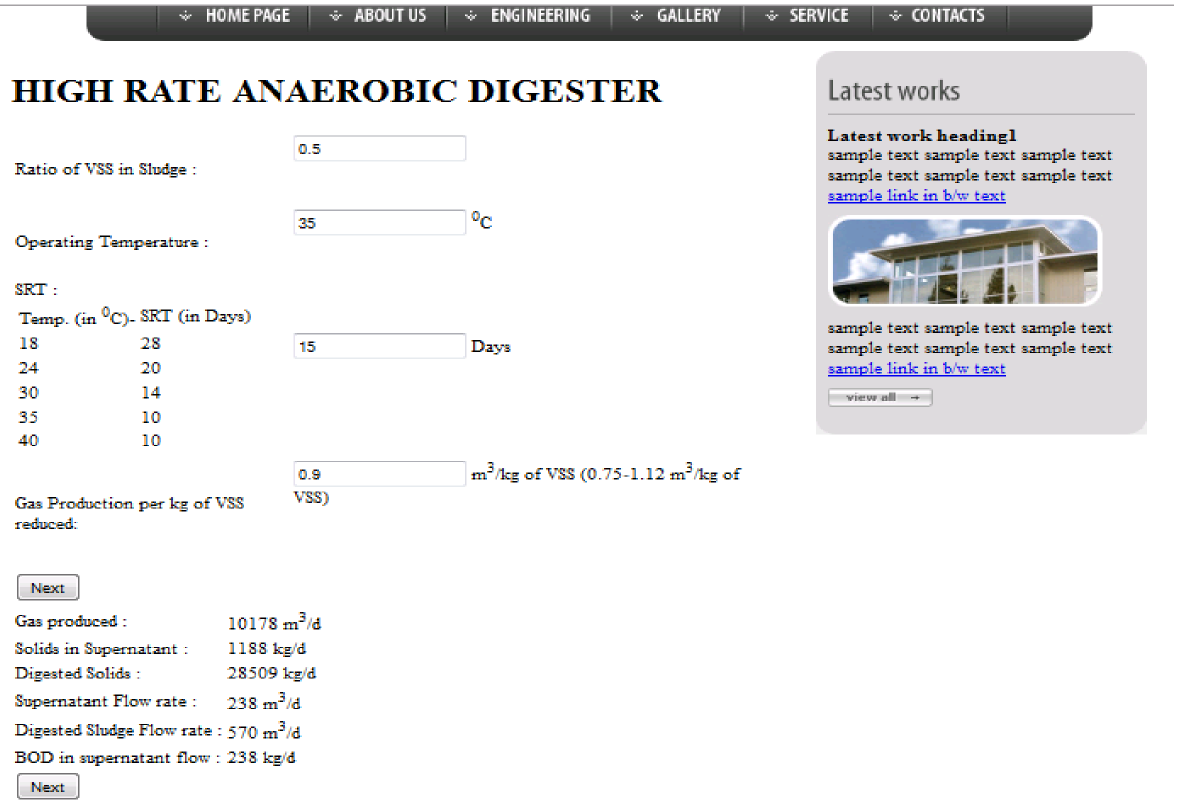

Fig 5.27 High Rate Anaerobic Digester

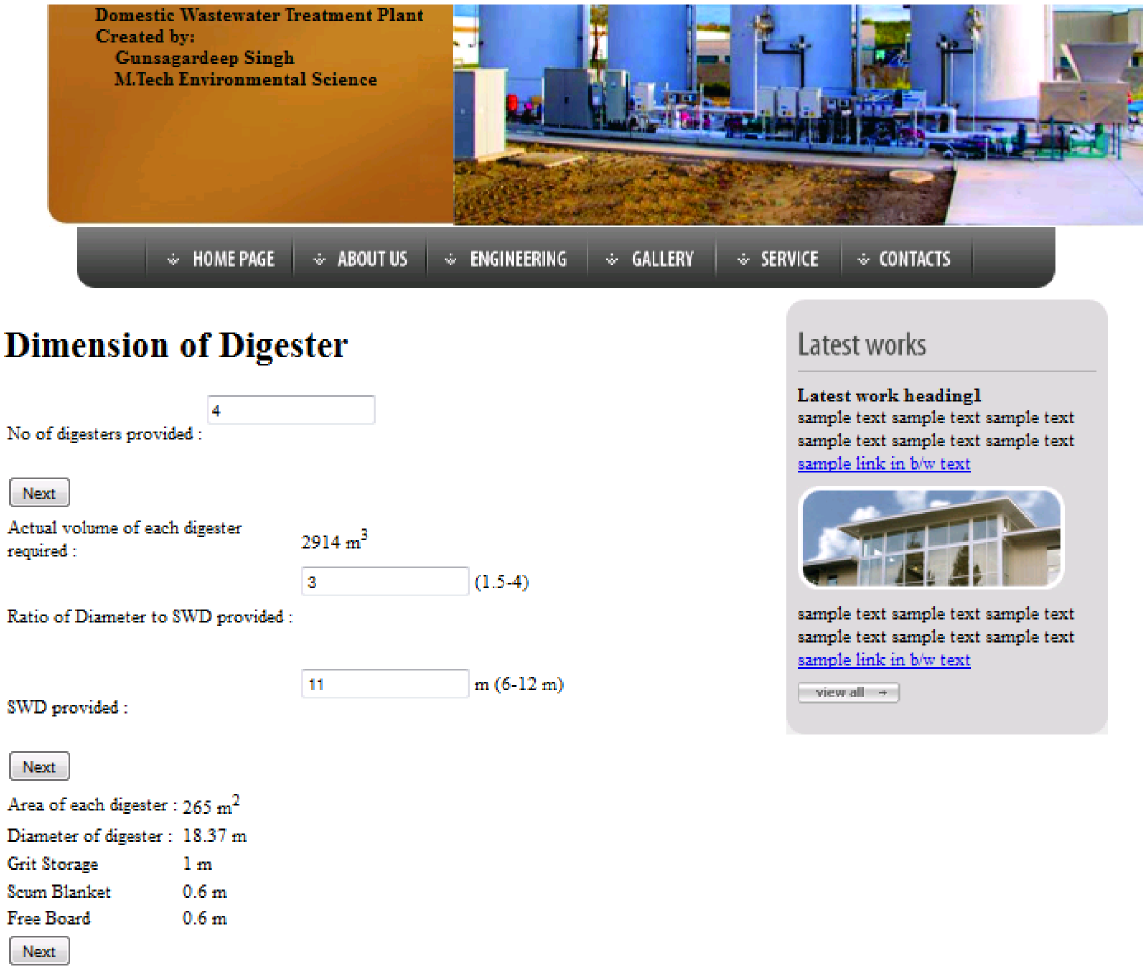

Fig 5.28 Dimension Of Digester

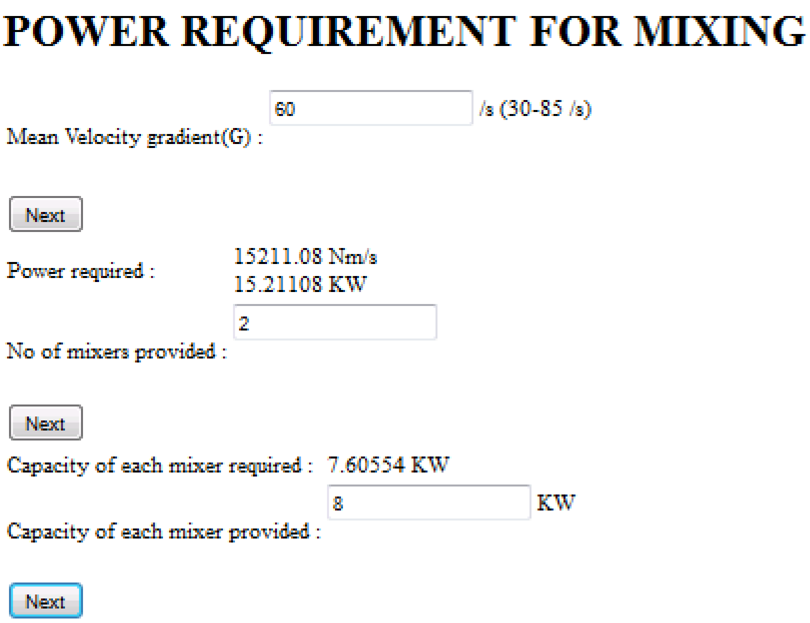

Fig 5.29 Power Requirement For Mixing

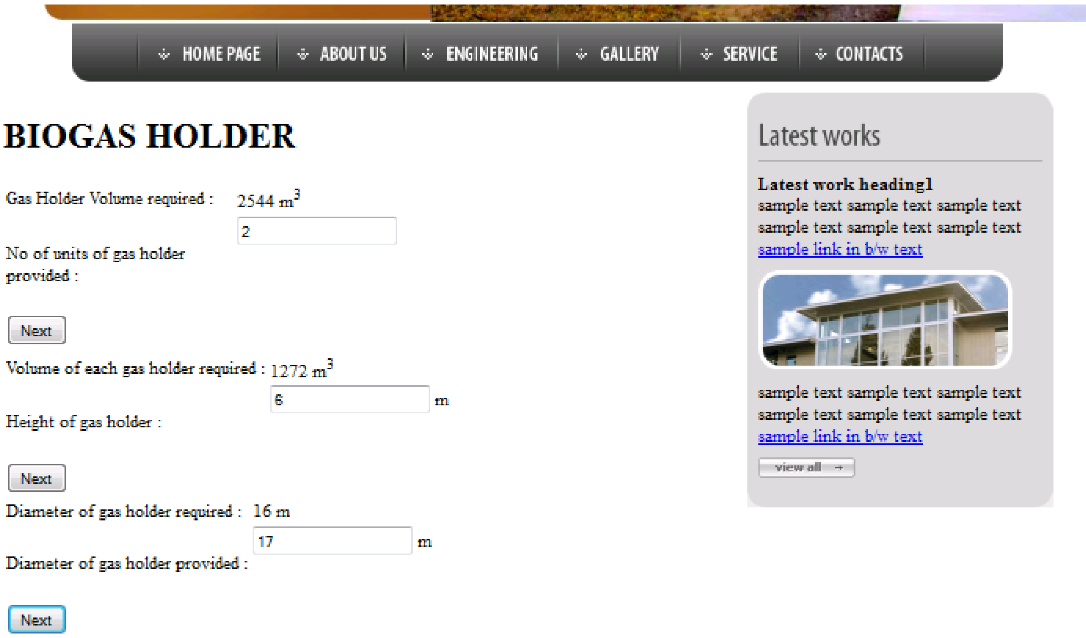

Fig 5.30 Biogas Holder

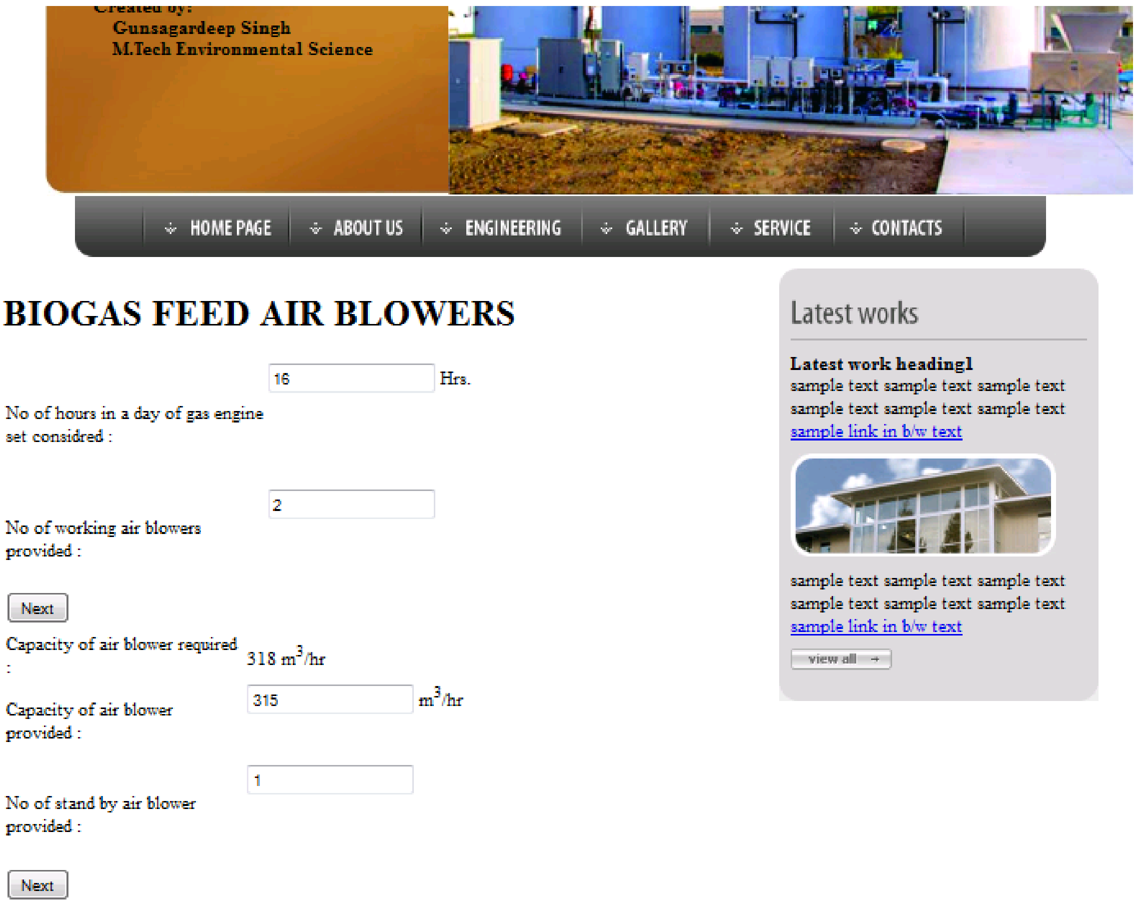

Fig 5.31 Biogas Feed Air Blowers

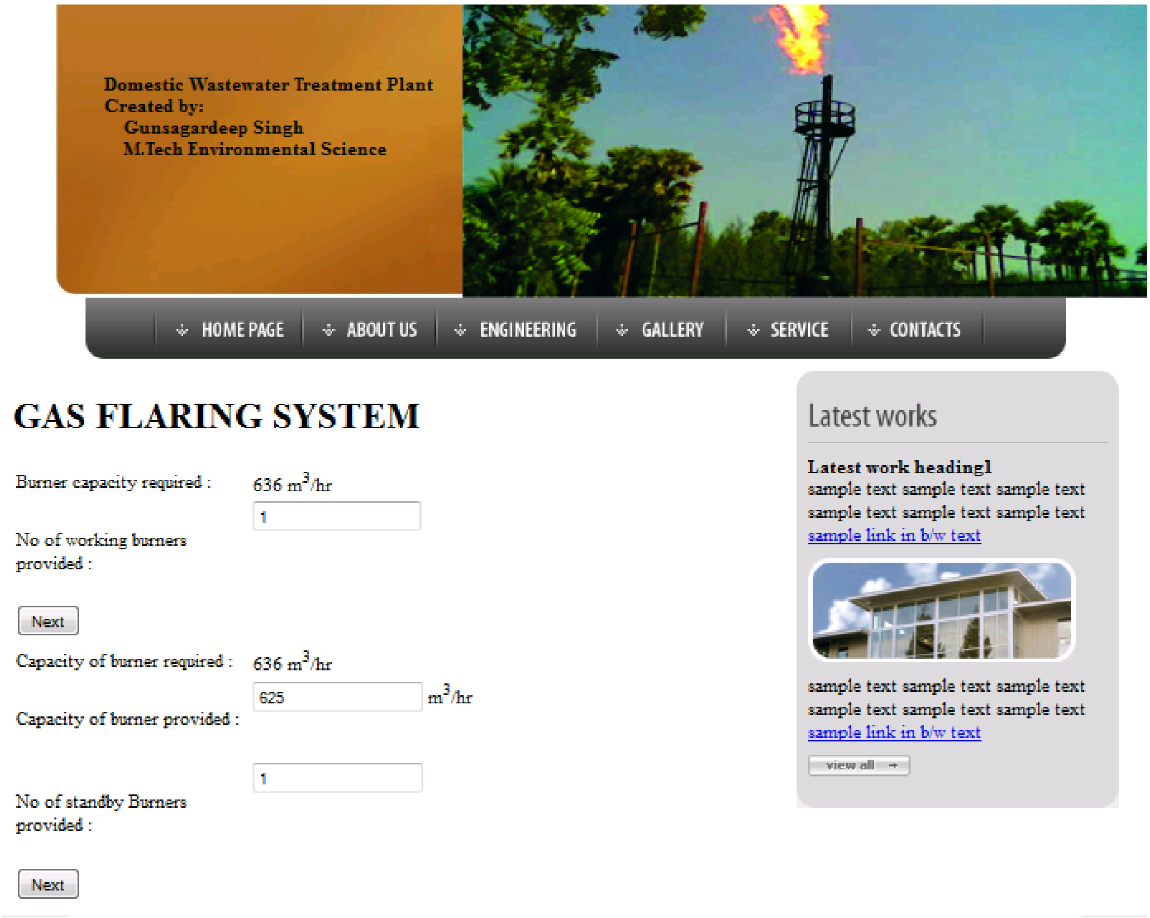

Fig 5.32 Gas Flaring System

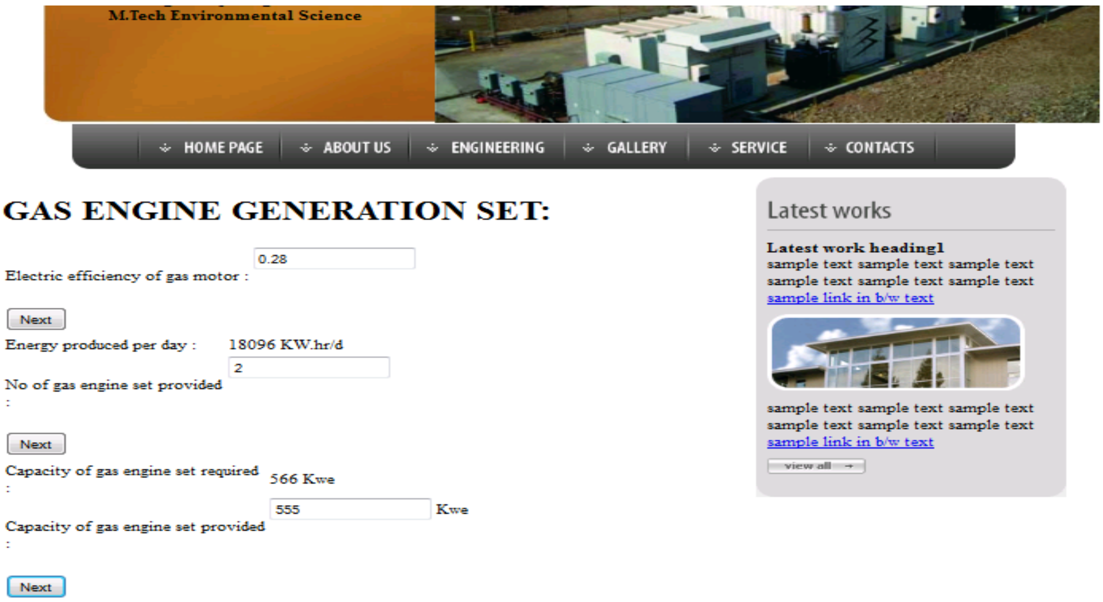

Fig 5.33 Gas Engine Generation Set

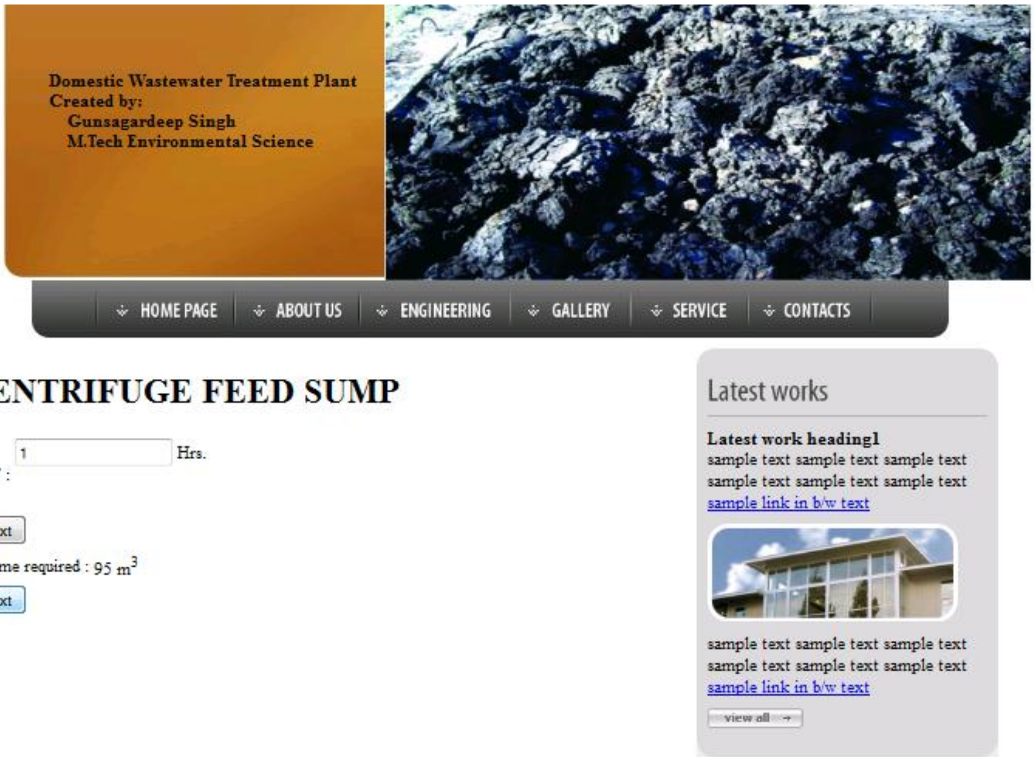

Fig 5.34 Centrifuge Feed Sump

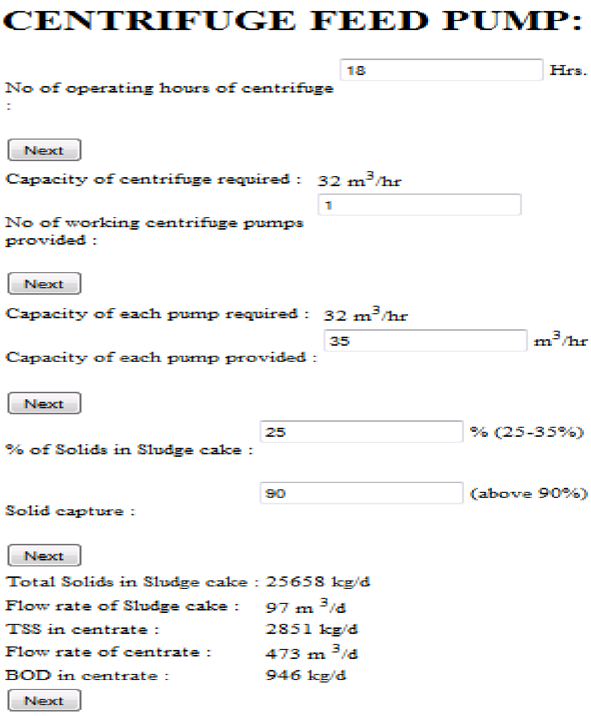

Fig 5.35 Centrifuge & Centrifuge Feed Pump

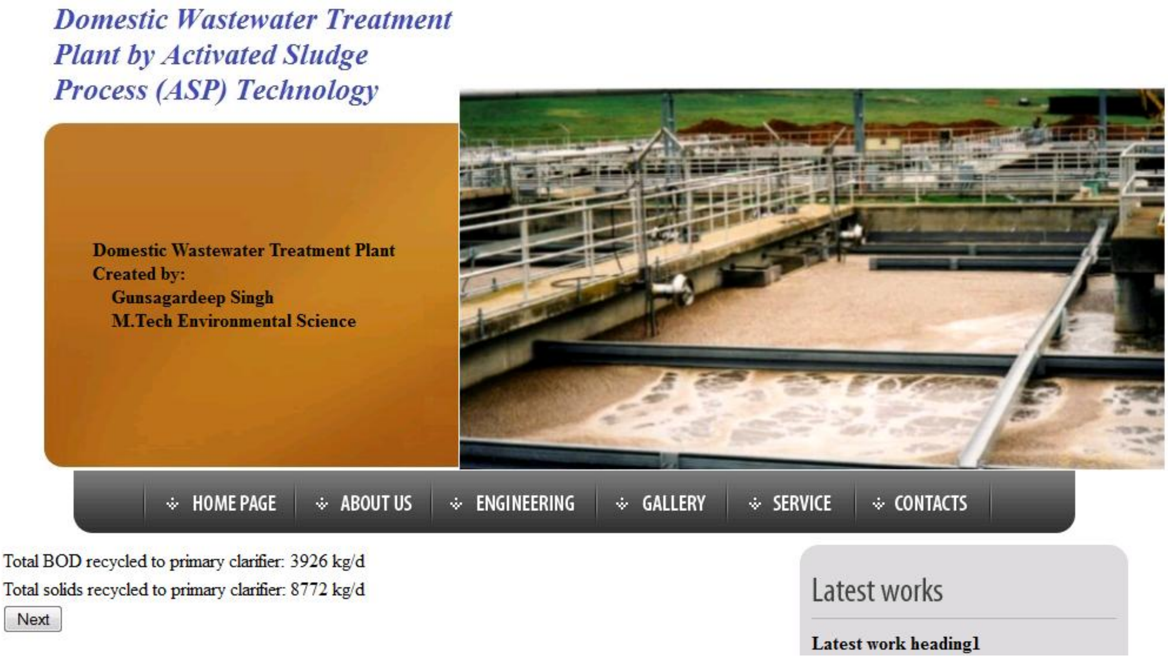

Fig 5.36 BOD and TSS recycled to primary Clarifier

Inlet Chamber:

Volume Provided: 46.9 m^3

Length Provided: 9.38 m

Depth: 2.5 m

Width: 2 m

Free Board: 0.5 m m

Coarse Screening:

Mechanical Screening:

No of screens: 3

width of Screen: 1.53 m

depth of screen: 1.02 m

NO of bars: 50

free board provided: o.3 m

Mannual Screening:

No of screens: 1

width of Screen: 1.53 m

depth of screen: 1.02 m

NO of bars: 50

free board provided: o.3 m

Continue......

Fig 5.37 (a) Results Sheet

RAW SEWAGE SUMP:

Volume of Sump provided : 942 m^3

Diameter of Sump Provided : 20 m

Side Water Depth (SWD) provided : 3 m

Free Board: 0.5 m m

FINE SCREEN:

No of Mechnical (Working) Screens : 3

No of Manual Screens : 1

MECHNICAL SCREENS:

Width of screen : 1.44 m

Depth of screen : 0.97 m

No of bars : 179

Free board : 0.3 m

MANUAL SCREENS:

Width of screen : 1.42 m

Depth of screen : 0.95 m

No of bars : 108

Free board : 0.3 m

Continue......

Fig 5.37 (b) Result Sheet

GRIT CHAMBER:

No of working units : 3

Size of the square basin : 8.88 m

Side water depth : 3 m m

Free Board : 0.3 m

Distribution Chamber:

Volume Requied : 93.75 m^3

Depth of Water : 3 m

width : 2 m

length : 15.62 m

PRIMARY CLARIFIER:

NO of clarifiers : 4

Dia of Clarifier Provided : m

Side Water Depth : 3.5 m

Volume Provided : 3176.11 m^3

Free board : 0.5 m m

Continue......

Fig 5.37 (c) Result Sheet

Aeration Tank:

Side Water Depth: 7.5 m
No. of tanks Provided: 6
Width of each tank: 22.5 m
Length of each tank: 75 m
Total Volume: 75937.5 m^3

Secondary Clarifier

No. of tanks: 6
Daimeter per Clarifier: 31 m
SWD Provider: 3.5 m
Free Board: 0.5 m Volume of each Clarifier Provided: 3176 m^3

Oxygen to be supplied:

AOTR : 24518 kg/d
SOTR : 2245 kg/h
Air flowrate in each Tank : 23240.17 m^3/hr

Secondary Sludge Recycle Pump:

No. of Working Pumps Provided : 1
Capacity of each Pump provided : 35 m^3/hr
No. of Standby pumps provided : 1

Fig 5.37 (d) Result Sheet

Primary sludge Pump

Volume : 38 m^3/d

Primary Sludge Pumps:

No. of Working Pumps Provided : 1

Capacity of each Pump provided : 35 m^3/hr

No. of Standby pumps provided : 35

Secondary Sludge Sump:

Volume : 9m^3

Secondary Sludge pump :

No. of Working Pumps Provided : 2

Capacity of each Pump provided : 35 m^3/hr

No. of Standby pumps provided : 1

Thickener:

No. of Thickener : 2

Daimeter of each Thickener: 24 m

Side water Depth: 3 m

Fig 5.37 (e) Result Sheet

Thickener:

No. of Thickener : 2
Daimeter of each Thickener: 24 m
Side water Depth: 3 m

Thickener Overflow Sump:

Thickner overflow vol: 208 m^3

Thickner overflow Pump:

No. of Working Pumps Provided : 2
Capacity of each Pump provided : 205 m^3/hr
No. of Standby pumps provided : 1

Sludge sump after Thickener:

Volume : 130 m^3

Sludge Digester Feed Pump:

No. of Working Pumps Provided : 1
Capacity of each Pump provided : 50 m^3/hr
No. of Standby pumps provided : 1

Next

Fig 5.37 (f) Result Sheet

HIGH RATE ANAEROBIC DIGESTER

No of digesters provided : 4
SWD Provided : 11 m
Diameter of digester : 18.37 m
Grit Storage : 1 m
Scum Blanket : 0.6 m
Free Board : 0.6 m

BIOGAS HOLDER:

No of units of gas holder provided : 2
Height of gas holder : 6 m
Diameter of gas holder provided : 17 m

BIOGAS FEED AIR BLOWERS:

No. of air Blowers Provided : 2
Capacity of airblower provided : 315 m^3/hr
No. of Standby airblower provided : 1

GAS FLARING SYSTEM :

No of working burners provided : 1
Capacity of burner provided : 625 m^3/hr
No of standby Burners provided : 1

Fig 5.37 (g) Result Sheet

GAS ENGINE GENERATION SET:

No of gas engine set provided : 2

Capacity of gas engine set provided : 13 m^3/hr

CENTRIFUGE FEED SUMP:

Volume : 95 m^3

CENTRIFUGE FEED PUMP:

No. of Working Centrifuge pump Provided : 1

Capacity of each Pump provided : 35 m^3/hr

CENTRIFUGE:

Capacity of centrifuge required : 32 m^3/hr

Effluent TSS from Secondary Clarifier : 27 kg/d

Effluent TSS from Secondary Clarifier : 14 kg/d

Go to daigram

Fig 5.37.(h) Result Sheet

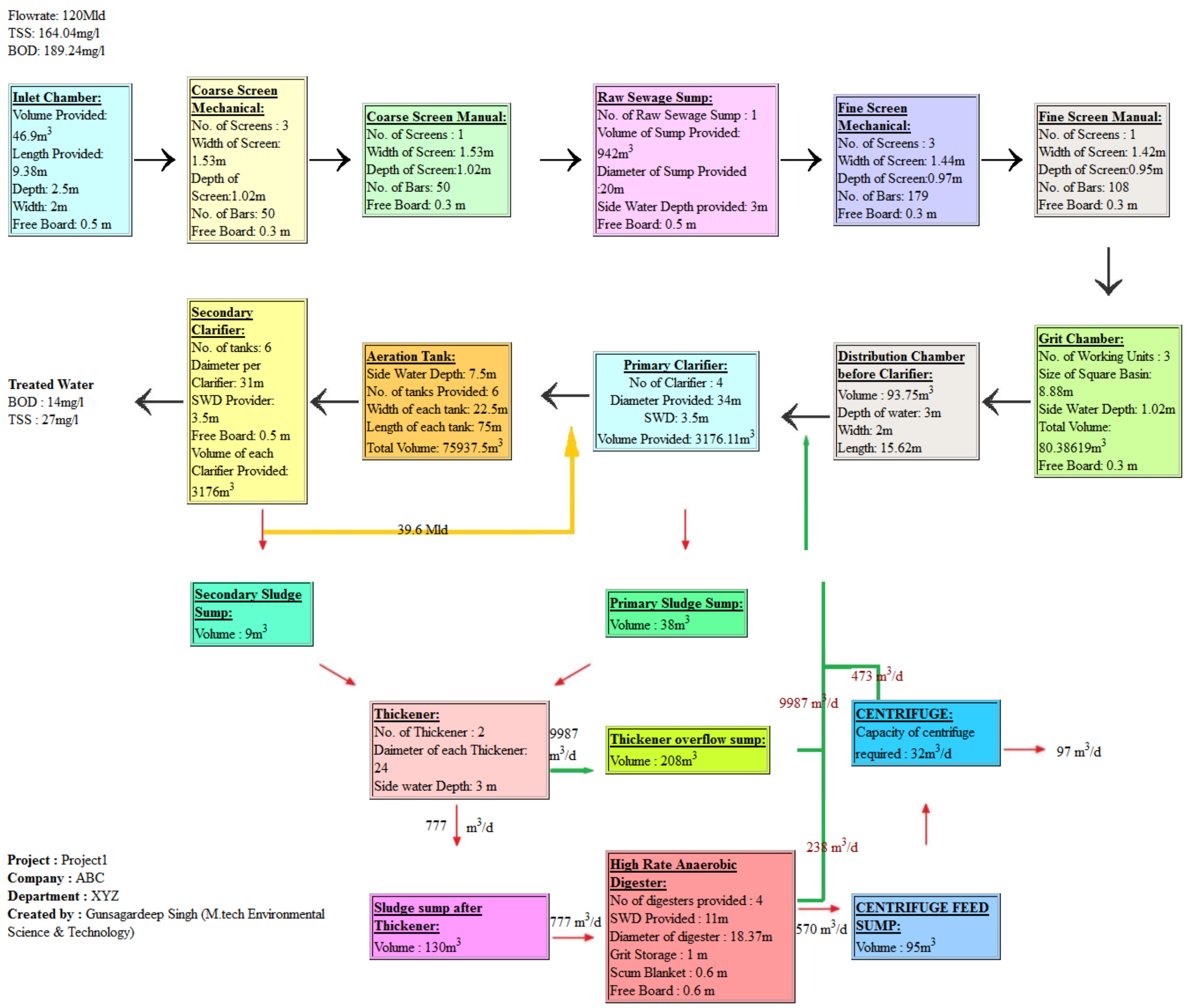

Fig 5.38 Flow Diagram of STP

5.2. Validation

For the validation of the software Nangal sewage treatment plant was tested. The plant is designed to cater for an average flow of 8 MLD (million litres per day) with a peak factor of 2.67.The biological treatment process used in the plant is activated sludge process. The plant is having one inlet chamber through which the wastewater enters the plant, two screen chambers by which debris may be removed and three grit chambers where grit is removed. The influent will be fed into one primary clarifier where suspended particles are removed by settling. The overflow liquid is then sent into a aeration tank where aeration is done by mechanical aerators. The aeration supplies oxygen required for biological decomposition of organic matter present in the liquid. The liquid effluent from aeration tank will be sent to secondary clarifier where the sludge is separated from the liquid. The supernatant from the secondary treatment will have the desired quality suitable for disposal. The sludge is sent for drying in sludge drying beds. The flow diagram of the plant is shown in **Fig 5.39.**

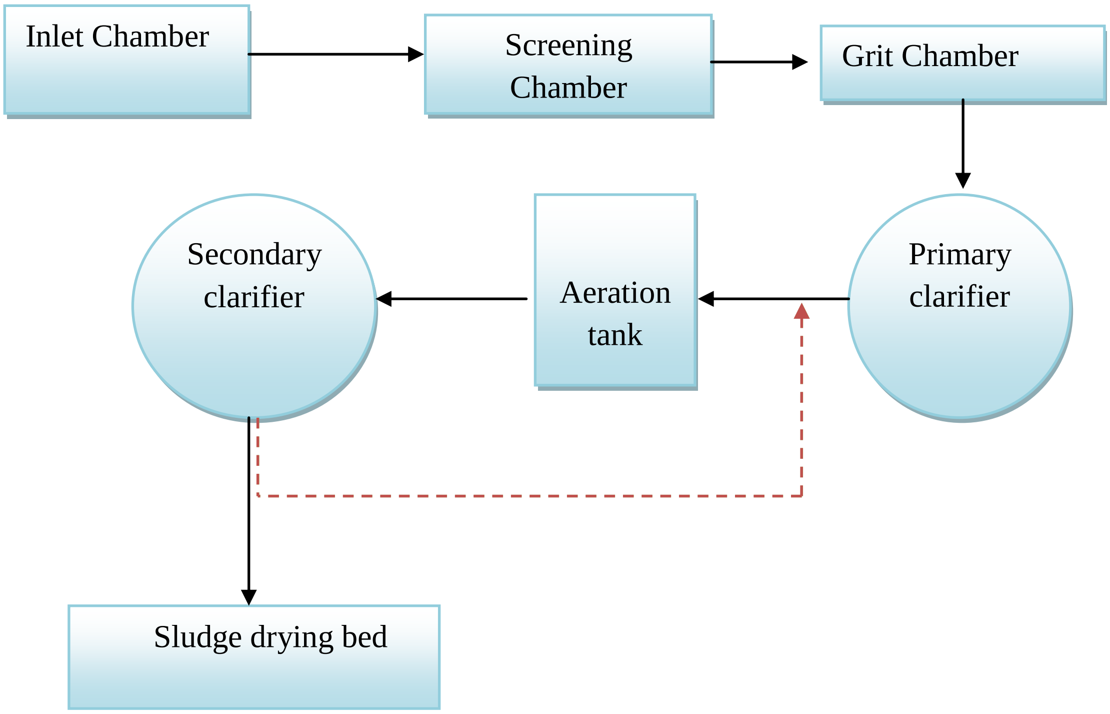

Fig 5.39. Layout of the Nangal sewage treatment plant

Output of the program when validated with Nangal sewage treatment unit:-

5.2.1. Raw Sewage Characteristics

ENTER TSS IN mg/l : 296

ENTER BOD_5 IN mg/l : 96

ENTER TEMPERATURE IN ^oC : 15

ENTER FLOWRATE IN MLD: 8

5.2.2. Inlet Chamber

ENTER PEAK FLOW/AVERAGE FLOW: 2.67

ENTER RETENTION TIME IN SECONDS: 15

ENTER DEPTH IN METRES: 2.5

ENTER WIDTH IN METRES: 2

5.2.3. Coarse Screening

ENTER NUMBER OF SCREENS PROVIDED: 2

5.2.4. Grit Chamber

ENTER NUMBER OF UNITS PROVIDED: 3

ENTER SIZE OF GRIT TO BE REMOVED IN mm: 0.15

ENTER SPECIFIC GRAVITY OF THE GRIT REMOVED: 2.65

ENTER HRT IN SECONDS: 45

5.2.5. Primary Clarifier

ENTER NUMBER OF UNITS PROVIDED: 1

SURFACE LOADING AT PEAK FLOW IN $m^3/m^2/d$: 80

SURFACE LOADING AT AVERAGE FLOW IN $m^3/m^2/d$: 35

SIDE WATER DEPTH PROVIDED IN METRES: 3.5

5.2.6. Aeration Tank

ENTER DESIRED MLSS IN g/m^3: 4500

ENTER NUMBER OF UNITS: 1

ENTER SIDE WATER DEPTH IN METRES: 3.5

5.2.7. Secondary Clarifier

ENTER TSS IN SLUDGE IN g/m^3: 11000

ENTER NUMBER OF UNITS: 1

ENTER SURFACE OVERFLOW RATE PROVIDED IN $m^3/m^2/d$: 35

ENTER SWD PROVIDED IN METRES: 3.5

The influent sewage characteristics of Nangal plant is shown in **Table 5.1.** .The results of the test-run of the program are summarized in the **Table 5.2**. The specifications of the existing plant were also given for comparison and validation. It was evident from the table that the design details obtained from the output of the program are in close proximity with those of the existing plant, hence the usefulness and the authenticity of the software are proved and verified. To further check the validation of the software the performance characteristics and efficiency of the individual units were studied. Removal efficiencies of the individual units in the wastewater treatment plant were analysed from the data obtained from the Nangal sewage Treatment plant and are given in **Table 5.3**.

Table 5.1 Influent Sewage Characteristics

S.No.	**Influent Wastewater Parameter**	**Value**
1	BOD_5	96 mg/l
2	TSS	226 mg/l
3	Temperature	15 ^{0}C
4	Flowrate	8 Mld

Table 5.2 Comparison between different units calculated by software and existing plant

S. No.	**Name of the units**	**Design details obtained from Software**	**Design details of the Existing plant**
1	Screen Chamber	Width = 0.48 m Depth = 0.32 m	Width = 3.2 m Depth = 0.70 m
2	Grit Chamber	Length = 2.83 m Width = 2.83 m Depth = 1.2 m Volume = 9.5 m	Length = 15 m Width = 0.9 m Depth = 0.65 m Volume = 9 m

3	Primary Clarifier	Diameter = 18 m Depth = 3.5 m Volume = 800 m^3	Diameter = 20 m Depth = 3 m Volume = 940 m^3
4	Aeration Tank	Volume of aeration tank =1147 m^3	Volume of aeration tank =1134 m^3
5	Secondary Clarifier	Diameter = 22 m Depth = 3.5 m	Diameter = 20 m Depth = 3.5 m

Table 5.3 Comparison between BOD and TSS value calculated by software and existing plant

S. No.	Effluent Wastewater parameter	Values obtained from software	Values obtained from existing plant
1	BOD	17 mg/l	9 mg/l
2	TSS	33 mg/l	24mg/I

It was found that the efficiency of the treatment plant was also matching the expected values with the variation of less than 5 % hence the software is valid, and it can be readily used in the field to design STP with activated sludge process for treating municipal waste from large cities.

Conclusion

Sewage treatment plays an important role in control of water pollution from different sections of the society. Proper design, operation and maintenance of STP gives good removal efficiency of pollutants. Manual design of large scale wastewater treatment plants is cumbersome and time consuming. Computer aided programming can do all iterations with accuracy and with little time. A computer program in ASP.NET was used for interactive computer aided design of sewage treatment plant design. The program can be used for the design of any wastewater treatment plant which is having activated sludge process as biological process with relevant input data. The program can also be used to check the design details of an existing plant to know the expansion works needed for increased hydraulic and organic loadings occurring in future. A sewage treatment plant located in Nangal was considered for validation of computer aided design. The existing plant data was used for verifying the software's authenticity. It was observed that the design values obtained from the program are matching the design values of the existing plant and hence concluded that the program can be utilised for designing. The performance of the existing plant is also evaluated by using the relevant data obtained from the plant and it was found to be satisfactory.

The program also includes sludge handling system and power generation system. The sludge is stabilized in digester and the biogas produced in the stabilization process is used up in power generation in gas engine set. So the design is not only efficient but also environmental friendly since the sludge is also utilised in making electricity which helps in running the pumps and other processes where electricity is required. Pumps and sumps, the oxygen required in the aeration tank and power required for mixing in the high rate anaerobic digester are also designed.

The software is unique in itself as it is not to be installed in the laptop or computer but is globally available online and only remembering its URL (www.oberoiinfra.co.in) is required to use it so the user need not to carry his laptop everywhere to use it. This software can also work on mobiles through opera mini.

REFERENCES

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Appendix-A

Abberiviations And Notifications

AI	Artifical Intelligence
AOTR	Actual Oxygen Transfer Rate
ASM	Activated Sludge Model
ASP	Activated Sludge Process
BEEP	Bioenergy Engineering Education Program
BNR	Biological Nutrient Removal
BOD	Biological Oxygen Demand
bVSS	Bio Degradable Volatile Suspended Solids
C	Operating Oxygen Concentration
CAD	Computer Aided Design
$CaCO_3$	Calcium Carbonate
CFD	Computational Fluid Dynamics
CH_4	Methane
CNG	Compressed Natural Gas
C/N	Carbon / Nitrogen
CO	Carbon Mono-Oxide
CO_2	Carbon d cvvvvvvvvioxide
COD	Chemical Oxygen Demand
C/P	Carbon / Phosphorus
$C_{s\_T,H}$	Average dissolved oxygen saturation concentration in the clean water in aeration tank at temp.T and altitude H.

$C_{s,20}$	Dissolved oxygen saturation concentration in the clean water at 20^{o}C and 1atm.
DO	Dissolved Oxygen
DT	Detention Time
ESS	Effluent Suspended Solid
F	Fouling Factor
F/M	Food to Microbes ratio
f_d	Cell debris
G	Mean Velocity Gradient
H_2	Hydrogen
H_2S	Hydrogen Sulphide
HRT	Hydraulic Retention Time
IWA	International Water Association
iTSS	Inert Total Suspended Solid
k_d	Endogenous decay rate
k_{dn}	Endogenous decay coefficient for nitrifying orgamisms
k_o	Half Saturation Coefficient for Do
K_n	Half –Velocity constant for nitrifying organisms
K_s	Half- Velocity
MLSS	Mixed Liquor Suspended Solid
MLVSS	Mixed Liquor Volatile Suspended Solid
MPC	Model Predictive Control
MSW	Muncipal Solid Waste
MW	Microwave Treatment
N	Nitrogen Concentration
N_R	Reynolds Number

$Na(HCO_3)$	Sodium Bicarbonate
NDBEPR	Nitrification Denitrification Biological Excess Phosphorus Removal System
NH_3	Ammonia
NH_4-N	Ammonical Nitrogen
NO_3	Nitrate
No_x	Nitrate Nitrogen
nbVSS	Non-biodegrable volatile suspended solid
O_2	Oxygen
OLR	Organic Loading Rate
OSMP	Optimal Sludge Management Program
OUR	Oxygen Uptake Rate
OOP	Object Oriented Program
P	Phosphorus
$P_{atM,H}$	Atmospheric pressure at altitude H
P_b/P_a	Change in atmospheric pressure with elevation
PEIO	Primary Energy Input To Output
PO_4	Phosphate
$P_{x,bio}$	Biomass as VSS per day
$P_{x,VSS}$	Net waste activated sludge produced each day in terms of volatile suspended solids
$P_{x,TSS}$	Net waste activated sludge produced each day in terms of total

	suspended solids
Q	Flowrate
R or RAS	Return Activated Sludge
S	Effluent Substrate Concentration
S_o	Influent Substrate Concentration
SO_4	Sulphate
SOTR	Standard Oxygen Transfer Rate
SRT	Solid Retention Time
SS	Suspended Solid
SWD	Side Water Depth
S_X	Siloxanes
TKN	Total Kjehldahl Nitrogen
TN	Total Nitrogen
TOD_{nit}	Theoretical Oxygen Demand For Nitrification
TP	Total Phosphorus
TSS	Total Suspended Solid
VFA	Volatile Fatty Acid
VSS	Volatile Suspended Solid
WWTP	Wastewater Treatment Plant
XS	Slowly Biodegrable
Y	Yield Constant
Y_n	Net biomass yield in nitrification

α	Oxygen transfer correction factor for waste
β	Salinity –surface tension correction factor
η	Desired coefficient of removal of grit particle
μ	Dynamic Viscosity
μ_n	Specific growth rate for nitrifying bacteria
$\mu_{N.m}$	Maximum specific growth rate of nitrifying bacteria
μ_m	Maximum specific bacterial growth rate
ν	Kinematic Viscosity