Design and Cost comparison of extended aeration and SAFF for typical domestic waste water

A

DISSERTATION

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Under the Guidance of

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Certificate

This is to certify that the thesis entitled **"Design and cost comparison of extended aeration and SAFF for typical domestic waste water"** submitted by Nagesh Vikram Singh in partial fulfillment of the of the requirement for the award of Degree of Masters of Technology in Environmental Science and Technology to Thapar University, Patiala, is a record of student's own work carried out under the guidance and supervision of Dr. Anita Rajor & Mr. Somesh Rastogi. This report has not been submitted for the award of any other degree or certificate in this or any other University or institute.

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

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(Nagesh Vikram Singh)

Abstract

In the present study we have done the feasibility study of two different technologies viz. extended aeration and SAFF, used for the treatment of sewage.

Before comprising the techno-economical feasibility of two different technologies, we have designed STP with both the technologies. Design basis of STP for a typical domestic waste water at the flow rate 100 KLD. BOD of untreated effluent and treated water is 300 and 30 mg/l respectively.

Cost estimation of each individual unit is based on the consultation with different suppliers and technically associated personnel of various organizations. The economical feasibility is dependent on civil and mechanical cost of each of the unit and both these factors are governed by current market values.

The basic design (O&G trap, bar screen, Equalization tank, Primary clarifier, Secondary clarifier and Sludge drying bed) is same for both the technologies except aeration tank and SAFF reactor. We estimated the civil cost as Rs 1042900 and Rs. 951650 for aeration tank and SAFF reactor respectively, while mechanical cost as Rs. 195064 and Rs.203189 respectively. So we can say that the cost can be reduced if we use SAFF technology for the treatment of typical domestic waste water.

On the basis of present study we observed that media provide more surface area to microorganisms for their rapid growth. Volume of SAFF reactor is reduced by 1/5 than that of extended aeration. MLSS concentration and DO maintenance dose not require for SAFF. These observations prove that SAFF technology is more techno-economical than extended aeration.

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Chapter 1: Introduction

Sewage treatment is the process that removes the majority of the contaminants present in sewage so as to produce an effluent and sludge which can be disposed of in the environment without causing health hazards or nuisance.

Sewage is the liquid waste from toilets, bath, shower, kitchen, etc. that is disposal of via sewar. In many areas sewage also includes some liquid waste from industry and commerce. In many countries, the waste from toilets is termed foul waste, the waste from items such as a basins, bath and kitchen is termed a sullage water and industrial and commercial waste is termed trade waste. Sewage are normally offensive and dangerous to the public health, it is necessary to the study the characteristic of sewage. This study is also help for pollution control of water course in to which sewage is discharge with or without treatment.

Discharge of untreated sewage is single most important cause for pollution of surface and ground water because there is a large gap between generation and treatment of domestic wastewater in India. Central Pollution Control Board in association with a consultant carried out a study for performance evaluation of the STPs in India.

The treatment method shall be selected based on the characteristics of the sewage and its treatability with anticipation of growth in usage.

Depending on unit operation and unit process employed, sewage treatments are classified follows

1- Preliminary treatment
2- Primary treatments
3- Secondary treatments
4- Tertiary treatment

The first three treatments viz., preliminary treatment, primary treatment and secondary treatment are often grouped together and termed as conventional treatment, while the last one viz., tertiary treatment is known as advanced treatment of sewage.

In preliminary treatment consist of removing from sewage such floating material as dead animals, wood pieces, tree branches heavy settable inorganic solid, fat, oil and grease. .

Primary treatment involves the physical separation of suspended and colloidal material from a water or waste water stream. In many cases it may also involve chemical coagulation and flocculation to enhance removal efficiencies. This is usually accompanied in plane sedimentation in plan sedimentation tank also known as primary clarifier.

Secondary or biological treatment involve removal of organic matter and the residual suspended material, and the residual suspended, material, and is generally accomplished by using biological unit processes-

(A) Aerobic process-

1- Suspended growth
2- Attached growth

Suspended growth processes commonly known as activated sludge process. Aerobic suspended growth system are two basic type, those which employ sludge recirculation, viz., conventional activated sludge process and its modification and those which do not have sludge recirculation.

(B) Anaerobic process

1- Suspended growth
2- Attached growth

(C) Combined process (aerobic and anaerobic)

In normal practice sewage treatment is confined up to secondary treatment only. Tertiary treatment or advance treatment is adopted when reuse of effluent for other purpose.

Now-a-days a lot of technologies are used for the treatment of sewage but the main problem is practical and economical feasibility of different technologies. In the present study, we compared two feasible technologies viz. extended aeration and SAFF and found that SAFF technology is more feasible in both the prospects.

2.1 Sewage-

"Sewage is the mainly liquid waste containing some solids produced by humans which typically consists of washing water, faces, urine, laundry waste, storm water and other material which goes down drains and toilets from households and in industry". (www.wikipedia.org)

2.2 Composition of untreated typical domestic waste water

Table - 1 composition of untreated typical domestic waste water

S. No.	Constituent	Unit	Concentration		
			Strong	**Medium**	**Weak**
1-	Solid , total(TS)	mg/l	1230	720	390
2-	Dissolved ,total(TDS) Fixed Volatile	mg/l	860 520 340	500 300 200	270 160 110
3-	Suspended solid, total Fixed Volatile	mg/l	400 85 315	210 50 160	120 25 95
4-	Settle able solid	mg/l	20	10	5
5-	Biochemical Oxygen Demand 4- day, 20°C,(BOD5, 20°C)	mg/L	350	190	110
6-	Total organic carbon (TOC)	mg/L	260	140	80
7-	Chemical oxygen demand (COD)	mg/L	800	430	250
8-	Nitrogen (total as N) Organic Free ammonia Nitrite Nitrate	mg/L	70 25 45 0 0	40 15 25 0 0	20 8 12 0 0
9-	Phosphorus (total as P) Organic Inorganic	mg/L	12 4 10	7 2 5	4 1 3
10-	Chloride	mg/L	90	50	30
11-	Sulfate	mg/L	50	30	20
13-	Oil and grease	mg/L	100	90	50
14-	Volatile organic carbon	mg/L	>400	100-400	<100
15-	Total coliform	No./100 mL	10^7- 10^{10}	10^7-10^9	10^6-10^8
16-	Fecal coliform	No./100 mL	10^5-10^8	10^4-10^6	10^3-10^5

Source: Techobanoglous et.al 2007

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2.3 Process description

2.3.1 Primary treatment

2.3.1.1 Screening

Screening is the first operation carried out at the sewage treatment plant. It consist of passing sewage through a screen so as to trap and remove floating materials present in sewage which would collect otherwise clog and damage pumps and other equipment, interfere with the satisfactory operation of treatment units or equipment, or cause objectionable shore line condition where disposal in to sea is practiced. A screen is a device with opening generally of uniform size. The screen element may consist of parallel bars or flats rods or wires, grating, wire mesh or perforated plate and the opening may be of any shape but generally they are circular or rectangular. (*"Techobanoglous et.al 2007.*

The bars or flat or rods used for bar screen are of rectangular trapezoidal section placed vertically or inclined at a slope varying from 30 to 80 with horizontal or curve and spaced at a closed and equal interval across a chamber or a channel through which sewage flows. The function performed by screen is called screening and the materials removed by it are known as screen or raking. *("Manual on sewage and sewage treatment"2007)*

A. Design consideration for screen and screen channels –

A.1 Velocity of flow-

The various considerations involved in the design of screen and screen channel are as follows:

The velocity of flow ahead of and through a screen various material and affect its operation. The velocity of flow produced through the screen opening does not exceed the 0.9m/s at the maximum or peak flow as per IS: 6280-1971. However, velocity of 0.6 to 1.2m/s through screen for the maximum or peak flow has been found satisfactory.

When considerable amount of storm water (or rain water) are to be the handled, approach velocity of about 0.8m/s is desirable to avoid grit deposition at the bottom of screen, through lower value 0.6 m/s is used in current practice.*("Manual on sewage and sewage treatment" 2007)*

A.2 Loss of head –

As the sewage flows the through a screen certain amount of loss of head take place. The loss of head through bar screen is a function of (i) bar shape, and (ii) velocity head of the following equation for head loss:

$$h_L = \beta(w/b)^{4/3} h_v \sin\theta \qquad (eq^n\text{-}1)$$

h_L= head loss in (in m)

β =bar shape factor

w= maximum cross-sectional width of bars facing direction of flow (in m)

b= minimum clear spacing of bar (in m),

h_V =velocity head ($V^2/2g$)of flow approaching screen (in m)

V =velocity of flow through the screen (in m/s)

θ = angle of inclination of screen with horizontal.

Kirschmer' s value of β for several shape of bars are given in table-2

The head loss calculated from equation -1 applies only when the bars clean. Head loss increases with the degree of clogging.

Table -2: Bar shape factors for different bar types

S. No.	Bar type	β
1	Shape edge rectangular	2.42
2	Rectangular with the semicircular upstream face Circular	1.83
3	Rectangular with the semicircular upstream and	1.79
4	Downstream and downstream faces.	1.67

Source: Sewage treatment & disposal and waste water 2007.

Another equation used for computing the head loss through clean flat bar screen is as follow.

$$h_L = .0729(V^2 - v^2) \qquad (eq^n\text{-}2)$$

h_L= head loss (in m)

V = velocity of flow through screen (in m/s) and

v =velocity of flow before the screen (in m/s)

Usually accepted practice is to provide loss of head of 0.15 m but the maximum loss of head with the clogged hand cleaned screen should not exceed 0.3 m for the mechanically cleaned screen the head loss is specified by the manufactures.

The head loss through fine screen may be calculated by means of the common orifice formula.

$$h_L = 1/2g\ (Q/CA)^2 \qquad (eq^n\text{-}3\ )$$

h_L= Head loss (in m),

Q = discharge through screen (in m^3/s),

A =effective submerged open area of screen (in m^2),

g =acceleration due to gravity (in m/s^2)

C =coefficient of discharge

Value of C and A depend on screen design factor such as size and milling of slot, the wire diameter and weave and particularly the percent of open area and must be determine experimentally .A typical value of C for clean screen is 0.60 . The head loss through clean screen is relatively in significant.

A.3 Materials

Bar screens are made of steel bars flats or rods fixed to a suitable frame. The minimum cross- section of bar screen is10mm×50mm and are placed with large dimension parallel to the direction to flow. They have perforated perforations or opening of size 1.5 to 3 mm.

A.4 Other considerations

i) The top of the screen is provided at least 300mm above the highest flow level of sewage.

ii) Large STP screen channel may be suitably divided in to have maximum screen width of 1.5 m.

iii) The sub merged area screen including bars and opening should be about 25 to 35 percent in excess of cross sectional area of approach channel.

iv) In the case of hand cleaned screen minimum drop of 150 mm should be provided and in the case of mechanically operated screen a minimum drop of 75mm should be provided.

v) The length of the screen channel should be sufficient so that the screen can be properly housed; enough working space available flow gets stabilized and eddies is avoided. The length of screen channel is given by the formula:

$$L = (d + .03) \cot \theta + 1.73(W + d_s) \qquad (eq^n\text{-}4)$$

L= length of the screen channel (in m);

d = depth of flow in screen channel (in m)

θ = angle of inclination of screen with horizontal

W = width of screen (in m) and

d_s = diameter of incoming sewer

vi) A minimum free-board of 300 mm should be provided which may suitably rise where required by turbulent condition in channel.

A summary of the design information for bar screens or bar racks is given below:

1. Minimum bar size	= 10 mm × 50 mm
2. Clear spacing between bars	= 15 mm to 75 mm
3. Slope with horizontal	
(a) Manually cleaned	= 45° to 60°
(b) Mechanically cleaned	= 60° to 90°
4. Minimum approach velocity	= 0.3 m/s
5. Velocity of flow through screen openings	= 0.6 m/s to 1.2 m/s
6. Allowable head loss	= 0.15 m

The Indian Standard IS: 6280- 1971 give detailed specification for sewage screens. *("Manual on sewage and sewage treatment"2007)*

2.3.1.2 Equalization tank

Flow equalization simply is the damping of flow rate variation to achieve a constant or nearly constant flow rate and can be applied in number of different situation, depending on the characteristics of collection system.

In sewage treatment flow equalization (or flow rate equalization) may be achieve either by the in line arrangement or off-line arrangement. In the in- line arrangement the entire flow passes through equalization basin. In the off-line arrangement only the flow above the average daily flow rate is diverted in to equalization basin. In the in-line arrangement it is possible to achieve considerable damping of constituent-concentration and flow rate. On the other hand in the off- line arrangement the amount of the constituent-concentration damping is considerably reduced, though pumping requirements are minimized in this arrangement. (Figure-1)

A. Design consideration for equalization tank

A.1 Location of Equalization Facilities

Equalization after primary treatment causes fewer problems with solids deposit and scum accumulation. If flow equalization systems are to be located ahead of primary settling and biological systems, the design must provide for sufficient mixing to prevent solid deposition and concentration variation and aeration to prevent odor problem.

A.2 In-line or off-line equalization

It is possible to achieve considerable damping of constituent mass loading to the downstream process with in-line equalization, but only slight damping is achieve with off-line equalization.

A.3 Volume Requirement for the Equalization Basin

The volume required for flow rate equalization is determined by using an inflow cumulative volume diagram in which the cumulative inflow volume is plotted versus the time of day. The average daily flow rate, also plotted on the same diagram, is the straight line drawn from the origin to the end point of the diagram.

The physical interpretation of the diagram shown in (**Figure-2)** as is show at the low point of tangency (flow rate pattern A) the storage basin is empty. Beyond this point, the basin begins to fill because the slope of inflow mass diagram is greater than that of average daily flow rate .For flow rate pattern B, the basin is filled at upper point of tangency.

In practice, the volume of the equalization basin will be larger than that theoretically due to continuous operation of aeration and mixing equipment will not allow complete drawdown, although special structure can be built and some contingency should be provided for unfrozen change in diurnal flow. Although no fixed value can be given, the additional volume will vary from 10 to 20 percent of the theoretical value, depending on the specific conditions. (*"Techobanoglous et.al 2007.*)

A.4 Other consideration:

Mixing equipments prevent deposition of solids in the basin. To minimize mixing requirements for blending a medium – strength municipal wastewater (**see Table- 1**), having a suspended solids concentration of approximately 210mg/L, range from 0.004 to 0.008 kW/m^3 (0.02 to 0.04 $hp/10^3$ gal) of storage. Aeration is required to prevent the wastewater from becoming septic and odorous. To maintain aerobic conditions, air should be supplied at a rate of 0.01 to 0.015 m^3 / m^3 min (1.25 to 2.0 $mg/ft^3/10^3$ gal. min). In equalization basins that follow primary sedimentation and have short detention times (less than 2 hr), aeration may not be required.

2.3.1.3 Sedimentation tank

A. Primary sedimentation and secondary sedimentation tank

Clarification, through the process of sedimentation is the separation of suspended particles by gravitational settling. This operation can be used for grit and solids removal in the primary settling basin, removal of oil and grease, removal of chemically treated solids when the chemical coagulation process is used or solids concentration in sludge thickeners. (Figure-3)

Efficiently designed and operated primary sedimentation tank should remove from 50 to 70 percent of the suspended solid and from 25 to 40 percent of the BOD. (*Techobanoglous et.al 2007.)*

Secondary settling tank, which receive the biologically treaded flow, undergo zone or compression settling zone, occurs beyond a certain concentration when the particle are close enough together that inter particulate forces may hold the particles fixed relative to one another so that the whole mass tends to settle as single layer or “blanket” of sludge. The rate at which a sludge blanket settles can be determined by timing its position in a settling column test.

B. Type of settling-

Basically, there are four categories of settling depending upon the behavior of particles, as follows:

B.1 Discrete settling

Discrete particles do not change their size, shape or mass during settling. Grit in waste water behaves like discrete particle.

B.2 Flocculent settling –

Particles in relatively dilute solution will not act as discrete particles but will coalesce during sedimentation. As flocculation occurs the mass of particles increase and it settles faster. The removal of raw sewage organic suspended solid in primary settling tank, settling of chemical flocks in settling tank and bio flocks in the upper portion of secondary sedimentation tanks are example of flocculent settling.

B.3 Hindered or Zone settling –

Hindered or zone settling occurs in addition to discrete and flocculent settling. Because of high concentration of particles, the liquid tends to move up through the interstices of the contacting particles. as a result the contacting particles tend to settle as a blanket or zone maintaining the same relative position with respect t each other. This phenomenon is called as “Hindered” or “Zone” settling. This type of settling is applicable to concentrated suspensions such as are found in secondary settling basing following activated sludge.

B.4 Compression-

In the compression zone, the concentration of particle become so high the particle are in physical contact with each other, the lower layer supporting the weight of upper layer.

C. Design criteria

For primary sedimentation tank, both surfaces over flow rate and detention period (Hydraulic Retention time) are important design criteria as the solid to be settled are flocculent in nature and undergo flocculation. The major design parameter for secondary settling tanks designed to remove biofloculated solid are solids loading rate or solid flux as well as surface over flow rate. The plan surface area of secondary settling tanks is determined using both criteria and the greater of two is adopted for design. *("Manual on sewage and sewage treatment" Page- 215)*

C.1 Over Flow rate or Surface loading rate

Over flow rate expressed cubic meter per square meter of surface area per day, m3/m^2.d. The selection of a suitable loading rate depends on the type of suspension to be separated. Design for municipal plants must also meet the approval of state regulatory agencies, many of which have adopted standard for surface loading that must be followed. When the area of the tank has been established, the detention period in the tank is governed by water depth.

The effect of the surface loading rate and detention time on suspended solids removal varies widely depends on the character of the waste water, proportion of settle able solid concentration of solid and other factor. (*"Techobanoglous et.al 2007.)*

C.2 Detention period

The rate of removal of BOD and SS is maximum during the first 2 to $2^1/2$hours of settling and there after decrease appreciably. Hence, increase in the detention time beyond 2 to $2^1/2$ hours will not increase the percentage of removal of BOD or SS proportionately. Longer detention period may affect the tank performance adversely due to setting in of septic condition, particularly in tropical climate. *("Manual on sewage and sewage treatment"2007)*

C.3 Solid loading rate

The solid loading rate or solid flux is an important design variable for design of secondary sedimentation tank receiving bioflocculated solids. The solid flux represents the solid loading per unit surface area of tank per unit time and is expressed as kg SS/m^2.d. Design solid loading at average and peak flow are represent in table-3*("Manual on sewage and sewage treatment" Page- 216)*

C.4 Weir loading

Weir loading influences the removal of solids in sedimentation tank, particularly in secondary settling tanks where flocculated solid are settled in tanks. However, certain loading rates based on practice are recommended both for primary as well as secondary tank. For all primary, intermediate and secondary settling tanks (except in secondary tanks for activated sludge process), weir loading of 125 m^3/d/m for average flow recommended. For secondary settling tank in activated sludge or its modification, the weir loading is around185 m^3/d/m. (*"Manual on sewage and sewage treatment", Page-217*

Table -3 Design parameters for settling tank

Type of settling	Overflow rate $m^3/m^2/day$		Solid loading Kg/ m^2/ day		Depth	Detention time
	Average	Peak	Average	Peak		
Primary settling only	25-30	50-60	-	-	2.5-3.5	2.0-2.5
Primary settling followed by secondary treatment	35- 50	60-120	-	-	2.5-3.5	
Primary settling with activated sludge return	25-35	50-60	-	-	3.5-4.5	-
Secondary settling for trickling filter	15-25	40-50	70-120	190	2.5-3.5	1.5-2.0
Secondary settling for activated sludge(excluding extended aeration)	15-35	40-50	70-140	210	3.5-4.5	-
Secondary settling for extended aeration	8-15	25-35	25-120	170	3.5-4.5	

Source- "Manual on sewage and sewage treatment", 2007

C.5 Depth

The depth sets the detention time in the settling tank and also influence sludge thickening in the secondary settling tanks of the activated sludge plants. The depths recommended for horizontal flow tank are given table-3. (***"**Manual on sewage and sewage treatment"*, *-2007)*

C.6 Inlet and outlet

All inlets must be designed to keep down the entrance velocity to prevent formation of eddy or inertial currents in the tanks to avoid short circuiting. In the design of inlets to rectangular tanks, the following method are used to distribute the flow uniformly across the tank

Multiple pipe inlets with baffle boards of depth 0.45 to 0.6m in front of the inlets, 0.6 to 0.9 m away from it and with the top baffle being 25 mm bellow water surface for the scum to pass over Channel inlets with perforated baffle side wall between the tank and channels, or

C.7 Type and shape

Circular tanks are more common than rectangular or square tanks. Up flow tanks have been used for sewage sedimentation but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and could be more economically designed where multiple unit are to be constructed in a large plant.

Table-4 Typical design parameter for Secondary Sedimentation Tank (circular)

Parameter	Range	Typical
Depth (m)	3-4.5	3.6
Diameter (m)	3-60	12-45
Bottom slope (cm/m)	6.3-17	8.5
Flight travel speed (m/ min)	0.02-0.05	0.03

Source- "Techobanoglous et.al 2007

Table-5 Typical design parameter for secondary sedimentation tank (rectangular)

Parameter	Range	Typical
Depth(m)	3-4.5	3.6
Length(m)	15-90	24-39
Width(m)	3-24	4.8-9.6
Flight speed(m/min)	0.6-1.2	0.9
Length: width	3:1	4:1
Length: depth	15:1	-
Bottom Slope(cm/m)	6.3-17	8.5

Source-" Techobanoglous et.al 2007.

2.3.2 Secondary treatment (Biological treatment)

Secondary treatment is designed to substantially degrade the biological content of the sewage as derived from human waste, food waste, soaps, and detergent. The objective of the biological treatment of sewage is two coagulate and remove the non settle able colloidal solids and stabilize the organic matter with the help of micro organism (bacteria). The majority of the sewage treatment plant settles the sewage liquor using an aerobic biological process. In all these methods, the bacteria and protozoa consume biodegradable soluble organic contaminants and bind much of the less soluble fraction in to flock.

Secondary treatments are classified as the fixed film or suspended growth. Fixed – film treatment process including trickling filter and rotating biological contactor where the biomass grows on the media and the sewage passes over its surface. In suspended growth system – such as the activated sludge- the biomass is well mixed with the sewage and can be operated in a smaller space than fixed film system that treat the same amount of water. (www.wikipedia.org)

2.3.2.1 Activated Sludge Process (ASP)

The most common suspended growth process used for municipal waste water treatment is the activated sludge process.

Activated sludge plant involves:

1- Waste water aeration in the presence of microbial suspended,
2- Solid-liquid separation following aeration,
3- Discharge of clarified effluent,
4- Washing of excess biomass, and
5- Return of remaining biomass to the aeration tank.

A. ASP variables

The main variables of activated sludge process are the mixing regime, loading rate, and the flow scheme.

B. Mixing regime

Generally two type of mixing regime are the major interest in the activated sludge process; plug flow and complete mixing. In the first one, the regime is characterized by orderly flow of mixed liquor through the aeration tank with the no element of mixed liquor or mixing along the path of flow. In the complete mixing, the contents of aeration tank are well stirred and uniform throughout. Thus at steady state, the effluent from the aeration tank has the same composition as the aeration tank contain.

C. Loading Rate

A loading parameter that has been developed over the years is the hydraulic retention time (HRT denoted as θ)

$$\theta = V/ Q \qquad (eq^n -5)$$

V = volume of the tank (m^3) and

Q = sewage inflow, m^3/d

Another empirical loading parameter is volumetric organic loading which is defined as the BOD applied per unit volume of aeration tank, per day.

A rational loading parameter which has found wider acceptance and is preferred specific substrate utilization rate q, per day.

$$q = [Q (S_0 – S_e) / [V X] \qquad (eq^n -6)$$

A similar loading parameter is mean cell resident time or sludge retention time (SRT), $\theta_{c,}$

$$\theta_c = [V X]/ [Q_w X_r + (Q- Q_w X_e) \qquad (eq^n -7)$$

Where S_0 and S_e are influent and effluent organic matter concentration respectively, measured as BOD_5 (g/m^3), X, X_e and X_r are MLSS concentration in the aeration, effluent and return sludge respectively, and Q_w = waste activated sludge rate.

Under steady state operation the mass of waste activated sludge is given by

$$Q_w X_r = [Y Q (S_{0 –} S_e) – k_d X V \qquad (eq^n -8)$$

Where Y = maximum yield coefficient (microbial mass synthesize / mass of substrate utilize) and k_d = endogenous decay rate (d^{-1})

From the above equation it is seen that $1/q_c = Y_{d –} k_d$

If the value of S_e is small as compared $S_{o,}$ q, may also be expressed as food to microorganism ratio, F/M,

$$F/M = [Q (S_o- S_e)]/[X V] \qquad (eq^n -9)$$

The q_c value depend for design controls the effluent quality, and settle ability and drain ability of biomass, oxygen requirement and quality of waste activated sludge.

Flow scheme- Refer to Figure-4

D. <u>Extended aeration</u>

In extended aeration process at a low organic loading long aeration time, high MLSS concentration and low F/M. The BOD removal efficiency is high because of high detention in the aeration tank, the mixed liquor solid undergo considerable endogenous respiration and get well stabilized. The excesses sludge dose not requires separate digestion and can be directly dried on sand beds. Also the excess sludge production is minimum.

The oxygen requirement for the process is higher and running costs are also therefore high. However operation is rendered simple due to elimination of primary settling and separate sludge digestion.

D.1 Design Consideration

The items for consideration in the design of activated sludge plant are aeration tank capacity and dimensions, aeration facilities, secondary sludge settling and recycle and excess sludge wasting.

D.2 Aeration Tank

The volume of aeration is calculated for the selected value of q_c by assuming a suitable value of MLSS concentration, X.

$$VX = [YQ\, q_c\, (S_o - S)] \,/\, [\, 1+ k_d\, q_c\,] \qquad (eq^n -10)$$

Alternatively the tank capacity may be design from

$$F/M = [Q\, S_o] / [XV] \qquad (eq^n -11)$$

X = MLSS concentration (mg/l)
V= volume of tank (m^3)

Step 1: Choose a suitable value of q_c (or F/M) which depends on the expected winter temperature of mixed liquor, the type of reactor, expected settling characteristic of the sludge and the nitrification required. The choice generally lies between 5 days in warmer climate to 10 day in temperature ones where nitrification is desired along with good removal and, complete mixing systems are employed.

Step 2: Two interrelated parameters are selected as HRT and MLSS concentration. It is seen that economy in rector volume can be achieved by assuming a large value of X. Consideration which govern the upper limit (refer Table-6) are: initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitation of oxygen transfer equipment to supply oxygen transfer equipment to supply oxygen at required rate in small reactor volume, increased solid loading on the secondary clarifier which may be necessitate a larger surface area, design criteria for the tank and minimum HRT for the aeration tank.

D.3 Dimension of aeration tank-

The length of the tank depends upon the type of activated sludge plant. Except in the case extended aeration plants and completely mixed plants the aeration tanks are designed as long narrow channels. The width and depth of aeration tank depend on the type of aeration equipment employed. The depth controls the aeration efficiency and usually kept between 5 to 10 m. The width controls the mixing and usually kept between 1.2 to 2.2 m. The length should be not be less than 30 or not ordinary longer than 100m. The horizontal velocity should be around 1.5m/s. tank free board is generally kept between 0.3 and 0.5m.The inlet should be design to maintain a minimum velocity of 0.2 mps to avoid the deposition of solids.

D.4 Oxygen requirements

Oxygen is required in the activated sludge process for the oxidation of a part of the influent organic matter and also for the endogenous respiration of micro-organism in the system. The total oxygen requirement of the process may be formulated as follows:

$$O_2 \text{ required (kg/d)} = [Q (S_0 - S_e)/ f] - [1.42\ Q_w X_r] \qquad (eq^n\ \text{-12})$$

where f= ratio of BOD_5 to ultimate BOD and

1.42 = oxygen demand of biomass (g/g).

The eq^n may be expressed as

$$O_2 \text{ requred (kg/ d)} = [Q(S_0 - S_e)/f] - [1.42(VX/ \theta_c)] \qquad (eq^n - 13)$$

The formula does not allow for nitrification but allows only for carbonaceous BOD removal.

Table-6 Characteristics and design parameter of activated sludge system for municipal waste water

Processes Type	**Flow regime**	**MLSS (mg/L)**	**MLVSS/ MLSS**	**F/M***	**HRT, (hr)**	**θ_C (day)**	**Q_r/ Q**	**Efficiency (%)**	**Kg O_2/ Kg BOD_5 removal**
Conventional	Plug flow	1500-3000	0.8	0.3 - 0.4	4- 6	5 - 8	0.25-0.5	85-92	0.8-1.0
Completely mixed	Completely mixed	3000-4000	0.8	0.3-0.5	4- 5	5-8	0.25-0.8	85-92	0.8-1.0
Extended aeration	Completely mixed	3000-5000	0.6	0.1 – 0.18	12-24	10-25	0.5-1.0	95 - 98	1.0-1.2

Source- “Manual on sewage and sewage treatment”,

**unit of F/M (Food: Microorganism) is (Kg BOD_5/ Kg MLVSS Day)*

D.5 Application for correction factors –

The actual amount of oxygen required must be obtained by applying factors to a standard oxygen requirement that reflect the effect of salinity- surface tension (beta factor), temperature, elevation, diffused depth (for diffused aeration systems), the desired oxygen operating level, and the effect of mixing intensity and basin configuration. The inter relationship of these factor is given by the following expression.

$AOTR= SOTR\ [\beta\ C_{STH} - C_L / C_{S20}]\ 1.024^{(T-20)}\ \alpha\ F$ (eq^n -14)

Where AOTR = actual oxygen transfer rate under field condition, kg O_2/ hr

SOTR = standered oxygen transfer rate in tap water at 20° C, and zero dissolved oxygen, kg O_2 / hr

β = salinity – surface- tension correction factor, typically 0.95 to 0.98

$C_{s,T,H}$ = average dissolve oxygen saturation concentration in clean water in aeration tank at temperature T and altitude H, mg/ l

$AOTR=(C_{s,T,H})\ [P_d] / [P_{atm,H}] + [O_t] / [21]$ (eq^n -15)

$C_{S,T,H}$ = oxygen saturation concentration in clean water at a temperature altitude H(mg/L)

P_d = pressure at depth of air release, kPa

$P_d = P_{atm} + [\{d \times 14.7\}/33.92]$ + frictional loss in piping system (eq^n -16)

O_t = percent oxygen concentration leaving tank usually 18 to 20 percent

$C_{s,20}$ = dissolve oxygen saturation concentration in clean water at 20 °C to and 1 atm, mg/L

T = operating temperature, °C

α = oxygen transfer correction factor for waste

F = fouling factor, typically 0.65 to 0.9

D.6 Aeration facilities

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the waste water against a specific level of dissolved oxygen in the waste water.

Aerator are rated based on the amount of oxygen they can transfer to the water under standard condition of 20 °C, 760 mm Hg barometric pressure and zero DO

1- Aeration removes odour and tasted due to volatile gases like hydrogen sulphide and due to algae and related organisms.
2- Aeration also oxidize Fe and Mn, increase dissolved oxygen contain in water, remove CO_2 and reduces corrosion and remove methane and other flammable gases.
3- Principle of treatment on the fact that voletile gases in water escape in to the atmosphere from the air- water interface and atmosphere oxygen take their place in the water provided the water body can expose itself over a vast surface to the atmosphere.

D.7 Diffused Aerators:

Various diffusing device have been classified as either fine bubble or coarse bubble, with the connotation that fine bubble were more efficient in transferring oxygen. The current preference is to categorize the diffused aeration system by the physical characteristics of the equipment. Three categories' are defined:

- Porous or fine- pore diffuser,
- Nonporous diffuser, and
- Other diffuser device such as jet aerators, and such as jet aerator, and aspirating aerators, and U-tube aerator ("*Techobanoglous et.al 2007*)

A diffused-air system consists of diffuser that are submerged in the waste water, header, pipe, air mains, and the blower and appurtenances through which the air passes. It consists of a tank with perforated pipes, tubes or diffuser plates, fixed at the bottom to release fine air bubbles from compressor unit. The tank depth is kept as 3 to 4 m and tank width is within 1.5 times its depth. If depth is more, the diffusers must be placed at 3 to 4 m depth below water surface.

Table-7 Description of commonly used air diffusion device

Type of diffuser	Transfer efficiency	Description
Porous		
Disk	High	Rigid ceramic disks mounted on air-distribution pipe near the tank floor.
Dome	High	Dome- shaped ceramic diffuser mounted on air distribution pipe near the tank floor
Membrane	High	Flexible porous membrane supported on disk mounted on air- distribution grid.
Panel	very low	Rectangular panel with a flexible plastics perforated membrane.
Non porous		
Fixed orifice	Low	Device usually constructed of model plastic and mounted on air-distribution pipe
Slotted tube	Low	Stainless-steel tubing containing perforation and slots to provide a wide band of diffused air.
Static tube	low	Stationary vertical tube mounted on basin bottom and function like on air – lift pump.

Source: "Techobanoglous et.al 2007432

2.3.2.2 Blowers-

Three types of blowers commonly used for aeration

1- Centrifugal
2- Rotatory lobe positive displacement, and
3- Inlet guide vane- variable diffuser

Centrifugal blowers are almost universally used where the unit capacity is greater than 425m^3/ min of free air. Rated discharge pressure range normally from 48 to 62 kN/ m^2. For higher discharge pressure applications [> 55 kN/m^2] and for capacities smaller than 425 m^3/ min of free air per unit, rotator- lobe positive displacement blower is commonly used. The power requirement for adiabatic compression is given in

$$P_W = [WRT_i \{(P_2/P_1)^{0.283} - 1\}]/ (29.70 \text{ ne}) \qquad (eq^n\text{-}17)$$

Power each blower (Kw) hp.

W = Weight of flow of air Kg / s (lb/s)

R = 8.314 KJ ml (K SI unit)

$T_{i\,=}$ absolute inlet temperature (k)

P_1 = absolute in let pressure (atm)

$P_{2\,=}$ absolute inlet pressure (atm)

e = .283

k = 1.395 for air

e = efficiency

R = 8.314 Kg/ K-mole

T_i = 293K

(“Techobanoglous et.al 2007)

A. Air Piping

Air piping consists of main valve, meter and other fitting that transport compressed air from the blowers to the air diffusers. Because the pressure are low [less than 70 kN/ m^2], lightweight piping can be used. Piping should be sized so that losses in air header and diffuser manifold are small in comparison to the losses in diffusers. Friction losses in air piping can be calculated using the Darcy-Weishbach equitation written in the following form:

$$h_l = f \quad L/D \qquad (eq^n\text{-}18)$$

h_l = friction losses, mm(in) of water.

f = dimension less friction factor obtained from moody diagram based on relative roughness. It is recommended that f be increased by at least 10 percent to allow for an increase in the friction factor as the pipe ages.

L= equivalent length of pipe, m (ft)

D= pipe diameter, m (ft)

H_i = velocity head of air, mm (in) of water.

(*Techobanoglous et.al 2007.)*

2.3.2.3 Sludge digester

In all biological waste treatment processes some surplus sludge is produced. The objective of residual management is:

1- Reduction of water content.
2- Stabilization of solids sludge.
3- Reduction in sludge solid volume.

In extended aeration process where aerobic digestion of surplus sludge is done, the sludge can be taken directly for dewatering and disposal. In case of activated sludge and trickling filter plants, the sludge is taken (along with the primary sludge) to sludge digester for further demineralization and there after it is dewater.

A. Sludge Dewatering Method

1- Natural: sludge drying beds, sludge lagoons
2- Mechanical: sludge thickener, Centrifuge, vacuum filter press.
3- Physical: heat drying, incineration.

Table-8 Stepwise reduction in moisture content in dewatering extended aeration sludge

Sludge source	**Moisture content % by weight**	**Weight, g/person-day**		
		Solid	**Water**	**Total**
Initial moisture content	99	30	2970	3000
After thickening	96	30	720	750
After other mechanical process	90	30	270	300
After natural or physical drying	60	30	45	75

Source- "Manual on sewage and sewage treatment",

B. Sludge Digestion or stabilization

Sludge digestion can be achieved by any of the following biological processes:

B.1 Anaerobic digestion

In the first stage of digestion known as hydrolysis, the complex organic matter like protein, cellulose, lipid are converted by extra cellular enzymes in to simple organic matter. In the second stage is known as acid fermentation, the soluble organic matter is converted by acetogenic bacteria also called 'acid former' in to acetic acid.

In the third stage known as methane fermentation, organic acid are converted in to methane with the help of group of methanogenic bacteria also called as 'methane former which are strictly anaerobic. During acid fermentation if pH below 6.0, methane formation cease and the more acid accumulates, thus bringing the digestion process to stand still. The methane bacteria are highly active in mesophilic (27-43°C) with the digestion period of four week and thermophilic range (35-46°C) with digestion period of 15-18 days.

Calcium, magnesium, and ammonium biocarbonate are example of buffering substance found in a digester. (Figure-5)

B.1.1 Type of Anaerobic Digester

The anaerobic digester are two type : standard rate and high rate. In the slandered rate digestion processes, the digester content are usually unheated and unmixed. The digestion period may vary from 30 to 60 d. In a high rate digestion process, the digester contents are heated and completely mixed. The required detention period is 10 to 20 d. (Figure-6)

1. Often a combination of standard and high rate digestion is achieved in two-stage digestion. The second stage digester mainly separates the digestion solids from the supernatant liquor: although additional digestion and gas recovery may also be achieved.

B.1.2 Design Details

Standard rate or low rate sludge digester are designed as single units for plants treating up to 4MLD. For larger plants, units are provided in multiple of two, the capacity of individual unit not exceeding 3MLD.

1- Tank size is not less than 6m diameter and not more than 55 m diameter.
2- Liquid depth may be 4.5 to 6 m and not greater than 9 m.
3- Floor slope are provided with hopper shaped bottom floor with the slope in the range of 1in 10 facility easy withdrawal of digested sludge.
4- Roofing of sludge digester can have either fixed or floating cover is used for gas holder in a digestion tank, and effective vertical travel of 1.2 to 2.0 m should be provided.
5- For fixed dome or conical roofs free board between the liquid level and the rim of digester wall should not be less than 6 m.
6- The digestor capacity may be determined from the relationship

$$V = [V_f - 2/3\ (V_f - V_d)]\ T \qquad (eq^n\ \text{-19})$$

Where V= capacity of digestor in m^3,

V_f = volume of fresh sludge

V_d = volumeof daily digested sludge a accumulation tank m^3/d,

T= digestion time in days required for digestion, d.

As indicated earlier additional capacity is required to store digested sludge during the monsoon period which is given by the expression

$$\text{Additional monsoon storage volume} = V_d\, T_2 \qquad (eq^n\ \text{-20})$$

T_2 = period of storage during monsoon (in, day); and notation are all other same as defined.

The required capacity of the digester including the monsoon storage volume is thus given by the following expression.

$$V = [V_f - 2/3\ (V_f - V_d) \times T] + [V_d\, T_2] \qquad (eq^n\ \text{-21})$$

(“Manual on sewage and sewage treatment”,)

B.2 Aerobic digestion

Aerobic digestion is the biological degradation of organic matter in the presence of free oxygen provided by long term aeration. The aerobic digestion is the similar to the activated sludge process. Aerobic digestion may also defined as a process in which micro organism obtained energy by endogenous or auto oxidation of their cellular protoplasm. The biologically degradable constituent of the cellular material are slowly oxidized carbon dioxide, water and ammonia. The ammonia from this oxidation is subsequently oxidized to nitrate as digestion process. The aerobic digestion produce volatile solids reduction comparable to those in anaerobic digestion has low BOD in supernatant, fewer operational problem and lower initial cost.

The factor that should be considered during digestion in an aerobic digester include detention time, loading criteria, oxygen requirement, energy requirement for mixing, environmental condition and process operation.

The volatile solid destroyed in aerobic digestion in about 10 to 12 days time, at a temperature of 20° C would be 35 to 45%. Higher temperature will result in reduction in the period of digestion. The oxygen requirement for the complete oxidation of cell tissue is 2kg/kg of cell and that for the complete oxidation of the BOD_5 containing in primary sludge varies from about 1.7 to 1.9 kg/kg of volatile solid destroys.

Table- 9 Values of design parameter for aerobic digester

S. no	Parameter	Value
1-	Detention time, day at 20 °C	
	Waste activated sludge only	10-12
	Activated sludge from plant operated without primary settling	12-18
	Primary plus activated or trickling filter sludge	15-20
2-	Solid loading, kg volatile solids/ m^3 per day	1.6- 4.8
3-	Oxygen requirement, kg/kg destroyed	
	Cell tissue	2.0
	BOD_5 in primary sludge	1.7
4-	Energy requirement for mixing	
	Mechanical aerator, kw/10^3 m^3	20-40
	Air mixing, m^3/ 10^3 m^3per min	20-40
5-	Dissolve oxygen level in liquid, mg/l	1-2

("Manual on sewage and sewage treatment", 2007)

B.3 Sludge conditioning

Sludge is conditioned expressly to improve its dewatering characteristics. The two methods most commonly used for sludge conditioning are

1- Chemical conditioning.

2- Heat treatment.

B.4 Sludge drying beds-

The method consists of applying the sludge in a 20 to 30 cm thick layer on the specially prepared open beds of sand and gravel and allowed to dry. The drying beds are commonly 6 to 8 m wide and 30 to 45 m long. A length of 30 m away from the inlet should not be exceeded with a single point of a wet sludge discharge, when the bed slope is about 0.5%. Area required for the drying beds range from 0.1 to 0.15 m^2 per capita with dry solid loading of 80 to 120 kg/ m^2 of bed per year for digested primary sludge, and from 0.175 to 0.25 m^2 per capita with dry solid loading of 60 to 120 kg/ m^2 of bed per year for digested mixed sludge. The average cycle time for drying may range from a few days to 2 weeks in warmer climates to 3 to 6 week or even more in unfavorable ones. The inlets to the drying bed are so arrange to easily drain and have a minimum diameter of 20 cm terminating at least 30 cm above the sand surface. **(Figure-7)**

2.3.2.4 SAFF-

Submerge Aerobic Fix Film Reactor (SAFF) an advance, modular sewage treatment systemideal for large residential and commercial developments requiring on site secondary waste water treatment. Utilizing proven submerged aerated filtration technology is design each SAFF waste water system to meet waste water treatment and land application requirement. SAFF systems are modular and can be sized for any application. The system can also be expanded at a letter date if required to suit an expanding location. (www.hynds.co.in)

SAFF technology for optimum performance and dependability. Using reliable, cost effective and energy efficient blower for aeration are with an integral flow management system and enter the biological treatment stage where it is aerated with fine bubble membrane diffuser. The continuous supply of oxygen together with the incoming food source encourage microorganism to grow on the surface of the submerged media, convening the waste water in to CO_2 and water in the process. Media of SAFF is providing more surface area for microorganism to grow.

Excess micro-organism (known as humus solids) that flows out of the biological treatment stage is separated from the final effluent in another settlement stage. (www.enso-international.com)

Features and benefit

1- Most advance waste water technology with low operation and maintenance costs.

Installation is completely below ground. Liquor from the primary settlement stage

Treated water may be available for reuse

2- Not restricted by conventional limits of mixed liquor suspended solid (MLSS) concentrations can operate in the range 12000mg/l – 23000 mg/l. (www.Conderproducts.com)

Process flow diagram of extended aeration and SAFF is 8&9 respectively.

Chapter 3: Objective

1- Design of STP by extended aeration activated sludge process
2- Design of STP by SAFF technology
3- Cost comparison between extended aeration and SAFF technology

Chapter4: Design of STP

4.1 Design basis

Table-10 Design of basis aeration and SAFF technology is based on following basis

S. No.	Parameters	Values
1-	Flow rate	100 KLD
2-	BOD (incoming influent)	300 mg / l
3-	BOD (after treatment)	30 mg/l
4-	Suspended concentration (SS)	250 mg/l

4.2 Bar screen design

Table-11 Assumption for design of bar screen channel

S. No.	Parameter	Units
1-	Size of bar	10mm ×50mm
2-	Spacing between bars	10mm
3-	Angle of inclination	45^{θ}
4-	Velocity through the screen	0.6 m/s
5-	Approach velocity	0.3 m/s
6-	Depth of flow of incoming screen	0.07m
7-	Dia of incoming sewer	0.06 m

Source- "Manual on sewage and sewage treatment",

Results:

1. Flow Rate = $100/3600\times24 = 1.15 \times 10^{-3}\ m^3/s$

2. Peak Flow Rate = 3 ×average flow rate

$= 3.45\times 10^{-3} m^3/s$

3- Area of opening of screen channel

$$A = Q/V$$

Q = flow rate = $3.45\times 10^{-3}\ m^3/s$

V = velocity through screen = 0.6 m/s

$\Rightarrow$ $A = 5.75\times 10^{-3}\ m^2$

4-Nomber of opening screen channel

Depth of flow of screen = 0.07m

Spacing between bars = 10 mm

Number of opening = [$5.75\times 10^{-3}] / [(0.07\times 10/1000)]$

$\Rightarrow$ Number of opening = 8

5. **Number of bars** = 8- 1

$\Rightarrow$ Number of bars = 7

6. **Total width of screen channel**

= (8 ×10/ 1000) **+** (7 × 10/1000)

$\Rightarrow$ Total width of screen = 0.15 meter

7. **Length of screen channel**

As per eq^n (4)

Where

L= length of the screen channel (in m);

$\theta = 45°$

W = 0.15 m

d = 0.07 m

$d_{s} =$ 0.06 m

$\Rightarrow$ L= 0.73m

8. **Velocity of flow in screen channel**

$$v = [Q]/ [W\times d] \qquad (eq^n\text{- }22)$$

W = width of screen channel= 0.15 m,

Q = flow rate = $3.45 \times 10^{-3}\ m^3/s$

d = depth of flow in screen = 0.07 m

$\Rightarrow$ V = 0.31 m/s

9. Head loss through the screen is given by

As per eq^n (2)

Where

h_L= head loss (in m)

V = 0.6 m/s

v = 0.31 m/s

$\Rightarrow$ h_L = 0.0196 m

(Summary of bar screen design results in Table – 26)

4.3 Oil and grease (O&G) Trap

Table-12 Assumption for design of oil and grease trap

S. No.	Parameter	Value
1-	Flow rate	100 KLD
2-	Resident time (HRT)	30 minute
3-	Depth from inlet	1 m
4-	Length	2 time of width

Source- "Manual on sewage and sewage treatment",

Results

1-**Volume of tank**

HRT= V/Q (eq^n- 5)

HRT= Resident time = 30 minute

Q = flow rate = 100 KLD

$\Rightarrow$ V= 2.08 $m^3 \approx 2\ m^3$

$\Rightarrow$ Standard depth = 1m from in let

$\Rightarrow$ Area= 2 m^2

$\Rightarrow$ Length = 2 m

$\Rightarrow$ Width = 1 m

(Summary of oil & grease design result in Table -27)

4.4 Equalization tank design

Table-13 Assumption for equalization tank design

S. No	Parameter	Values
1-	HRT	16 hr
2-	Flow rate	100 KLD
3-	Depth	3.0 meter

Sources: *"Techobanoglous et.al 2007.*

Results:

1- Volume of tank

$$HRT = V/Q \qquad (eq^n\text{- } 5)$$

V= volume of tank (m^3)

Q= flow rate (m^3/s)

⇒ V = 66.6 m^3

⇒ Area = 22.3 m^2

⇒ Length = 5 meter

⇒ Width = 4.50 meter

⇒ Free board =0.5 meter

Air should be provided at a rate of 0.01 to 0.015m^3/m^3Minute or 0.6 to m^3/m^3.

("Wastewater Engineering" by Met calf & Eddy, Inc Page-344)

⇒ So air supplied = 66.6×0.8

= 53.28 m^3/ hr

(Summary of equalization design result in Table -28)

4.5 Design of Primary Sedimentation Tank

Table- 14 Assumption for primary sedimentation tank design

S. No.	Parameters	Values
1-	Over flow rate	1 m^3/ m^2. hr
2-	Flow rate	100 KLD =4. 16 m^3 /hr
3-	Detention time	2.5 hr

4-	Depth of tank	2.5 meter
5-	Baffle	0.2meter
6-	Slope	45°
7-	Dia of outlet	0.30 m

Source- "Manual on sewage and sewage treatment",

Results:

Area of tank

Area = [flow rate] / [over flow rate]

$\Rightarrow$ Area =4.16 m^2

$\Rightarrow$ L= 2.03 meter

$\Rightarrow$ W = 2.03 meter

*Lower portion of tank slope by 45° (1: 1) and dia of outlet 0.30 meter so

$\Rightarrow$ Total volume of tank 9. 39 m^3

$\Rightarrow$ HRT = 2.25 hr

$\Rightarrow$ sump dimension = 0.5m×0.5m×2.5m

(Summary of Primary clarifier design result in Table - 29)

4.6 Design of aeration tank

Table-15 Assumption for extended aeration tank design

S. No.	Parameter	Value
1-	MLSS	4000mg/l
2-	S_0 (influent BOD)	300 mg/l
3-	S_e (effluent BOD)	30 mg/g
4-	F/M	0 .1
5-	SRT	20 day
6-	Depth of tank	3.5 meter

Source- "Manual on sewage and sewage treatment",2007

Results:

1. Volume of aeration tank

F/M= [BOD load]/ [MLSS ×Volume of tank] (eq^n- 9)

$\Rightarrow$ Volume of tank = 67.5 m^3

2. Hydraulic retention time (HRT)

As per eq^n -5 Q = 100 KLD

V= volume of tank = 67.5 m^3,

⇒ HRT = 16.2 hr

3. Dimensions of tank

Volume = 67.5 m^3

Area of tank = 19.2 m^2

⇒ Width = 2× depth = 7 m

⇒ Length = 2.75 meter

4. O_2 Required for aeration

As per eq^n -13

Q = 100 KLD

S_0 = 300mg/ l

V= 67.5 m^3

S_e = 30 mg/

X = 4000mg/l

θ_c = 20 day

⇒ O_2 required = 20.52 Kg/day

For safe point we take 100% extra O_2 demand so = 20.52 × 2

= 41. 02 Kg/day

= 1.71 Kg O_2/ hr

⇒ **Air requirement** = 41.04 /0.21 (w/V)

= 195 Kg air /day

= 8.14Kg air/ hr

= 0.135 Kg air/minute

5. Actual O_2 transfer rate

As per eq^n 15

P_{atm} = 14.7 Psi

d= 11.4 feet

Frictional losses = 1.5 Psi

$\Rightarrow$ P_d = 20 .71 Psi = 20.71 mmHg

Assume 7% of O_2 bubbling through water absorb

O_t = 21(1- x)

$\Rightarrow$ O_t = 19.5%

So compute value of C_{STH} of depth

Where

C_{STH} = 8.17 mg/l

$\Rightarrow$ $C_{S\text{-}TH}$ = 9.39 mg/l

≈ 9.50 mg/l

As per eq^n -15

SOTR = 1.2 kg O_2 / h

β = 0.95

α = 0.5

F= 0.72

T = 27 °C

C_{s20} = 9.08

$C_{S\text{-}TH}$ = 9.50 mg/l

C_L = 1.5

$\Rightarrow$ AOTR = 0.41 Kg O_2 /hr

6. Number of diffuser = [Demand]/ [Supply]

Where

Demand =1.71 Kg O_2/ hr

Supply = 0.41 Kg O_2 /hr

Number of diffuser = 4.17

≈ 5

7. Volume of air supply by diffuser

$$AOTE = SOTE\ [\beta\ C_{STH} - C_L / CS20] 1.024^{(T-20)}\ \alpha\ F \quad (eq^n\text{-}23)$$

Where

AOTE = Actual oxygen transfer efficiency of diffuser

Slandered oxygen transfer efficiency of diffuser SOTE = 30%

$\beta = 0.95$

$\alpha = 0.5$

F= 0.72

T = 27 °C

$C_{s20} = 9.08$

$C_{s20} = 9.08$

$C_{S\text{-}TH} = 9.50$ mg/l

$C_L = 1.5$

AOTE = 10.25%

Efficiency of diffuser = 10%

So O_2 should be supplied = [41.04 Kg/ day]/ [0.10]

. = 410 .4 Kg O_2 / day

Air should be supplied =1784.34 Kg air / day (23% w/w of air)

= .0206Kg/s

Volume of air should be supplied at (STP) = 62.55 m^3 /hr

8. Blower capacity = 5HP at 7 Psig (Refer Kay international Limited)

(Summary of aeration tank design result in Table -30)

4.7 Secondary Sedimentation Tank

Table -16 Assumptions for Secondary Sedimentation Tank design

S. No.	Parameter	Values
1-	Average surface over flow rate	$1 m^3/m^2$ hr

2-	Flow rate	100KLD
3-	Depth of tank	3.5 m

Source- "Manual on sewage and sewage treatment",

Results:

1. Surface area for secondary sedimentation tank

(a) Adopting surface over flow rate at 1m^3/ m^2 hr ^{at} average flow rate

Where

$Q = 4.16\ m^3/hr$

$\Rightarrow$Surface area required = 4.16 m^2

$\Rightarrow$ L = 2.0 m

$\Rightarrow$ W = 2.0 m

Lower portion of tank slope by 45° (1: 1) and dia of outlet 0.30 meter so

$\Rightarrow$ Total volume of tank = 11.25 m^3

$\Rightarrow$ HRT = 2.70 hr

$\Rightarrow$ Sump dimension = 0.5m×0.5m×3.5m

(Summary of secondary clarifier design result Table -31)

4.8 Sludge digester design

(a) Determination of Quantity and volume of Primary Sludge

Table-17 Assumption for quantity primary sludge

S. No.	Parameter	Values
1-	SS	250 mg/l
2-	Consistency of Sludge	2% = 20,000 mg/l = 20Kg/ m^3
3-	% of removal of SS in primary sedimentation tank	30%
4-	Flow rate	100 KLD

Source- "Manual on sewage and sewage treatment", 2007

Results:

1- TSS load in sedimentation tank

⇒ TSS load = 25Kg /day Sludge

Sedimentation tank efficiency 30% Removal = 25× 0.30

⇒ Quantity of primary sludge = 7.5 Kg/day Sludge

⇒ Volume of Sludge = 0.375 m^3/day

$\approx$ 1 m^3/ day

(b) Determination of Quantity and volume of Secondary Sludge

Table- 18 Assumption for quantity of secondary sludge

S. No.	Parameter	Values
1-	Consistency of secondary sludge	0.8% = 8000mg/l = 8Kg/m^3
2-	Sludge generate from per Kg BOD load	0.2 Kg
3-	Flow rate	100KLD
4-	Height of digester	3.0 m

Source- "Manual on sewage and sewage treatment", 2007

Result:

$$\text{BOD load} = Q\,(S_0 - S_e) \qquad (eq^n - 23)$$

⇒ BOD Load = 27Kg/ day

⇒ So total secondary sludge generate = 5.4 Kg /day

⇒ Volume of Sludge = 0.67 m^3 /day

Secondary sludge does not go to digester because we assume that secondary sludge already digested.

(c) Volume of primary sludge after digestion

Table-19 Assumption for design of sludge digester

S. No.	Parameter	Values
1-	TSS load	25 Kg/day
2-	volume of primary sludge	$1m^3$/ day
3-	Approximate % of volatile matter (VM) in Primary sludge	70%

Source- "Manual on sewage and sewage treatment", 2007

Suspended concentration of primary sludge = TSS load/ Volume of primary sludge

For 50% destruction the digested sludge under mesophilic condition HRT require = 20 Days

⇒ Quantity of V.M. digested sludge = 0.5×17.5

⇒ Suspended concentration of primary sludge = 25 kg/m^3

⇒ Quantity of V.M. in raw sludge = 0.70×25

= 17.5 Kg/day

⇒ Quantity of nonvolatile or inorganic solid = 0.3×25

= 7.5 Kg/day

Approximate % of destruction of V.M. is = 50%

= 8.75 Kg/day

Quantity of non digested in organic solid = 7.5 Kg/ day

⇒ Total primary digested sludge = 7.5+ 8.75

= 16.25 Kg/day

⇒ Percentage of V.M. indigested sludge = 53%

⇒ Percentage of inorganic matter = 45%

Assume after digestion consistency 4% = 40Kg/ m^3

Volume after digestion = 16.25/40

⇒ Volume of primary sludge after digestion = 0.40 m^3/ day

(d)Therefore volume of digester-

As per eq^n -20

Where

$V_f = 1m^3$/day

V_d = 0.40(in m^3/ day), and

T_1= 20 in days,

⇒ Volume of digester = 7 m^3

(e) Dimension of digester tank

Height = 3m (Assumption as per ***"****Manual on sewage and sewage treatment"2007,)*

⇒ Area = 2.33 m^2

⇒ Diameter of digester = 1.80 m

⇒ Free board = 0.5m

⇒ So total depth of digester = 3. 5 meter

(Summary of secondary clarifier design result Table -32)

⇒ Volume of primary digested sludge + secondary digested sludge = 1.07 m^3

4.9 Sludge drying bed design

Assumption as per *"Manual on sewage and sewage treatment 2007,*

Dewatering drying and sludge removal = 7 days

Depth of application of sludge = 0 .3m

⇒ Total plane area of sludge drying Bed = 1.07×7/.30

= 25 m^2

Number of Beds is assuming to be 7

⇒ Area of bed = 3.57 m^2

⇒ Length = 1.90 m

⇒ Width = 1.90 m

Free board = 0.5 m

Total depth with sand, gravel and free board = 1.4 m

Total volume of tank = 35 m^3

(Summary of sludge drying bed design in Table-33)

4.10 Design of SAFF reactor

Table- 20 Assumption for SAFF reactor design (as per discussed with different suppliers)

S. No.	Parameter	Values
1-	MLSS	23000mg/l
2-	F/M	0.1
3-	Design flow rate	100 KLD
4-	Incoming BOD	300Mg/l
5-	Out coming BOD	30 Mg/l
6-	BOD load	27 kg/m^3

Sources: (www.hynds.co.in)

Results:

Step -1 volume of tank

F/M = [BOD load]/ [MLSS× Volume of the tank] (eq^n –5)

$\Rightarrow$ Volume of tank = 12 m^3

$\Rightarrow$ Depth of tank = 3m

$\Rightarrow$ Area of tank = 4 m^2

$\Rightarrow$ Width of tank = 1.5 m

$\Rightarrow$ Length of tank= 2.5 m

$\Rightarrow$ Depth of media of reactor= 2.5 m

$\Rightarrow$ Length of media = 2.5 m

$\Rightarrow$ Width of media = 1.5 m

$\Rightarrow$ free board= 0.5

(Summary of results shown in Table- 34)

Hydraulics diagram refer in figure – 10

Chapter 5: List of mechanical Unit and its cost

Table-21 List of mechanical unit and cost for extended aeration

S. No	Name of the equipment	Detail	Qty.	Rs/equipment	Cost (Rs)
1	Bar screen (Steel bars)	4 kg	7	38	1064
2	Raw sewage pump	0.75kw, 1.0 HP	6	10000	60000
3	Air blower for aeration tank and equalization tank.	5HP, 7 Psig, Model- 42*	2	42000	84000
4	Diffuser for aeration (Fine bubble diffuser)	1.2- 2.4 kg O_2/ kwh	5	2000	10000
5	Sludge pumps (Centrifugal)	0.75kw, 1.0 HP	4	10000	40000
				Total	195064

Additional (depends on need) cost for Piping, fittings, valves, headers is 10% (approx) of total cost (Mechanical and civil).

Table-22 List of mechanical unit, and cost for SAFF

Sr. No	Name of equipment	Detail	Qty.	Rs/ equipment	Cost(Rs)
1	Bar screen (Steel bars)	4 kg	7	28 kg	1064
2	Raw sewage pump	0.75kw, 1HP	4	10000	40000
3	Air blower for aeration tank and equalization tank.	5HP, 7Psig Model- 42*	2	42000	84000
4	Media for SAFF reactor,	2.5m×1.5m×2.5m	2	3000Rs/ m^3	28125
5	Diffuser for aeration (Fine bubble diffuser)	1.2- 2.4 kg O_2/ kwh	5	2000	10000
6	Sludge pumps (Centrifugal)	0.75kw, 1HP	4	10000	40000
				Total	203189

Additional (depends on need) cost for Piping, fittings, valves, headers is 10% (approx) of total cost (Mechanical and civil).

*** Model-42 is provided by** Kay International Ltd.

Chapter 6: List of civil work &Price Schedule

Table-23 List of civil work and cost for extended aeration-

S. No.	Unit	Quantity	Size (m^3) (l×w×d)*	MOC	Cost(Rs)
1	Screen chamber	1	0.73m×0.15m×0.07m	RCC	2000
2	Oil& grease trap	1	2m×1m×1m	RCC	14000
3	Equalization tank	1	5m×4.5m×3m	RCC	402000
4	Primary clarifier	1	2.03m×2.03m×2.5m	RCC	72000
5	Aeration tank	1	2.75m×3m×3.5 m	RCC	173250
6	Secondary clarifier	1	3m×2.2m× 3.5m	RCC	138600
7	Sludge holding tank (sump)for primary clarifier	1	0.5m×0.5m×2.5m	RCC	4375
8	Sludge holding tank (sump)for secondary clarifier	1	0.5m×0.5m×3.5m	RCC	6125
9	Sludge digester	1	Height 3.5, diameter 1.80(7m^3)	RCC	49000
10	Sludge drying bed	1	13.5m×1.90m×1.4m	RCC	179550
11	Foundation for static equipment pump, blower railing, stair case, inter connecting	4	Suitable	RCC	2000
				Total	1042900

* This is applicable only for rectangular design

Table-24 List of civil work and cost for SAFF reactor –

S. No	Unit	Quantity	Size(m^3) (l×w×d)*	MOC	Cost(Rs)
1	Screen chamber	1	0.73m×0.15m×0.07 m	RCC	2000
2	Oil& grease trap	1	2m×1m×1m	RCC	14000
3	Equalization tank	1	5×4.5×3 m	RCC	402000

4	Primary clarifier	1	2.03m×2.03m×2.5m	RCC	70000
5	SAFF reactor	1	2.5m×1.5m×3m	RCC	84000
6	Secondary clarifier	1	3m×2.2m× 3.5m	RCC	138600
7	Sludge holding tank(sump)	1	0.5m×0.5m×2.5 m	RCC	4375
8	Sludge holding tank (susmp)for secondary clarifier	1	0.5m×0.5m×3.5m	RCC	6125
9	Sludge digester	1	Height 3.5, dia1.80(7m^3)	RCC	49000
10	Sludge drying bed	1	13.5m×1.90m×1.4m	RCC	179550
11	Foundation for static equipment pump, blower railing, stair case, inter connecting	4	Suitable	RCC	2000
				Total	951650

Table-25 Cost comparison of Extended Aeration & SAFF Technology

S.No	Technology	Civil cost (Rs)	Mechanical unit cost (Rs)	Additional Cost*(Rs)	Total cost (Rs)
1	Extended aeration	1042900	195064	123796.4	1361760.4
2	SAFF technology	951650	203189	115483.9	1270322.9

* Additional cost (depends on need) for Piping, fittings, valves, headers is 10% (approx) of total cost (Mechanical and civil).

* Civil cost has been taken according to experience of civil engineer, in current market situation 6000 Rs./ m^3 for large tank and 7000 Rs./m^3 for small tank.

* Steel cost in current market situation 38 Rs/Kg.

* Given mechanical units, diffusers, pumps, blower and SAFF media cost according to current market situation.

Chapter7: Conclusion

In the present study we have done the feasibility study of two different technologies viz. extended aeration and SAFF, used for the treatment of sewage. During our study we found the following results:

Result of design-

1- Dimension of Oil and grease trap unit is (2m×1m×1m)
2- Number of bar and width of screen channel is 5, and 0.15m respectively. Length and head loss is 0.73m, head loss 0.0196m
3- Dimension of equalization tank is 5m×4.5m×3m.
4- Dimension of primary clarifier and primary sump is 2.03m×2.03m×2.5m and 0.5m×0.5m×2.5m.
5- Dimension of aeration tank is 2.75m×3m×3.5m and no of diffuser and blower capacity is 5 and 5HP, respectively.
6- Dimension of secondary clarifier is 2m×2m× 3.5m and secondary sludge sump dimension is 0.5×0.5×0.5 respectively.
7- Height and dia of design Digester is 3m and 1.80m. Total plane area sludge drying bed 25m^2 number of bed 7 with 3.57 m^2.
8- Design of SAFF reactor is 2.5m×1.5m×3m and volume of media 2.5m×1.5m× 2.5m.

The basic design (O&G trap, Bar screen, Equalization tank, Primary clarifier, Secondary clarifier and Sludge drying bed) is same for both the technologies except aeration tank and SAFF reactor.

The economical feasibility is dependent on civil cost as well as on electrical installation cost and both these factors are market dependent. On the current market situation, we estimated the total (civil, mechanical and additional) cost for extended aeration and SAFF technology is Rs. 1361760.4and Rs.1270322.9 respectively.

SAFF media is provided to increase the surface area for the growth of microorganisms and that the required volume of reactor is reduced by 1/5 than that of aeration tank. MLSS and DO maintenance does not require for SAFF technology. Because of that, this (SAFF) technology becomes more feasible either economically or technically.

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http://www.keepprocess.com

http://pumpschool.com

http://swagelok.com

Chapter 9: Summary of results

Table- 26 Summary of bar screen design

S. No.	Parameter	Value
1	Peak flow rate	$3.45\times 10^{-3} m^3/s$
2	Area of opening of screen channel	5.75×10^{-3} m^2
3	Number of bar	7
4	Total width screen channel	0.15m
5	Velocity of flow in screen channel	0.31m/s
6	Length of screen channel	0.73 m
7	Head loss through in screen channel	0.0196 m

Table- 27 Summary of oil and Grease trap (O&G trap)

S. No.	Parameter	Value
1	Volume of tank	$2m^3$
2	Length	2m
3	Width	1m
4	Standard depth	1m

Table – 28 Design Summary of equalization tank

S. No.	Parameter	Values
1	Volume of tank	66.6 m^3
2	Depth of tank	3 m
3	Length of tank	5 m
4	Width of tank	4.50 m
5	Free board	0.4 m
6	Air supplied	53.28 m^3/ hr

Table-29 Summary of primary sedimentation tank design

S. No	Parameter	Value
1	Area of tank	4.16 m^2
2	Length of tank	2.03m
3	Width of tank	2.03 m
4	Depth of tank	2.5 m
5	Total volume of sedimentation tank	9.39 m^3
6	HRT	2.25 hr
7	Sump dimension	0.5m× 0.5m×2.5 m

Table-30 Summary of aeration tank design

S. No	Parameter	Value
1	Volume of tank	67.5 m^3
2	HRT	16.2 hr
3	Depth of tank	3.5 meter
4	Area of tank	19.2 m^2
5	Length	2.75 m
6	Width	7 m
7	O_2 required	1.71 Kg/ hr
8	Air requirement	8.14Kg air/ hr
9	Number of diffuser	5
10	Volume air	62.55 m^3 /hr
11	Blower capacity	5 HP

Table- 31 Dimension of secondary sedimentation Tank

S. No	Parameter	Value
1	Depth of tank	3.5 meter
2	Length	2 meter
3	Width	2 meter
4	Volume of tank	11.25 m^3
5	HRT	2.70 hr
6	Sump dimension	0.5m×0.5m×3.5m

Table-32 Summary of digester tank design

S. No	Parameter	Value
1	Volume of digester tank	7 m^3
2	Height of digester	3 m
3	Diameter of digester	1.80 m
4	Free board	0.5 m

Table-33 Summary of sludge drying bed design

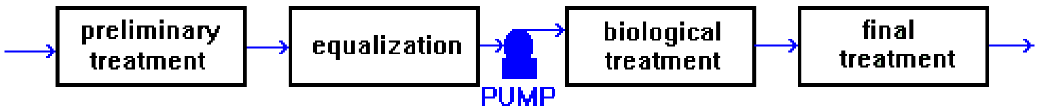
S. No	Parameter	Value
1	Total plane area sludge	25 m^3
2	No. of bed	7
3	Area of bed	3.5 m^2
4	Length	1.90 m
5	Width	1.90 m
6	Total volume of digester	35 m^3
7	Free board	0.5 m

Table-34 Summary of result SAFF reactor design-

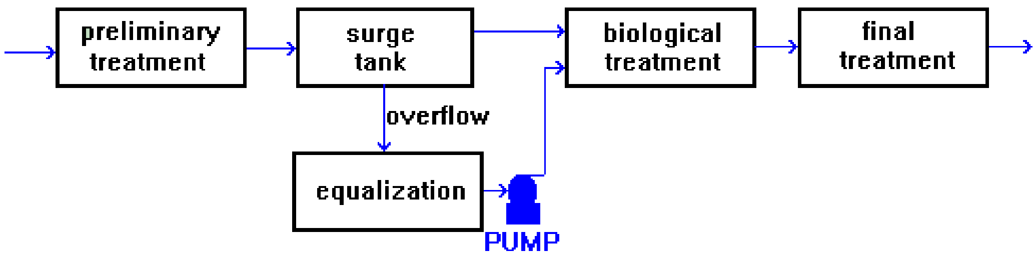
S.No	Parameter	Value
1	Volume of tank	12 m^3
2	Depth of tank	3m
3	Area of tank	4 m^2
4	Width of tank	1.5 meter
5	Length of tank	2.5 m
6	Depth of media of reactor	2.5 m
7	Length of media	2.5 m
8	Width of media	1.5 m
9	Volume of media	9. 375 m
10	O_2 requirement	1.71 Kg/ hr
11	Air requirement	8.14Kg air/ hr
12	Number of diffuser	5
13	Efficiency of diffuser	10%
14	Volume of air provide	62.55 m^3 /hr
15	Blower capacity	5 HP

Chapter 10: Figures & Diagrams

Figure 1: Inline/Offline equalization



Inline equalization flow diagram



Off line equalization flow diagram

Figure 2: Determination of volume of equalization tank

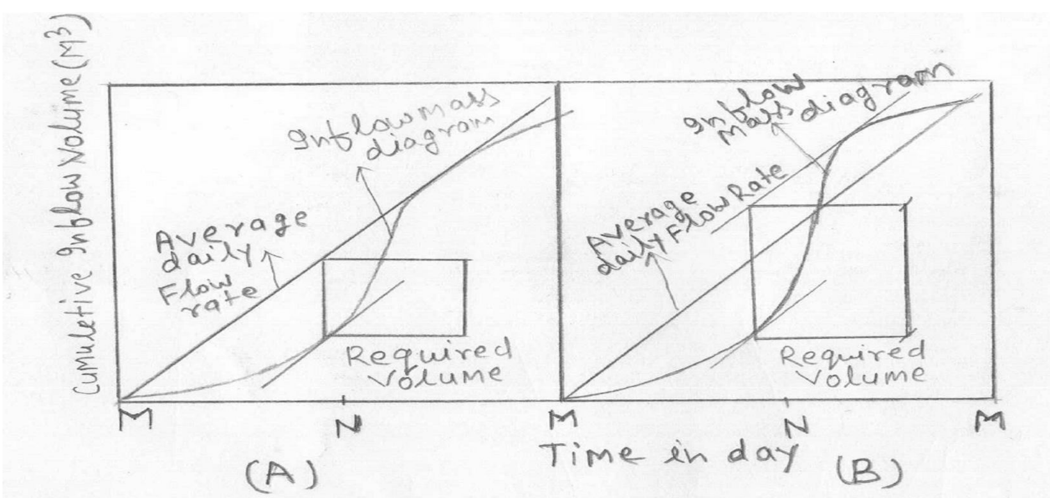


Figure: 3 Rectangular Sedimentation Tank

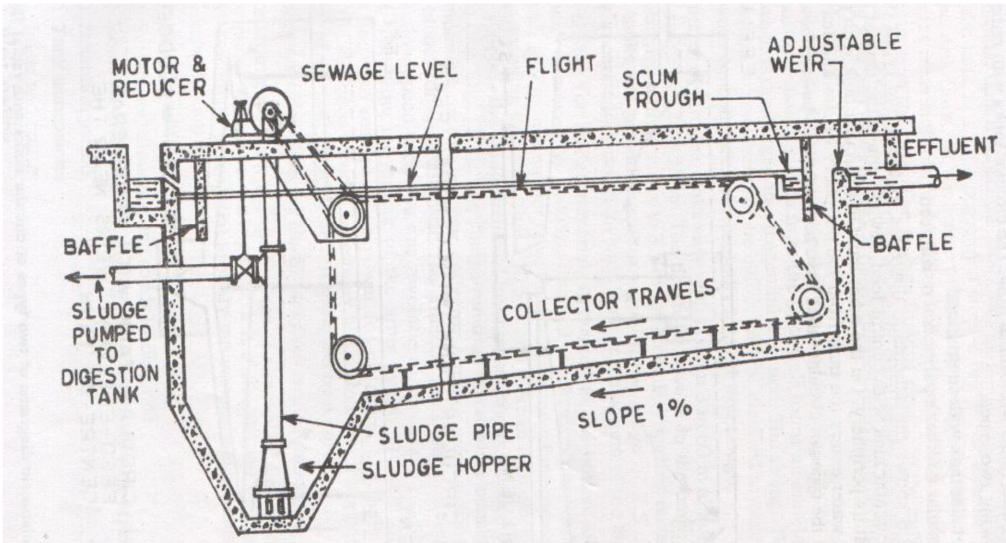


Figure4: Flow Diagram for Aeration tank

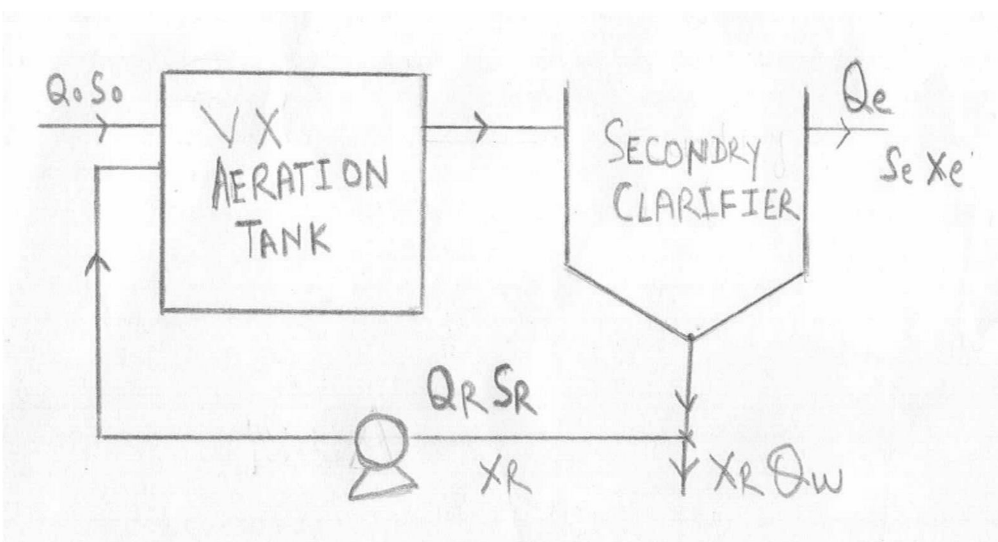


Figure 5: Anaerobic sludge digester

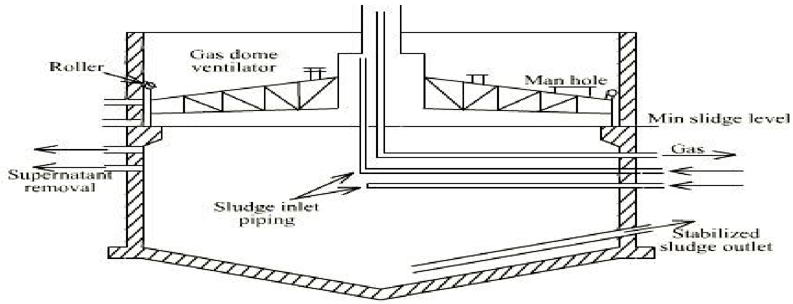


Figure 6: Types of anaerobic digester

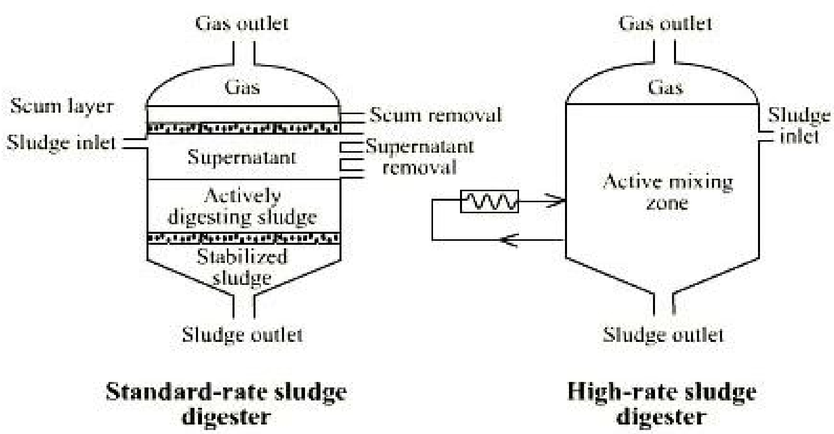


Figure: 7 Sludge Drying Bed

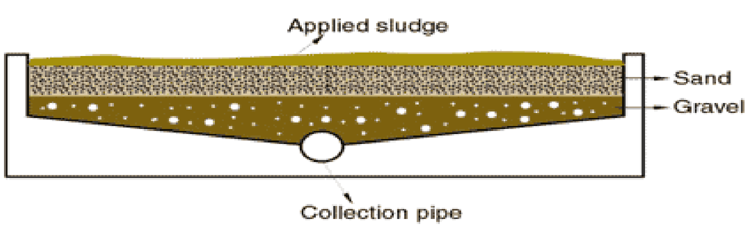


Figure: 8 Flow Diagram of Extended Aeration

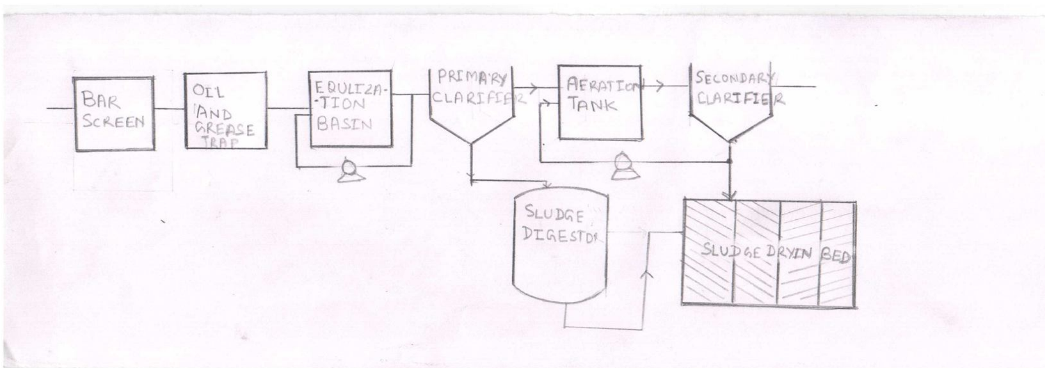


Figure: 9 Flow Diagram of SAFF Technology

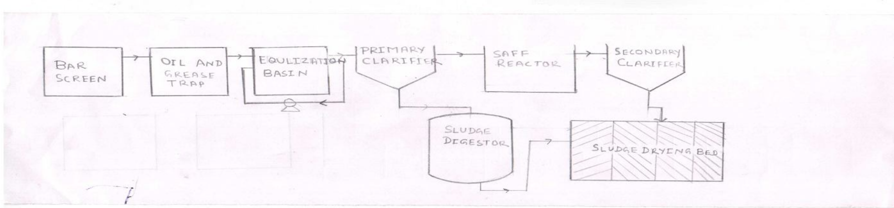
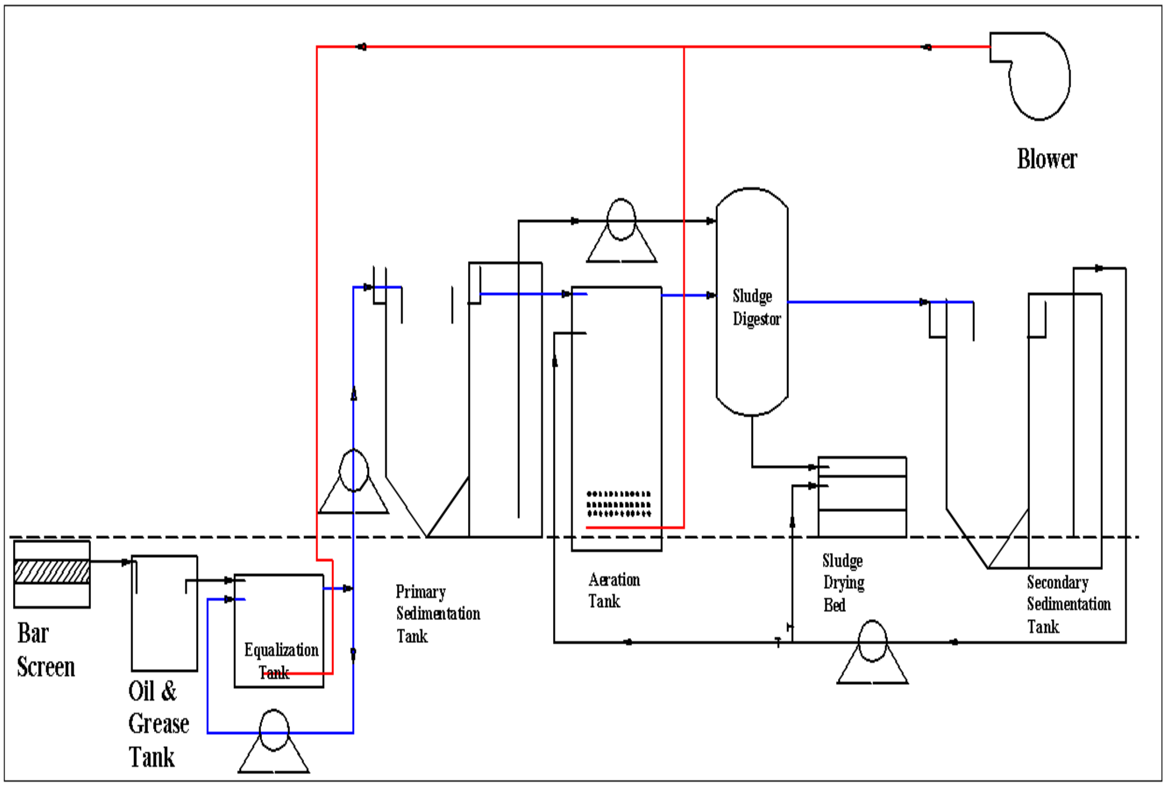


Figure 10-Flow diagram of designed STP with dimension



Dimension of bar screen
Spacing between bar = 10 mm
Angle of inclination = 45^{θ}
Total with of the screen = 0.15 m
Length of screen channel = 0.73m

Dimension of oil and grease trap
Depth = 1 m
Length = 2m
Width = 1m

Dimension of Equalization tank
Depth = 3 m
Length = 5 m
Width = 4.50 m

Dimension of sludge digester
Height= 3.0 m
Diameter = 1.80 m

Dimension of aeration Tank
Depth = 3.5 m
Length= 2.75 m
Width = 7 m

Dimension primary sedimentation tank
Depth = 2.5 m
Length = 2.03 m
Width = 2.03 m
Dimension of Sump = 0.5×0.5×2.5 m
Slope = 45^{θ}

Dimension of sludge drying
Length = 13.30 m
Width = 1.90 m
Depth = 1.4 m

Dimension of Secondary sedimentation tank
Depth = 3.5 m
Length = 2.0 m
Width = 2.0 m
Dimension of sump= 0.5×0.5×3.5 m