32. SEWAGE /WASTEWATER TREATMENT TECHNIQUES (Primary, Secondary and Tertiary

32.1 Purpose of Sewage Treatment

The raw sewage must be treated before it is discharged into the river stream. The extent of required treatment of wastewater / sewage depends following consideration

i) Characteristics and quality of the sewage

ii) Quality of receiving water body

iii) Assimilative capacity of receiving water body to tolerate the impurities present in the wastewater /sewage effluents

iv) Intended use of receiving water body

The purposes of the wastewater treatment based on these considerations are as following

a) To stabilize the sewage without causing odour and nuisance

b) To protect public health

c) To avoid potential damages to quality of ultimate receiving water body (Rivers, canals & Streams)

d) To protect the aquatic life from the potential threats posed by untreated wastewater.

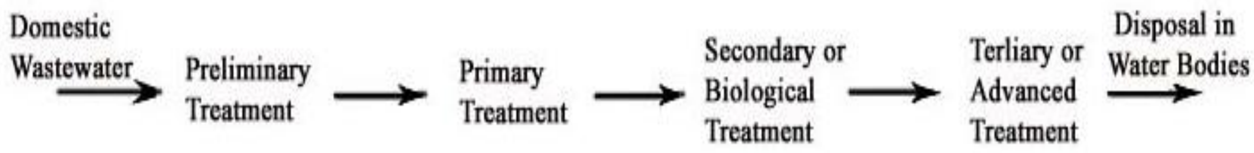
32.2 Steps in Sewage Treatment

There are three main steps in for sewage treatment

1. Separation of Solids from liquids
2. Treatment and disposal of liquid
3. Treatment and disposal of solids

33. WASTEWATER TREATMENT METHODS

The layout of conventional wastewater treatment is as follows:



33.1 Preliminary Treatment

Wastewater contains varying quantities of floating and suspended solids, some having considerable sizes. Material such as rags, pieces of wood, plastics, metals, rubber or fragments of masonry enter in sewer and eventually may reach the treatment plant. These constituents necessary to remove as their presence will interfere the treatment processes and have potential to cause damage to the installed machinery e.g. pumps and motors etc.

The process of removal of rags, pieces of wood, plastics, metals, rubber or fragments of masonry is called preliminary treatment.

Preliminary treatment is done through physical methods to remove coarse, suspended matter, floating materials and oil and grease in the sewage / wastewater. Treatment units / methods involved in the preliminary treatment are

33.1.1 Balancing /Equalization Tank

33.1.2 Screens

33.1.3 Cutting Screens or Comminutors

33.1.4 Grit Chambers

33.1.5 Skimming Tanks / Floatation

33.1.6 Evaporation

33.1.2 Screening

A screen is a device with openings for removing bigger suspended or floating matter in sewage which would otherwise damage equipment or interfere with satisfactory operation of treatment units.

i) Types of screens

There are two types of screens

a) Bar Screen or Racks

These can be further of two types

- Manually / Hand Cleaned (used on small sewage treatment plants)
- Mechanically Cleaned (These are used on large sewage treatment plants)

b) Mesh Screen /Fine Screens

These are not generally for screening of sewage. These are used for industrial wastewater where particle to be removed mostly have uniform size as compare to sewage.

ii) Design Considerations

Parameter	**Types of Screen**	
	Manually Cleaned	**Mechanically Cleaned**
Bar width(mm)	6 – 15 (1/4” – 5/8”)	6 – 15 (1/4” – 5/8”)
Bar Spacing (mm)	25 -50	15 – 75
Approach Velocity (m/s)	0.3 – 0.6	0.6 – 1.0
Velocity should not be more than 1 m/Sec to avoid Excessive Head Loss and should not less than 0.6 m/Sec to avoid Sedimentation in Screen Chambers		
Slope	$<30^0$	$>75^0$
Length	Approachable	
Location	Before pumps	Before pumps
Number of Screens	02	02
Surface Area perpendicular to flow	2 times the area of sewer	
Screened material (m^3/m^3 flow)	$1.6x\ 10^{-6} – 1.5x10^{-6}$	
Nature of Screened material	Contaminated with Faecal Matter	
Disposal of Screened Material	Landfill or Incineration	

33.1.3) Comminutors

Rather than removing the large size particles, communitors reduce them in size so that will not harm the equipment. The chopped or ground solids are removed in the sedimentation process.

Comminutors are provided with fixed screen and moving cutter. Some floating comminutors draw the flexible material through screen rather than chopping them. This can create nuisance in un-skimmed clarifiers, trickling filters and aeration basin.

Comminutors are selected on the basis of flow. For small plant single unit rated for peak flow may be used in parallel with manually cleaned bar rack. For larger plants multiple number of units are used with capacity so that remaining units may handle the peak flow, with one or two out of service.

Head loss depends upon the screen details and flow and ranges from 50 -100 mm

33.1.4) Grit Chambers

The Suspended Solids in wastewater/sewage consists of inert inorganic materials such as sand, metal fragments, eggshells etc.

Grit Chambers are used to remove the inert inorganic particles (sand, metal fragments and eggshells etc) from sewage having size 0.2mm or larger and specific gravity equal or more than 2.65

i) Purposes of Grit Chambers

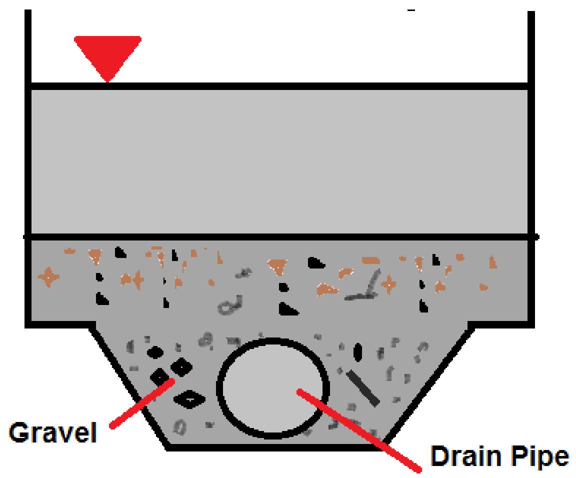
a) To protect moving mechanical equipment from abrasion and abnormal wear.

b) To avoid heavy deposition in pipelines, channels and conduits.

c) To avoid frequent cleaning of digesters to remove excessive deposition

ii) Design Considerations

Parameters	Value
Velocity (m/s)	0.25 – 0.3
Detention Time (Sec)	40 – 60
Length (m)	10 – 20
Cleaning Intervals (Weeks)	2
Velocity Control	Through Proportional Flow Weir OR Sutro Weir



33.1.5) Skimming Tank

The Skimming tanks are used where the sewage contain excessive oil and grease. A basin with detention time of about 10 minutes is satisfactory. The tank generally includes in aerating device which blows air through the sewage at a rate of about 0.1 cubic feet of air/gallon at a pressure ranging from 40 – 50 Psi. The rising air tends to coagulate the grease and oils and cause them to rise and cause them to rise to the surface, where they can be removed easily either manually or mechanically.

33.2 Primary Treatment

Primary treatment is intended to remove settle-able suspended organic solids and floating materials. Primary treatment also reduces the load on subsequent biological units. Chemicals are also sometimes added in primary clarifiers to assist in removal of finally divided and colloidal solids, or to precipitate phosphorous.

This treatment is done through mostly by physical methods to remove settle-able and suspended solids. The unit involved in this treatment process are

33.2.1 Sedimentation tanks
33.2.2 Septic tanks
33.2.3 Imhoff tanks

33.3.1) Sedimentation Tanks

Plain Sedimentation is the separation or removal of suspended solids from the wastewater by gravity. If the specific gravity of the suspended particle is more than 1, it will settle with certain velocity under gravity.

Therefore if water is allowed to remain in a basin or tank for certain time, the suspended particles will settle and reach at the bottom of the tanks and thus the wastewater will become clear. A sedimentation basin may also be referred as sedimentation tank, settling basin or settling tank.

i) Principle of Sedimentation

Discrete Particle Settling

It refers to the sedimentation of particles in a suspension of low solid concentration. Particles settle as individual entities and there is no significant interaction with neighbouring particles. It removes grit and sand particles form water. There particles settles under “Stokes’s law”

$$Vs = \frac{g}{18\mu}(ps - p)d^2$$

Where

Vs	=	terminal settling velocity (L/T)
ps	=	the mass density of particles (M/L^3)
p	=	The mass density of fluid (M/L^3)
g	=	Acceleration due to gravity (L/T^2)
µ	=	Absolute viscosity of the liquid (M/LT)

The above equation can be written as

$$V_s = \frac{g}{18v}(G_s - 1)d^2$$

Gs = Specific Gravity of the particles

v = Kinematic viscosity

The above law is applicable when $R_N < 1$

ii) Ideal Sedimentation Basins

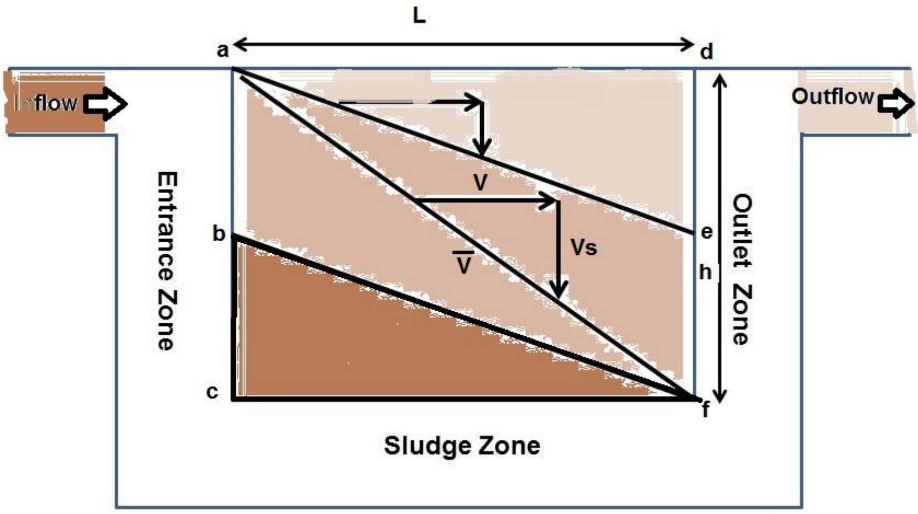
The design of the sedimentation basin is based on concept of Ideal Sedimentation Basins. Assumptions made for design of Ideal Sedimantation Basins are as followings:

a) Complete mixing and uniform suspension at the inlet zone.

b) Uniform horizontal velocity in the settling zone.

c) No flocculation (Discrete free settling)

d) Particles that reach at bottom are permanently removed.



iii) Surface Overflow Rate (SOR)

The particle entering the basin will have a horizontal velocity equal to the velocity of the fluid

V = Q/A = Q/wh

w = Width of tank & h is depth of tank

The other velocity is settling velocity "Vs" of that particle defined by Stoke's law. If the particle is to be removed, the settling velocity (Vs) and horizontal velocity (v) must be such that their resultant velocity ($\bar{V}$), must carry it to the bottom of the tank before outlet zone is reached. If the particle entering at the top f the basin (point a) is so removed, all the particles with same velocity will also be removed.

Consider slope of the velocity vector from point "a" to "f" and dimension of the basin itself. We can write

$$\frac{Vs}{V} = \frac{h}{L}$$

Or

$$Vs = \frac{Vh}{L} \qquad V = Q/A = Q/Wh$$

$$Vs = Q/WL$$

Hence Surface Overflow Rate (SOR)is defined form above equation, which is numerically equal to the flow divided by the plan area of the basin but which physically represents the settling velocity of the slowest settling particles which is 100% removed. Those particles which settle at velocities equal to or greater than SOR will be entirely removed.

iv) Detention Time

It is defined as the tank volume divided by the flow

Detention Time (DT) = V/Q

v) Types of Sedimentation Tanks

Primary sedimentation tanks remove suspended particles upto 50 – 60%. BOD removal in these tanks is associated with removal of suspended solids which normally ranges from 25 – 40 %. Sedimentation tanks are of following types

a) Rectangular Tanks

b) Circular Tanks

c) Square Tanks

a) Rectangular Tanks

Rectangular basin employ a horizontal flow pattern with the flow along the long axis. Such a flow pattern minimizes the effects of inlet and outlet disturbances.

- **Design Criteria**

Sr.No.	Parameter	Value
i)	Design flow	Average Daily Flow
ii)	Settling Velocity of Particles	0.3 – 0.7 mm/s
iii)	Surface Overflow Rate(SOR)	25 – 40 m/day
iv)	Detention Time (DT)	1.5- 2.5 hour
v)	Depth	2-5 m
vi)	Maximum Length	30 m
vii)	L : W (ratio)	4 : 1
viii)	Depth	2 – 5 m
viii)	Weir Loading	Not more than 120 m^3/m.day
vix)	Sludge Accumulation	2.5 Kg wet solids / m^3 flow
	Secondary Sedimentation Tank	
i)	Design Flow	Average Daily Discharge
ii)	SOR	30-40 m/day
iii)	Detention Time (DT)	2 – 3 hours
iv)	Depth	2.5 – 5 m

b) Circular Tank

In circular Tank the flow may either at periphery or at the centre. It has been observed that average detention time in peripheral feed basin is greater. Generally diameter of 30 m is provided. However diameter more than 100 m is not recommended.

c) Square Basin

Square basin may be used in situations where land area is limited. The length of one side may be 21 m.

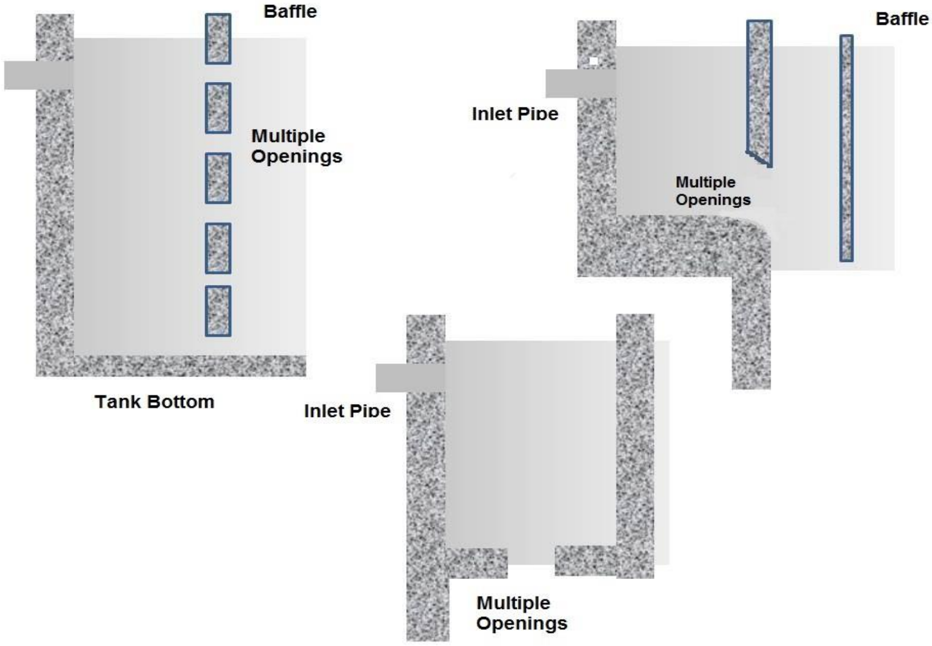
vi) Inlets and outlets

Careful design of inlets and outlets is important to assure reasonable performance of sedimentation tanks.

a) Inlets

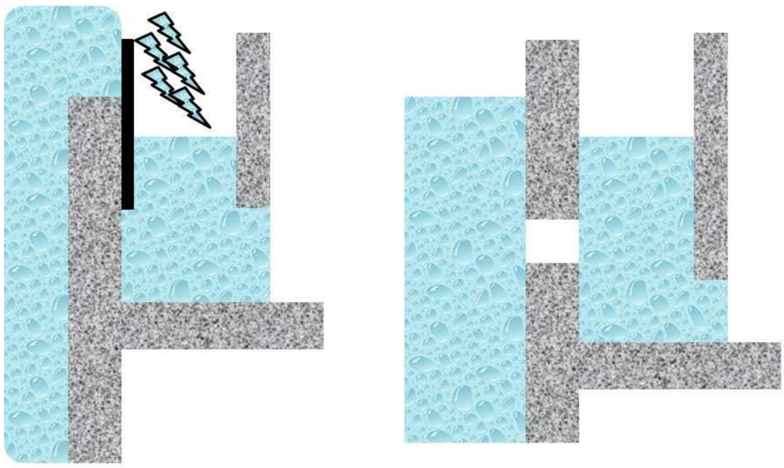
A proper inlet design of a sedimentation tank provides following advantages.

- It reduces the entrance velocity of effluent
- It distributes the water though out the width and depth of tank and
- It mixes it with water already present in the tank to prevent density currents.



b) Outlets

In sedimentation tanks outlets are consist of free-falling weirs. Weir loading rates are limited to prevent high approach velocities near outlets. Weir loading rates are specified in volume /unit length per day, which are used to calculate the length of the weir but not length over which overflow occurs. Outlets are placed as far as possible from the inlets i.e. at the end of rectangular tanks and towards centre and along the radii of peripherally fed tanks. Weirs frequently consist of v-notches approximately 50 mm in depth and placed at 300 mm on centre.



Problem: *Calculate the settling velocity of a sand particle of o.4 mm in size in water at 10 °C. Specific gravity of sand particle is 2.65. How much surface area of an ideal settling tank will be required to remove these particles? Dynamic viscosity of water is $1.31x10^{-2}$ poise. The incoming flow is 28512 m^3/day*

Solution

D	=	0.04 mm		0.004 cm
S.G	=	2.65		
Viscosity (μ)	=	$1.31x10^{-2}$ poise (gm/cm-sec)		
Q	=	28512 m^3/day		
Vs	=	?		
A	=	?		

$$Vs = \frac{g}{18v}(Gs - 1)d^2$$

$$V = \underline{\mu/p} = 1.31x10^{-2} / 1 \qquad p = 1\ gm/cm^3$$

$$Vs = \frac{9.81(2.65-1)(0.004)^2}{18x1.31x10^{-2}} = 1.098 \times 10^{-3}\ m/sec$$

Now for 100% removal SOR should be equal to the settling velocity i.e.

SOR	=	Vs		
	=	Q/wL	=	1.098×10^{-3}
Surface Area	=	wL	=	2852 / (1.098×10^{-3}x24x60x60)
			=	300.5 m^2

Problem: *A sedimentation basin is 30 m long, 15 m wide and 4m deep and has an overflow rate of 24 m^3/day-m^2. What is detention time?.*

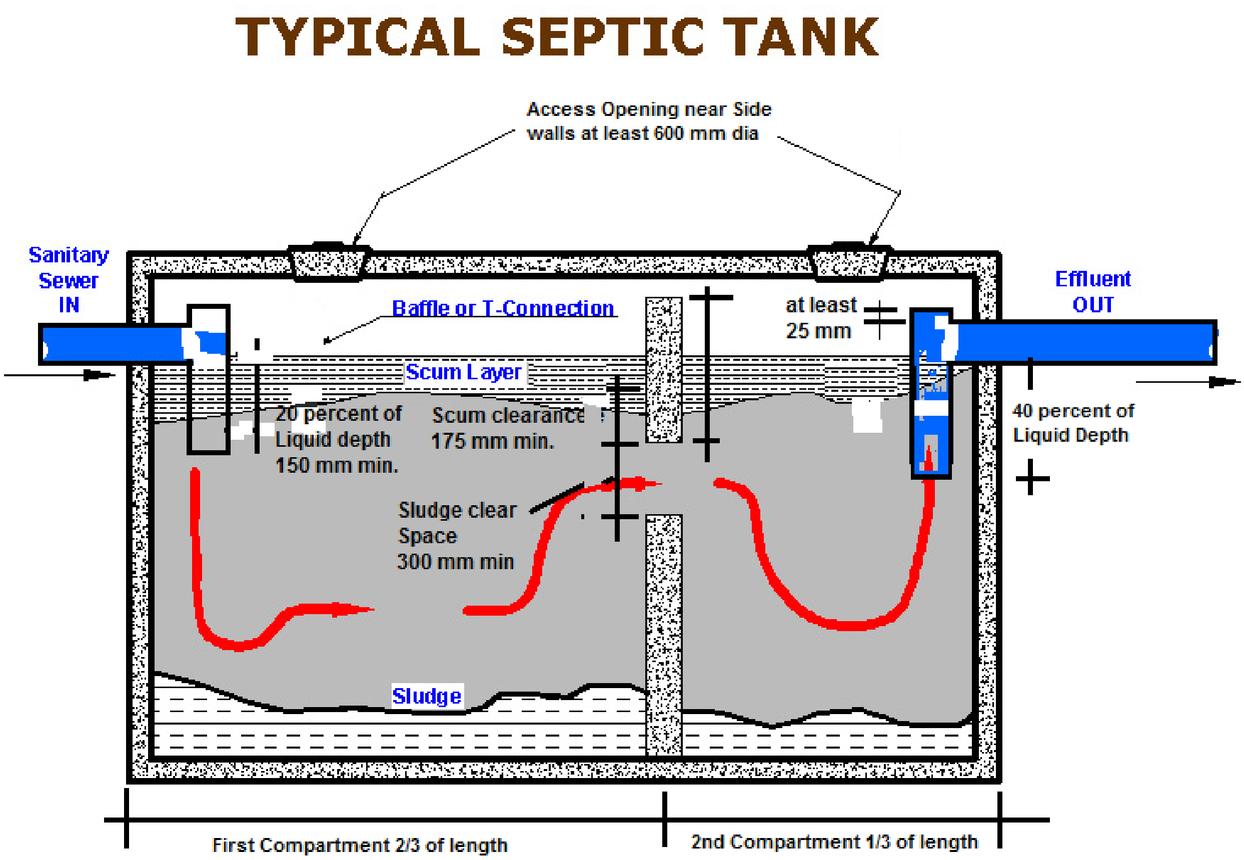
Solution

L = 30m	W = 15m	H = 4 m		SOR = 24 m^3/day-m^2.	DT = ?
DT	V/Q				
V(vol)	=	30 x 15 x 4 m^3		1800 m^3	
Also					
SOR	=	Q/WL			
Q	=	SOR x WL	=	24 x 15x30 m^3/day	10800 m^3/day
DT	=	V/Q	=	1800 / 10800 day	0.1667 day
DT	=	4 hours			

33.2.2) Septic Tanks

Septic Tanks are Primary Sediments basin although a minor degree of solid destruction may occur due to anaerobic digestion. Units are ordinarily sized to provide detention time of 24 hours at average daily flow.

The tanks are usually made of concrete but steel and fibreglass are used. The effluent of a septic tanks is offensive and potentially dangerous.. The mean BOD concentration observed in the number of septic tanks ranged from 120 – 270 mg/l and mean Suspended Solids from 44 – 69 mg/l. Effluent of septic tank required further treatment, either in an additional process or by soil disposal.



33.2.3) Imhoff Tank

Imhoff tanks incorporate solid separation and digestion in single unit. Modern version of the process employ mixing and heating in the digestion zone. Advantages of these units over septic tanks is questionable. These tanks have been widely used in the past, particularly small plants designed to provide primary treatment. Old Imhoff tanks have been converted to other uses in recent years.

33.3 Secondary Treatment

Secondary treatment systems are advanced wastewater treatment process intended to remove constituents of wastewater by using biological wastewater treatment process, which remain even after primary treatment. They employed for

- Soluble Organic matter (BOD)
- Colloidal Suspended Solids by improving sedimentation process
- Removal of Phosphorous Nitrogen and Phenolic compounds etc

33.3.1 Physical and Chemical processes

i) Coagulation or Chemical Precipitation

Wastewater generated from urban areas having industry may also contain very fine particles and heavy metals which may not settle in the reasonable time or stabilize by the microorganisms. Under these condition certain chemicals are added to wastewater to be treated to achieve following objectives

a) To improve the performance of sedimentation tank

b) To improve the performance of filtration.

These chemicals are known as coagulants. These chemicals form precipitates in wastewater which catch the tiny particles and ions of heavy metals and increase in size through process of agglomeration and thus settle down in the sedimentation tanks.

Most commonly used chemicals include Alum (Aluminium Sulphate), Ferric chloride, Ferric Sulphate, Sodium Dichromate, Ferrous Sulphate and Lime. The most commonly used coagulant is Alum.

Effective dose for higher removal efficiency for a particular waste is determined in laboratory through “Jar test”. For removal of colloidal particles by adding coagulants, the wastewater is subjected to rapid mixing, flocculation and final removal in sedimentation tank.

ii) Adjustment of pH

Adjustment of pH is important. The pH of the wastewater has much effect on the efficiency of the coagulation. The optimum pH values for various coagulants is as followings:

Coagulant	Range of pH
Alum	4.0 – 7.0
Ferrous Sulphate	3.5 and above
Ferric Chloride	3.5 – 6.5 and above 8.5
Ferric Sulphate	3.5 – 7.0 and above 9.0

Removal efficiency of Suspended Solids (SS), Organic Material and Bacteria through chemical process /treatment vary and given as below:

Suspended Solids (SS)	80 – 90 %
Organic Matter	50 - 55 %
Bacteria	80 – 90 %

iii) Neutralization

Neutralization is a process in which acid or base is added to neutralize the effluent. Neutralization is necessary for successful performance of biological units as most the microorganisms are sensitive towards the pH values of the wastewater to be treated.

Dose of acid or base to be used for this purpose is determined in the laboratory.

33.3.2 Biological Treatment

Secondary treatment is usually understood to employ biological treatment process. Wastewater in addition organic matter also contains large number of microorganisms which are able to stabilize the waste in the natural purification process. In this process the microorganisms remove soluble and colloidal organic matter from the waste. In order to carry out this natural process in a reasonable time, large number of microorganisms should be available in relatively small tank/container. Basic principle of all secondary treatment process is same but the methods/techniques may vary. These processes are designed as

i. Suspended Growth process

ii. Attached Growth process

i. Suspended Growth Process

This is done to remove organic solids through biological processes. Suspended growth processes maintain adequate biological mass in suspension within the reactor by employing either natural or mechanical mixing. In most processes the required volume is reduced by returning bacteria form the secondary clarifiers (or sedimentation) tank in order to maintain a high solid concentration. The suspended growth processes include

1) Activated Sludge Process
2) Oxidation Ponds
3) Aerated Lagoons
4) Sludge Digestion Process

1) Activated Sludge Process (ASP)

This process was devised by Arden and Lockett in Manchester in 1914.

Activated sludge is a sludge flock (i.e. body of microorganisms gathered in a crowd) produced in raw or settled sewage by the growth of bacteria and other organisms in the presence of dissolved oxygen and accumulated in sufficient concentrations by returning sludge flock previously formed.

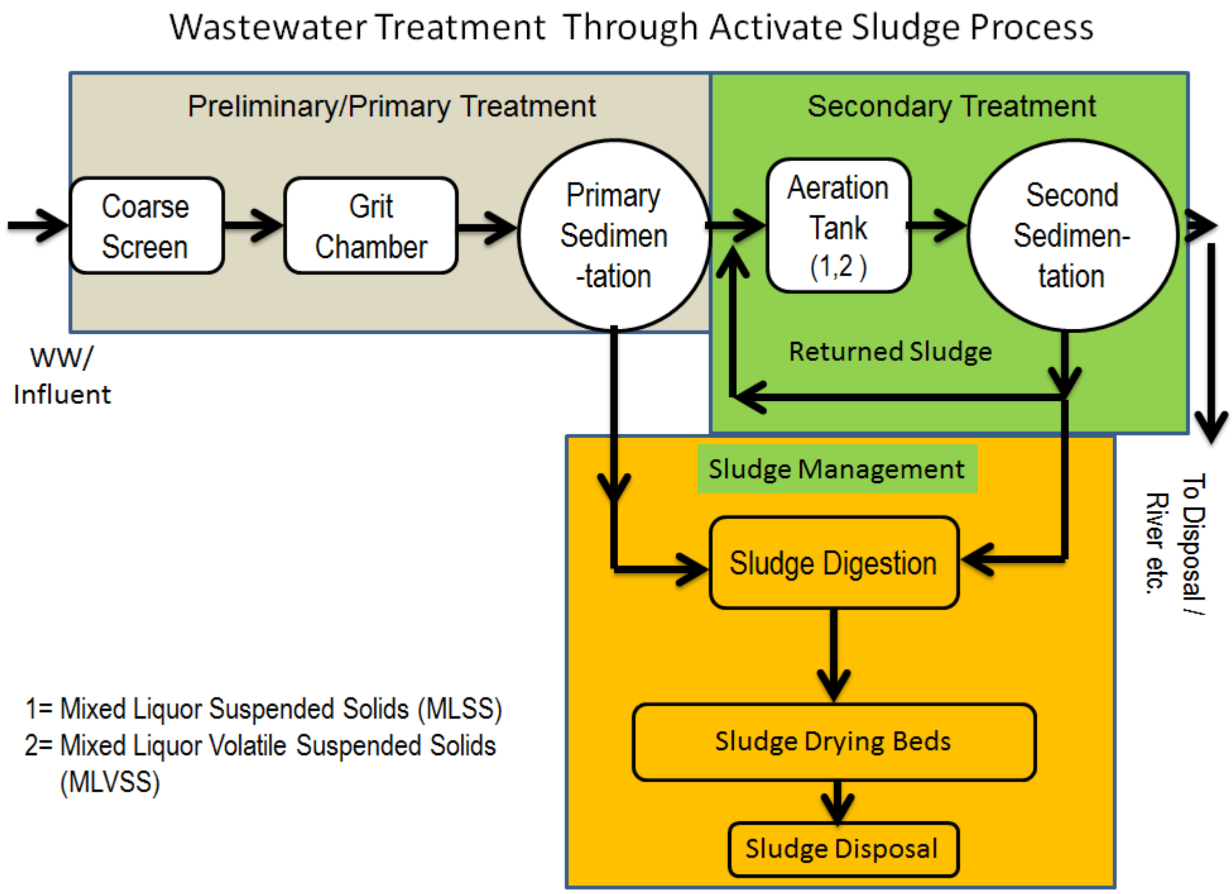
In this process a mixture of sewage and activate sludge is agitated and aerated in an Aeration Tank. Bacteria present in the activated sludge aerobically metabolize the Organic Matter present in the influent. The Organic Matter is oxidized to Carbon Dioxide (CO_2), Water (H_2O) and ammonia (NH_3) etc. and a portion of it is converted into new Bacterial Cells.

Organic Matter ________________ $CO_2 + H_2O +$ NH_3 & A portion into Bacteria (New)

The activated sludge is subsequently separated from the Mixed Liquor (It is mixture of sewage and activated sludge in aeration tank) by sedimentation in the final clarifiers and wasted or returned to the aeration tank as needed. The treated effluent overflows from the final clarifier.

Sometimes wastewater is subjected to Modified Activated Sludge Process. It is modified form of Conventional Activated Sludge Process. The only difference is less aeration period. Therefore less microorganisms will be maintained in the tank which will need less and ultimately less power consumption will also less. In this case BOD removal is 60 – 70 % whereas in the conventional ASP, it is 90 – 95%.

The degree of treatment in Activated Sludge Process depends upon the settle- ability of sludge in the final clarifier. The biological floc settles by gravity and leaves a clear supernatant for disposal. However, if Filamentous Microorganisms grow in the aeration tank, they do not settle by gravity and contribute to Biochemical Oxygen Demand (BOD) and Suspended Solids (SS). Excessive carryover of floc, resulting in the Inefficient Operation of final clarifier is referred as Sludge Bulking.



1.1 Advantages of Recirculation

The advantages of sludge re-circulation include

i) An increase in biological solids in the system and continuous seeding with re-circulated sloughed solids.

ii) Maintenance of more uniform hydraulic and organic load,

iii) Dilution of influent with better quality water and

iv) Thinning of biological slime layer

Recirculation may not exceed the efficiency in all circumstances particularly with relatively dilute wastes. Recirculation rates ranges from 50 – 100 % of the wastewater with usual rates being 30 – 50 %

1.2 Condition Promoting Growth of Filamentous Organisms

Condition Promoting Growth of Filamentous Organisms which result in sludge Bulking are as following:

i) Insufficient Aeration

Insufficient aeration which results in DO level less than 2 mg/l in wastewater. Sludge floating due rising Nitrogen gas. In the absence of O2, bacterial will convert ammonium to nitrate and finally into Nitrogen gas which will make sludge to float

ii) Lack of Nutrients (Nitrogen and Phosphorous)

Lack of nutrients including Nitrogen (N)) and Phosphorus (P). Problem may be corrected by providing lacking nutrients. Guiding values of BOD and nutrients namely Nitrogen and Phosphorous are of the order of 100:5:1

iii) Low values of pH

Low pH promotes fungal growth in the basin. Neutralization and pH adjustment may cover this problem

iv) Overloading

High food to microorganisms ratios may also results in excessive growth of Filamentous Microorganism and thus bulking of sludge.

1.3 Food to Microorganism Ratio (F:M)

Food to Microorganism Ratio (F:M) is expressed in terms of Kg of BOD applied per day per Kg of MLSS. If Q is the sewage flow in m3/day and it has a BOD expressed in mg/l then

$$\text{Food} = \frac{(Q \times BOD)}{1000} \quad \text{Kg BOD/day}$$

If V is volume of the aeration tank in cubic meter (m^3) and it has an MLSS concentration expressed in mg/l, then

$$\text{Microorganism} = \frac{(V \times MLSS)}{1000} \quad \text{Kg MLSS}$$

$$F:M = \frac{(Q \times BOD)}{(V \times MLSS)} \quad \text{per day}$$

$$\text{As } Q/V = t$$

$$F:M = \frac{(BOD)}{(t \times MLSS)} \quad \text{per day}$$

Where “t” is aeration time in days

1.4 Design Criteria

Design criterion for both conventional and modified Activated Sludge Process are as followings:

Parameter	**Value**
Aeration Period	4 – 8 hours
Air Required	90 – 125 m3/kg of BOD5
SS in MLSS	1500 - 2000 – 2500 - 3000 mg/l
F:M	0.5 – 1.0 however 0.25 to 0.5 per day is usually employed and it gives settling characteristics of sludge
Return Sludge	25% - 100 % of sewage flow
Dissolved Oxygen (DO)	2 mg/l
No. of Aeration Tanks	2 (as minimum)
Depth	3 – 5 m
L :W	5:1
Aeration Systems	
a)	Diffusers with pressure of 40 KPa
b)	Surface Aerators
Q_r/Q =	Vs/(1000 – Vs)
Where	
Vs =	Volume of Settled Sludge in "ml"
Qr =	Flow of Return Sludge
Q =	Flow of Sewage

1.5 Diffuser System

Major characteristics are as followings

Diffusers size	1500 dia Ceramics Domes
Bubble Size	2.0 – 2.5 mm
Distance between Diffusers	0.6 – 1.0 m
Location	Bottom of the tank
Operation	
	■ Noiseless
	■ Less formation of aerosols

1.6 Surface Aerators

Mechanical surface aerators are employed in the aeration tank. They spin partially in and partially out of the mixed liquor. The mixed liquor is violently thrown across the surface of the tank for adsorption of oxygen from the air.

Surface aerators require less maintenance and provide visual evidence of break down.

1.7 Measurement of Sludge Settle-ability

1.7.1 Sludge Volume Index (SVI)

Sludge volume index indicates the sludge settling characteristics. It is the volume measured in milliliters (ml) occupied by one (1) gram of settled suspended solids.

1.7.2 Determination of SVI

i) Draw sample of mixed liquor at the end of aeration tank

ii) Measure the volume of the settled sludge (Vs) in a one liter graduated measuring cylinder in 30 minutes.

iii) Measure the Suspended Solid (SS) content in mg/l of the mixed liquor samples which is called MLSS.

$$SVI = \frac{(Vs \times 1000)}{MLSS} \quad (ml/g)$$

Value of SVI	Sludge Characteristics
< 50	Thick and hard sludge difficult to pump
40 – 100	Good sludge
>200	Sludge has bulking tendency ie. Poor settling characteristics

1.8 Types of Activated Sludge Processes (ASP)

1.8.1 Conventional (Plug Flow)

This process consists of a rectangular basin, a clarifier, and a solid return line from the bottom of the clarifier. Exceed solids are usually wasted from the clarifier underflow. Separate sludge wasting from the reactor is possible and usually preferred. The return sludge are mixed with incoming waste and pass through in a plug flow fashion.

Air is provided uniformly along the length of the basin through porous diffusers.

i) Disadvantages

a) F/M ratio varies as the wastewater travels in the tank.

b) Air is provided uniformly along the length of the basin through porous diffusers.

c) Since there is high concentration of BOD as well as microorganisms, the oxygen at inlet of the tank is consumed rapidly due to raid exertion of BOD by microorganisms. Therefore, oxygen demand may be difficult to meet at inlet.

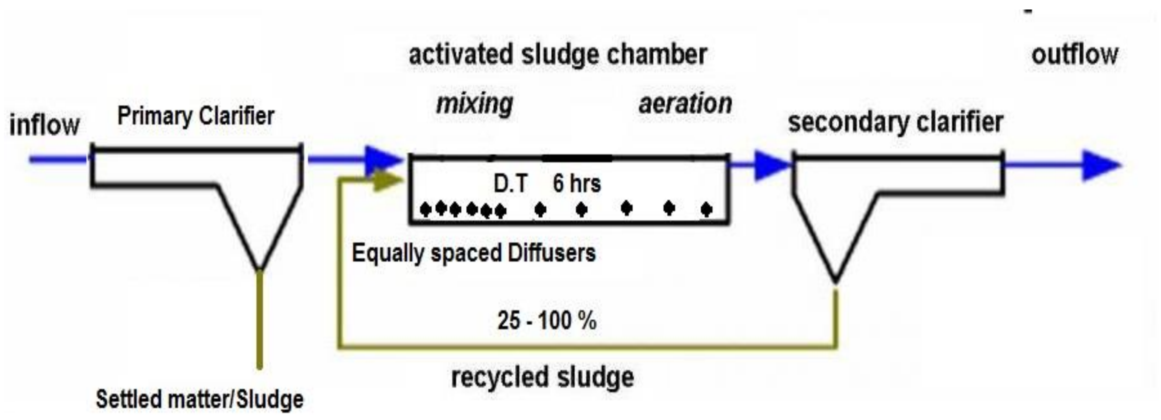
d) Air supplied may become excess on the other end of tank.

1.8.2 Tapered Aeration

The rate of oxidation in Conventional Activated Sludge is highest at inlet of the tank due to high concentration of microbial solids and BOD. Therefore, it may sometime become difficult to maintain aerobic condition throughout the tank length by providing uniform air distribution. Various studies indicated following Oxygen utilization by mixed liquor in 6 hours aeration time.

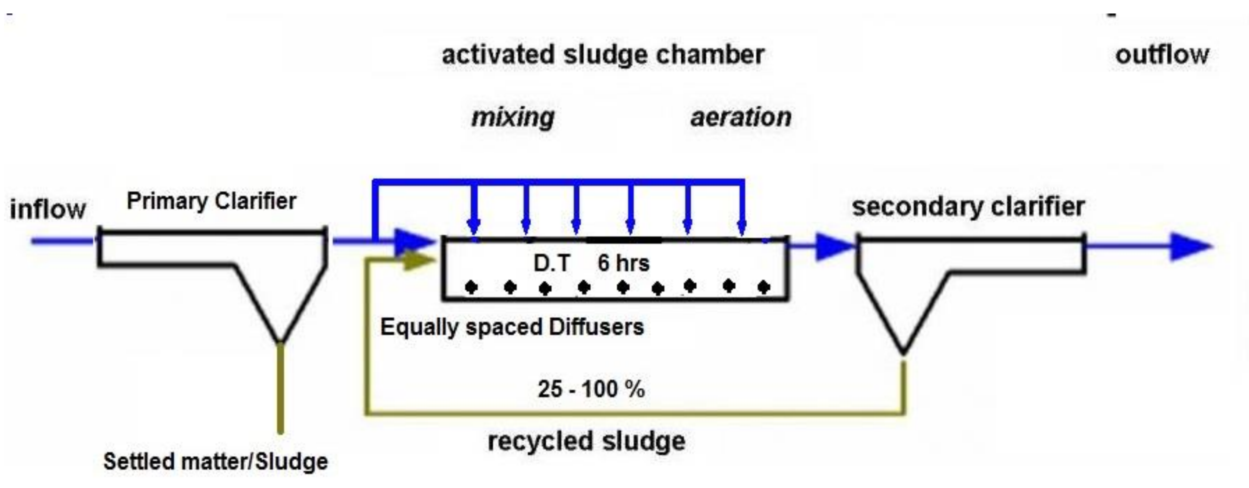
Time	BOD Removal (%)
Ist 2 hours	50
Next 2 hours	30
Next 2 hours	20

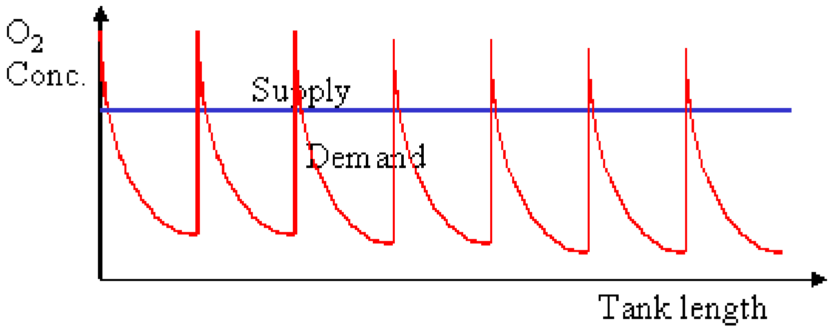
The tapered aeration system is similar to the conventional activated sludge process. However, in tapered aeration system diffused air is applied at varying rate along the length of the tank i.e. air supply is gradually reduced along the length of the basin. Although same volume of air is used but air is concentrated at the inlet to cope the high demand of microorganism there.



1.8.3 Step Aeration

This is based on the idea that organism are unable to handle the full load if all the sewage enters the aeration tank at a same point. Step aeration is addition of increments of sewage along line of flow of the aeration tank. Air supply is kept constant along the length of the tank i.e diffusers are equally spaced but incoming load (sewage) is distributed or added in steps along the length of the basin to have almost constant F:M ratio.





1.8.4 Extended Aeration

This is the same as complete mix, with just a longer aeration. Advantage - long detention time in the aeration tank; provides equalization to absorb sudden/temporary shock loads. The purpose of extended aeration is to oxidize the sludge to decrease its volume and consequently reduce the capacity (size) of the sludge digester. Less sludge is generally produced because some of the bacteria are digested in the aeration tank.

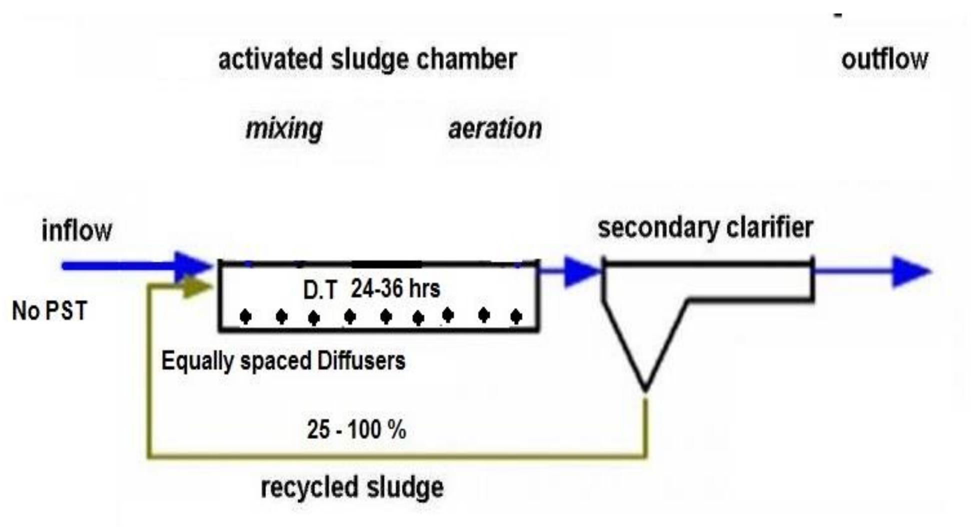
i) This method is suitable for small sewage flows (<1MGD or 0.04 m3/s)

ii) F:M ratio is $\simeq$ 0.05 to 0.1 per day

iii) Also used to treat industrial wastewater containing soluble organics that need longer detention times.

iv) Aeration time varies from 24 hours to 36 hours

v) Primary treatment (sedimentation tank) is not provided.



1.8.5 High Rate Activated Sludge Process

This type is usually employed where high Biochemical Oxygen Demand (BOD_5) and Suspended Solid (SS) removals are not required. F:M ratio is increased by reducing the size of the tank i.e.

F:M = BOD/(MLSS x t)

Due to high F:M ratio the biomass mostly remains in suspension in the effluent. Hence effluent quality is poor.

1.8.6 Completely Mixed Process

In this method the contents of the aeration tank (i.e. Influent sewage and return sludge) are completely mixed. This complete mixing is done to achieve constant F:M ratio though out the tank.

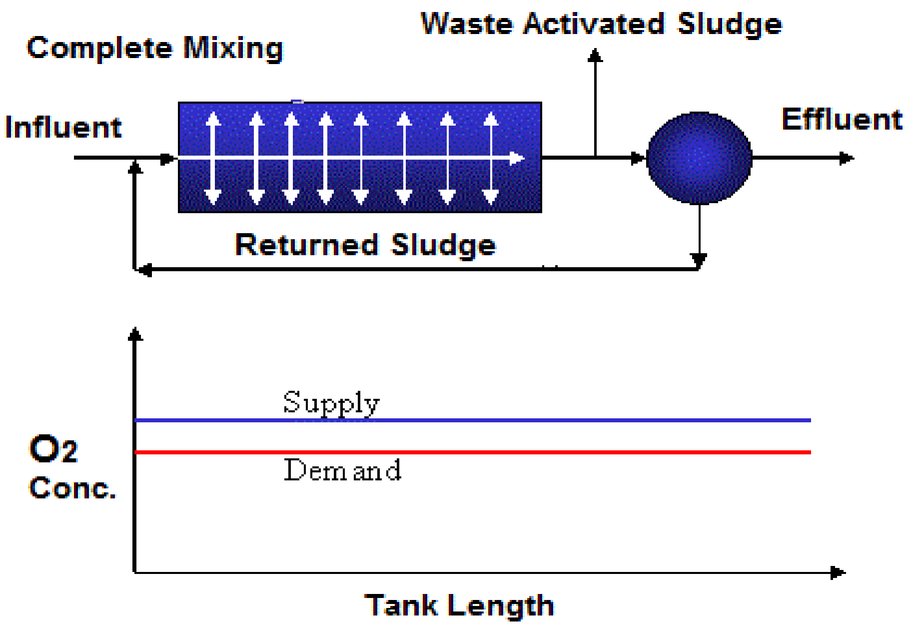
Diffusers or surface aerators are spaced equally in the tank to result in uniform supply of oxygen.

This process is most widely used.

i) Advantages

This process can effectively handle Shock Loads (i.e. Instant variations in influent BOD, temperature, pH etc.)

Complete Mixed Activated Sludge Process



1.9 Operational Control of Activated Sludge Process

The basic parameters used in operational control are given as below:

1.9.1 Dissolved Oxygen (DO)

Maintain 1-3 mg/l DO in aeration basin

1.9.2 Mixed Liquor Suspended Solids (MLSS)

70% of Mixed Liquor Suspended Solids

1.9.3 Sludge Volume Index (SVI)

Microscopic examination of sludge to check filamentous growth

1.9.4 Food : Microorganism (F:M)

1.9.5 Return Sludge

1.9.6 pH

a) range 6.0 to 9.0 Standard units

b) Ideal 6.8 to 7.4 Standard Units

1.9.7 Temperature

a) Colder water may require longer treatment times

b) Industrial discharges may increase the temperature

Most microorganisms do best under moderate temperatures (10-25ºC). Aeration basin temperatures should be routinely measured and recorded.

1.9.8 Nitrogen Content

1.9.9 Phosphorus Content

1.10 Advantages of Activated Sludge Process

i) BOD removal rates are high i.e > 95%

ii) Less land areas are required

iii) Odour free operation

iv) Suitable to treat industrial wastewater as well.

1.11 Disadvantages

i) Process and operation are extremely sensitive /sophisticated

ii) Successful operation requires skilled manpower or workforce.

iii) Sludge bulking problem is very common

iv) High operating cost

v) Electric power is required for operations

vi) Not suitable for developing countries facing shortage of power, revenue and skilled manpower

Problem: *The Mix Liquor Suspended Solids Concentration in an aeration tank is 2400 mg/l. The volume of the sludge after 30 minutes settling in one liter cylinder is 220 ml. Calculate Sludge Volume Index.*

Sol.

MLSS = 2400 mg/l
Vs = 220 ml/l
SVI = ?

$$SVI = \frac{(Vs \times 1000)}{MLSS}$$

$$= (Vs \times 1000) / 2400$$

$$= (220 \times 1000) / 2400 \quad 91.67$$

Problem: *Design an Activated sludge process system for wastewater flow of 20,000 m^3/day having BOD 200 mg/l and F/m ration as 0.4 and MLSS 2000 mg/l.*

F/m = 0.4 day-1 Q = 20000 m^3/day BOD = 200 mg/l MLSS = 2000 mg/l
T = ? V = ? Air Supply ?

Detention Time" t"

F:M = = $\frac{(\text{BOD})}{(\text{t x MLSS})}$ = t = BOD/(MLSSxDT)

t = 200 / (0.4 x 2000) = 0.25 days

Volume

V = t x Q = 5000 m^3

Let No. units= 5

Volume of each unit = 5000/5 = 5 m^3

Dimension

Depth of aeration tank = 4 -6 m = 5 m

Area = 1000/5 = 200 m^2

L :W = 1:5 L = 5W

= 5WxW = 200

L = 32 m W = 6.5 m

Air Supply =

= 10m3 of air / m3 of sewage

Q = 20000 m^3/day

Air Supply = 20,000 x 10 = 20,0000 m^3/day

Air Supply per tank = 200000/5 = 40000 m^3/day

Return Sludge

Qr/Q = Vs / (1000-Vs)

Sludge volume Index (SVI) = (Vs x1000) / MLSS

MLSS = 2000 mg/l & Assume SVI = 100

Vs = 200 m^3

Qr = (Vs x Q) /(1000 – Vs) (200 x 4000) / (1000-200) =1000 m^3/d

Problem: ***C****alculate MLSS concentration required to result in an operating F/m ratio of 0.4 day^{-1} for the treatment of 15140 m^3/day of sewage with a BOD of 200 mg/l. Assume detention time in the aeration tank as 6 hours. Also calculate the volume of the aeration tank.*

Sol.

F/m = 0.4 day-1 Q = 15140 m^3/day BOD = 200 mg/l DT = 6 hours
MLSS = ? V = ?

F:M = $\frac{(\text{Q x BOD})}{(\text{V x MLSS})}$

DT = V/Q

V = DT x Q = $\frac{6}{24}$ x15140

= 3785 m^3

$$F:M = \frac{(Q \times BOD)}{(V \times MLSS)}$$

$$0.4 = \frac{(15140 \times 200)}{3785 \times MLSS}$$

$$MLSS = 2000 \text{ mg/l}$$

Problem: *An Activated Sludge Process system aeration tank has volume of 900 m^3 treating a sewage flow of 4000 m3/d with BOD of 250 mg/l. It is desired to achieve a SVI of 80 mg/l by adopting a re-circulation ratio (Qr /Q) of 0.25. Calculate F/M ratio at which it should be operated.*

Sol

F/m = ? Q = 4000 m^3/day BOD = 250 mg/l DT = ?

MLSS = ? V = ?

DT =V/Q = 900/4000 = 0.225 days

Qr/Q = Vs /(100-Vs)

Vs = 0.25(1000-Vs) = 250- 0.25 Vs = Vs

1.25Vs = 250

Vs = 200 m^3

SVI = (Vs x 1000) / MLSS

MLSS = = (200x1000)/80 = 2500 mg/l

= 3785 m^3

$$F:M = \frac{(BOD)}{(DT \times MLSS)}$$

$$= \frac{(250)}{0.225 \times 2500}$$

= 0.44 / day

Problem: *An Asp is to be designed to treat a sewage of 6 m^3 / min with BOD of 200 mg/l using F/M ration 0.4 per day and MLSS of 3000 mg/l. Calculate volume of aeration tank if SVI of 100 ml /l is maintained. How much sludge should be re-circulated?*

Sol.

F/m = 0.4 Q = 6 m^3/min = ______ m^3/day BOD = 200 mg/l

MLSS = 3000 mg/l SVI = 100 ml/l

Qr/Q = ? DT = ?

$$F:M = \frac{(BOD)}{(DT \times MLSS)}$$

DT =(200) / (DT x 3000) = 0.16 = 230.4 min

V = Q x DT = 6 x 230.4 = 1382.4 m^3

SVI = (Vs x 1000) / MLSS

Vs = 300 m^3

Qr/Q = Vs /(1000-Vs)

Qr = (Vs x Q) / ((1000-300) = 2.5 m^3 /min

2) Oxidation Ponds

These ponds may be considered to be completely mixed biological reactors without solids return. Mixing is provided by natural processes (wind, heat, fermentation). However mixing is augmented by mechanical or diffused aeration. These are also known as stabilization ponds or wastewater lagoons. These are ponds of controlled shape in which sewage/wastewater is treated by biological process.

2.1 Advantages

i) Suitable for small communities where land is cheep.

ii) Capital cost is low

iii) Minimum mechanical equipment are required.

iv) No imported parts / machinery is required

v) Easy to operate and maintain hence less Operation and Maintenance cost

vi) No skilled manpower is required for operation and maintenance of these ponds.

vii) Negligible odour problems

2.2 Disadvantages

i) Fecal coliform removal efficiency is poor.

ii) Needs final clarifiers

iii) Sludge handling problems are associated with these ponds.

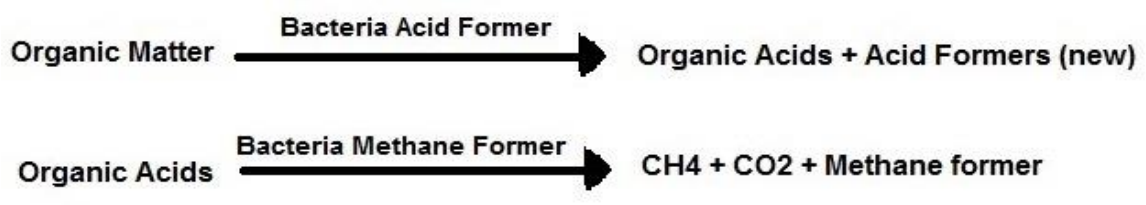
2.3 Classification of Oxidation Ponds

The oxidation ponds are classified on the basis of biological actions that occur in these ponds.

2.3.1 Anaerobic Ponds

These ponds are devoid of oxygen throughout their depth except top thin layer.
The depth of these ponds ranges from 2-6 meters. These ponds are designed to treat strong organic wastes or those containing high concentrations of Suspended Solid (SS). These ponds can remove high volumes of organic matter in relatively short detention time.

In anaerobic ponds, most of the suspended solids along with worms, eggs and pathogenic bacteria settle to the bottom where under anaerobic conditions organic matter is converted into CO_2, CH_4, H_2S, NH_3 and other end products.



The gases are dispersed into atmosphere through the liquid phase. Floating materials in the pond from scum which also absorbs the escaping gases. A liquid detention time 1-5 days is generally employed.

The effluent from anaerobic ponds require treatment in facultative ponds before final discharge. The sludge accumulation rate in these ponds is very slow and as such they require de-sludging every three to five years.

BOD removal efficiency of these ponds is better at higher temperature. At 20 0C BOD removal efficiencies are :

Detention Time (days)	01	2.5	5
Removal Efficiency (%)	50	60	70

These ponds are designed on the basis of detention of time and volumetric loading.

Parameter	**Value**
Detention Time	1-5 days
Volumetric loading	100-300 gm / m^3-day

For the design of ponds select a detention time and check for loading. Then adopting suitable depth, calculate the area.

i) Disadvantages

a) Formation of H_2S gas especially in the presence of sulphate ions (SO_4^{-2}) in concentration more than 100 mg/l causes odour.

b) Ponds should be located at least 1 Km away from residential areas to avoid nuisance.

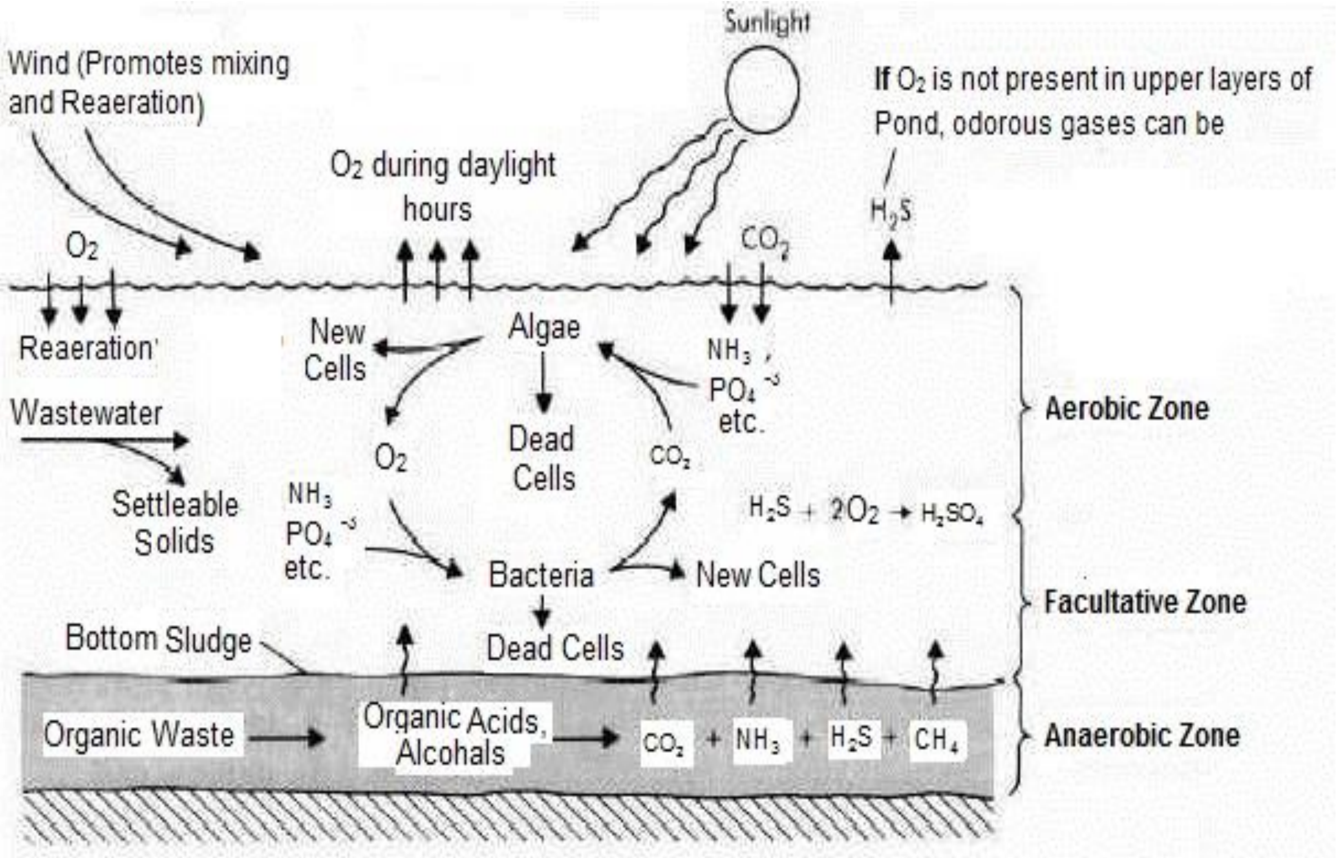
c) Effluent of these ponds needs further treatment in other types of ponds before final disposal.

Problem: *Design an anaerobic pond treat a flow of 2000 m^3/day of sewage with BOD of 600 mg/l.*

Let detention time	=	5 days		
Volume of Pond	=	5 x 2000	=	1000 m^3
BOD of sewage	=	600 mg/l	=	

2.3.2 Facultative Ponds

In these ponds oxygen persists throughout the liquid depth at least day time but oxygen is absent near the bottom of these ponds. The settle-able particles deposit on the bottom and organic matter in the sludge layer undergoes anaerobic breakdown. Soluble fermentation products enter the liquid layer above and are broken down aerobically along with soluble and colloidal organics in the incoming wastes. Prolific growth of algae takes place in the facultative ponds. Algae supply oxygen through photosynthesis for the bacterial breakdown of organic matter. Algae in turn utilize CO_2 released by bacteria in breaking down the organic components of the wastewater. The algae-bacterial symbiosis is shown in figure.



i) Design Criteria

Parameter	Value
Detention Time	20-40 days
Depth of water	1.5 – 2 meter

The effluent from facultative pond treating sewage is expected to have a BOD between 75 – 85 mg/l under local condition (Aziz, 1997). Facultative ponds require desludging every 10-15 years and thus pose minimum health risk to handler.

2.3.3 Maturation Ponds

These are fully aerobic ponds and are usually 1.5m deep. These are used as polishing stage after facultative ponds and their principle function is the destruction of pathogens. Further reduction in BOD are also achieved through use of maturation ponds. In practice maturation ponds are designed for removal of pathogens using faecal coliforms as indicator organisms. Generally a detention time of 7-14 days is employed in maturation ponds depending upon the nature of reuse of treated effluent form system.

With properly designed maturation ponds receiving treated domestic sewage from facultative ponds, it is possible to have an effluent with BOD around 30 mg/l, faecal coliform content id less than 1000 per 100 ml and a helminth count of less than 1 nematode egg per liter.

2.3.4 Aerobic Pond

In the aerobic pond oxygen is present throughout the pond and all biological activity is aerobic decomposition. Aerobic ponds are a maximum of two feet deep, so that the sunlight can reach throughout the entire depth of the pond, which will let the algae grow throughout. Algae give off Oxygen to permit aerobic process microorganisms to live.

i) Diurnal Variations in Aerobic Ponds

a) Diurnal Variation in Dissolved Oxygen level (wind current, vertical mixing of oxygen etc.)

b) Diurnal Variation in pH value

ii) Design Criteria

Parametrs	value
Depth (m)	0.15- 0.5
Retention time (day)	2 – 6
BOD_5 loading(lb/acre day)	100-200
BOD_5 removal(%)	80-90
Algae concentration(mg/l)	100-200
Effluent suspended solids concentration(mg/l)	150-350

Aerobic ponds are not used in colder climates because they will completely freeze in the winter.

ii. Attached Growth Process

This is also known as Surface Growth Process. This process utilize a solid media upon which bacterial solids are accumulated in order to maintain a high population. The area available for such growth is an important design parameter

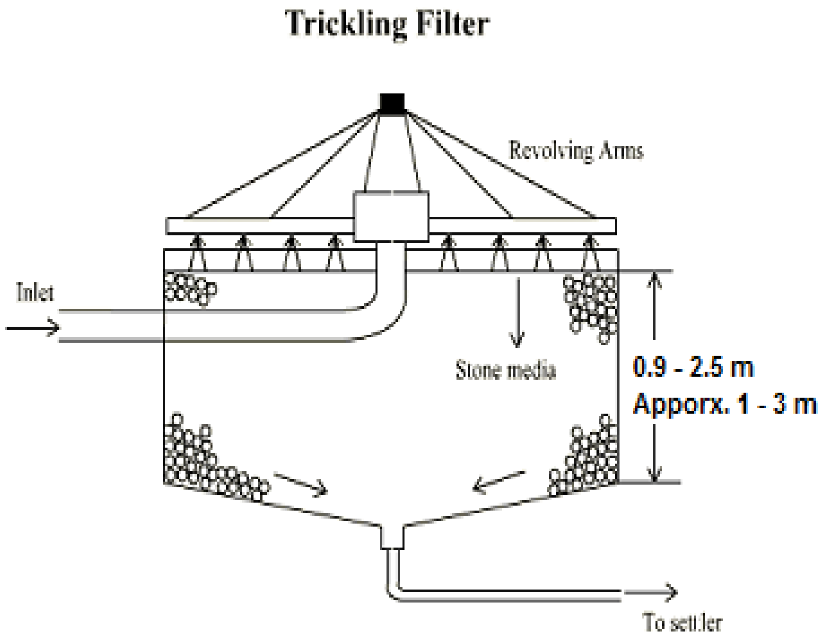
Surface Growth Process includes

1) Trickling Filters

2) Rotating Biological Contactors

1) Trickling filters

These consist of a bed of highly permeable media to which microorganisms are attached and through which wastewater is percolated. The filter media is usually consists of rock, stones varying in size from 25 – 100 mm in dia. Stone having size preferably from 60 – 90 mm are also employed. Depth of filter media varies with particular design but usually ranges from 1 – 3 m with 2 m being average value. Filters are usually circular in shape and sewage / wastewater is sprinkled over the top of the filter media/bed by a Rotary distributer, generally having four arms.



Trickling filters are provided with under-drain system for

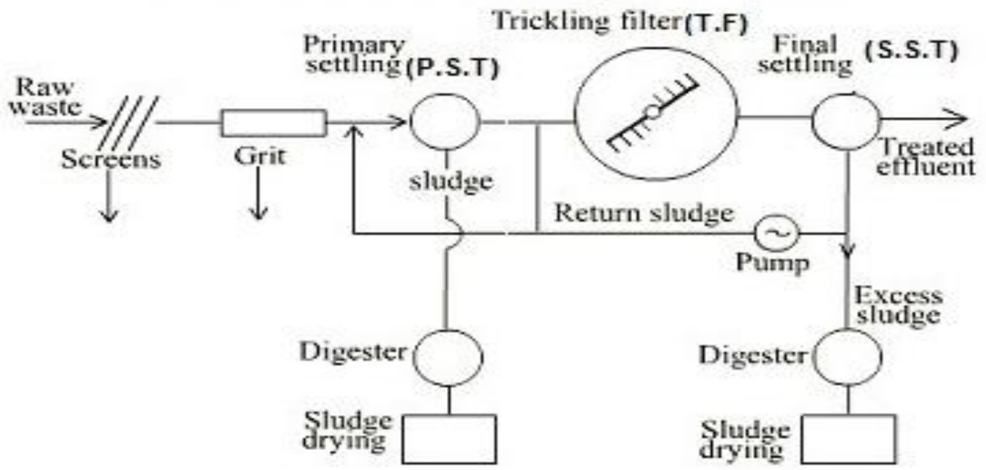
a) For collection of treated wastewater or any other detached biological matter
b) To allow air circulation through filter

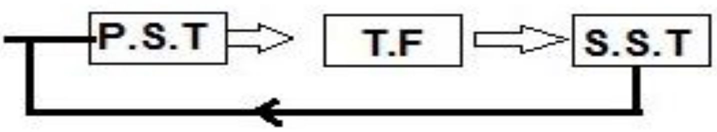
The organic matter present in the wastewater is degraded by the population of microorganisms attached to the filter media. These include Protozoa, Fungi, Algae and Bacteria.

1.1 Process Description

- Air circulation in the void space, by either natural draft or blowers, provides oxygen for the microorganisms growing as an attached biofilm.
- During operation, the organic material present in the wastewater is metabolised by the biomass attached to the medium. The biological slime grows in thickness as the organic matter abstracted from the flowing wastewater is synthesized into new cellular material.
- The thickness of the aerobic layer is limited by the depth of penetration of oxygen into the microbial layer.
- The micro-organisms near the medium face enter the ***endogenous phase*** (near dying) because the substrate is metabolised before it can reach the micro-organisms near the medium face due to increase in thickness of the slime layer and loose their ability to cling to the media surface. The liquid then washes the slime off the medium and a new slime layer starts to grow. This phenomenon of losing the slime layer is called ***sloughing***. Sloughing is primarily a function of organic and hydraulic loading.
- The sloughed off film and treated wastewater are collected by an under-drainage system which also allows circulation of air through filter. The collected liquid is passed to a settling tank used for solid- liquid separation.

Flow sheet of a trickling filter system





1.2 Classification of Trickling Filters

Depending upon the Organic and Hydraulic loading rates the filters can be classified as

1.2.1 Low Rate Trickling Fliters

1.2.2 High Rate Trickling Filters

Sr. No.	Parameter	Unit	Trickling Filter	
			Low rate	**High Rate**
1	Hydraulic Loading	m^3 /m^2-day	1 - 4	10 – 40
2	Organic Laoding Rate	Kg BOD/m^3-day	0.3 – 1.5	1.5 – 18.7
3	Depth of media	M	1.5 – 3.0	1.0 – 2.0
4	Recirculation Ratio (Qr/Q)	-	0 (No Recirculation)	1:3 – 2:1
5	Filter media	-	Rock, slag etc.	Rock, slag and synthetic, material
6	Power Required	KW/10^3 m^3	2 – 4	6 – 10
7	Sloughing		Intermetent	Coninuous
8	Dosing Interval		Not more than 5 mins.(generally intermittent)	Not more than 50 Sec.(Almost Continuius)
9	Odour / Fly		Comparatively more	Less
10	Configuration		Single Stage	Single or double stage

1.3 Role of Re-circulation

There are two types of re-circulations , while dealing with trickling filters.

1.3.1 Recirculation of the trickiling filter effluent

1.3.2 Recirculation of the Effluent form final clarifier

i) Recirculation of the trickling filter effluent to the trickling filter results in the return of viable organisms, thus improves the efficiency.

ii) Recirculation of the effluent from the clarifier to Trickling filter helps to increase its efficiency (details in book)

iii) Recirculation of both types reduces Odour and fly problems and helps in handing the shock loads.

1.4 Advantages of Trickling Filters

i) Satisfactory reduction of BOD i.e. upto 95 %

ii) Effective in handling shock loads

iii) Low operating cost

1.5 Disadvantages of Trickling Filters

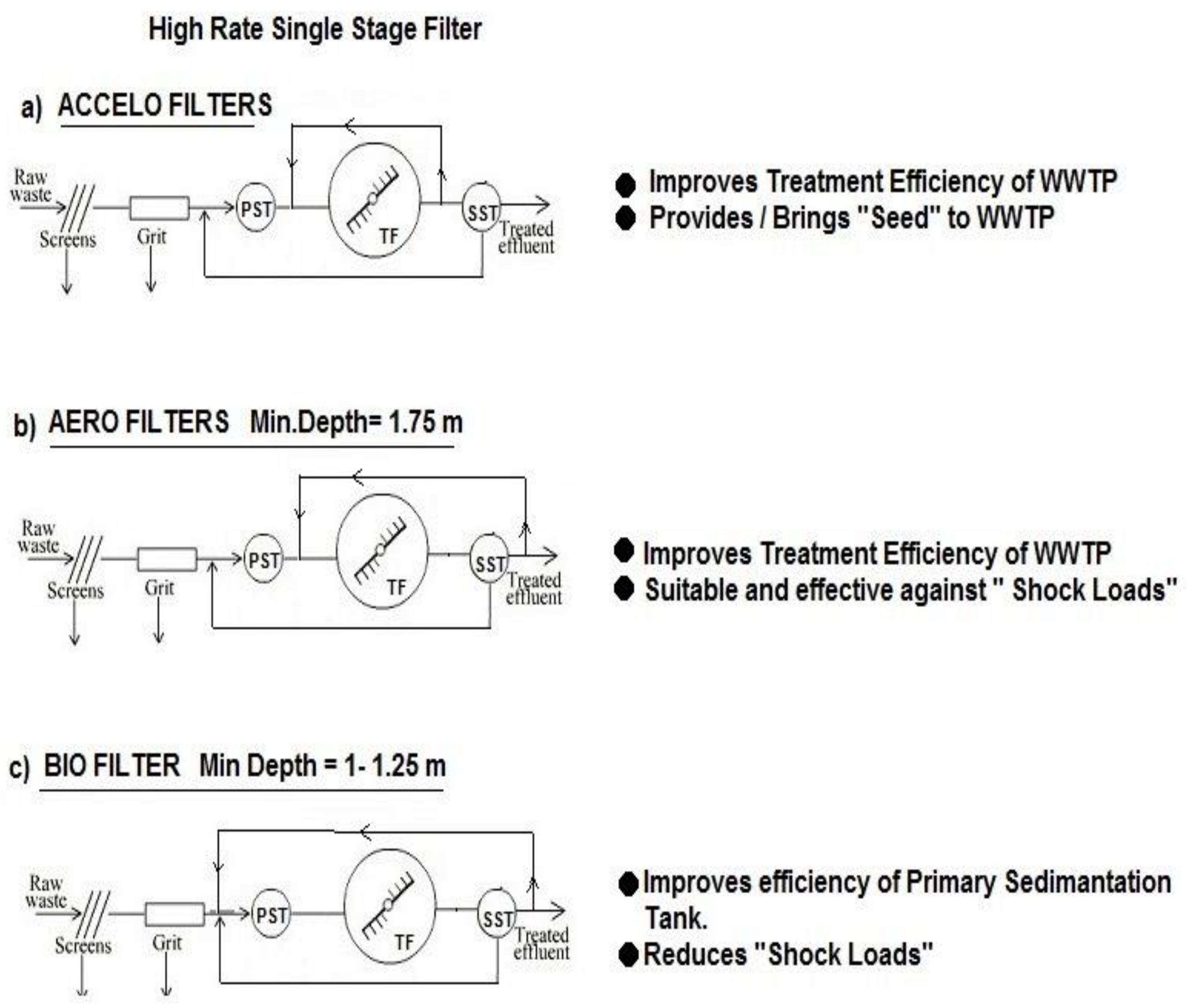
i) Head loss is significant

ii) Large areas are reqauired

iii) High construction cost

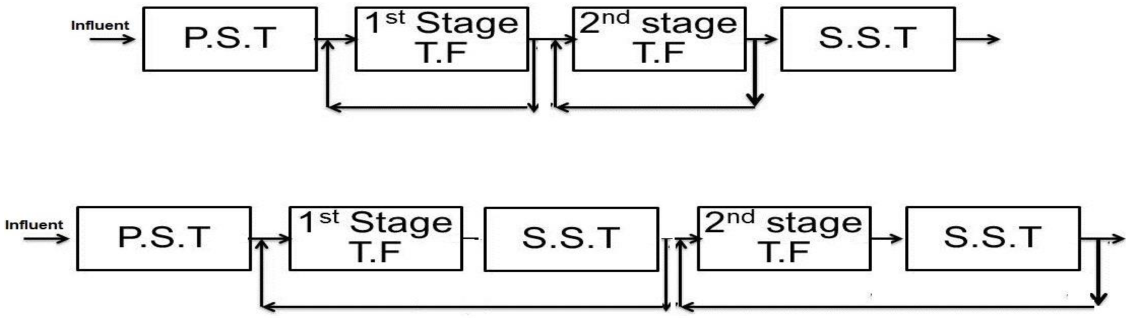
iv) Nuisance due to odour and fly (Psychoda)

1.6 High Rate Single Stage Trickling Filters



1.7 Double Stage High Rate Filters

Flow Sheet High Rate Two Stages Trickling Filter



For flow sheet diagrams of double stage high rate filters please consult the Book

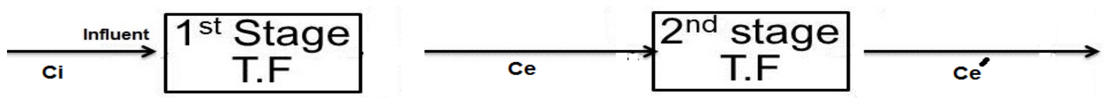
1.8 Performance of Trickling Filters

To estimate the performance of the trickling filters " National Research Council (NRC)" empirical formula is used. The formula is bsed uon the data collected at military base in USA during World War II. The emperical and semi empirical formulas used for design of filters are used with satisfactory results as long as the designers do not try to apply them in conditions different form these for which they were derived. NRC formula is used for design of both single stage and two stage trickling filters.

$$\frac{Ci - Ce}{Ci} = \frac{1}{1 + 0.532\sqrt{\frac{QCi}{VF}}}$$

This formula is for a single stage filter or for the 1st stage of the two stage system

<u>For the 1st stage of the System</u>



Where:

Ci = Influent BOD

Ce = Effluent BOD from Ist stage Trickling Filter

V = Volume of 1st Filter (m^3)

F = Recirculation Factor $= \frac{(1 + r)}{(1 + 0.1\, r)^2}$ **Where:** $r = \frac{Qr}{Q}$

For the 2nd stage of the System

$$\frac{Ce - Ce'}{Ce} = \frac{1}{1 + 0.532\sqrt{\dfrac{Q\,Ce}{\overline{V}\,\overline{F}}}}$$

Where:

Ce' **=** Effluent BOD of 2nd stage Trickling Filter

$\overline{V}$ = Volume of 2nd stage Trickling Filter

$\overline{F}$ = Re-circulation Factor for 2nd stage

1.9 Activated Sludge Process Vs. Trickling Filter Process

The basic difference between Activated Sludge Process (ASP) and the action involved in a Trickling Filters is that, in case of Trickling Filters, the bacterial film coating the contact material is Stationary and likely to become clogged after some time. Whereas, in case of Activated Sludge process, the fine suspended matter of sewage itself contain the bacterial film, which is kept moving because of constant agitation. The so called sludge flocks are active, free-loving organisms which are being continuously swept through the sewage and which in search of food and work oxidize the organic matter present in the sewage in a much more efficient way, which results in better efficiency of ASPs.

Advantages of ASP over Trickling Filters are summarized as below:

1.9.1 Advantages of ASP over Trickling Filters

i) Efficiency of Activated Sludge Plants is higher than that of Trickling Filter.

ii) Less land area is required

iii) The operating head is comparatively less

iv) Higher degree of treatment. Effluent produced is clear, sparkling and non-putrescible.BOD removal is 80-95% and Coliform removal is 90-95%

v) Greater flexibility of treatment permitting a control over the quality of effluent desired.

vi) Free from nuisance of odour and production of fly etc. as process is carried out under water.

1.9.2 Disadvantages of Activated Sludge Process in comparison of Trickling Filters

i) Relatively high cost of construction and operation

ii) Greater skilled attendance is required due to large mechanical works.

iii) Not suitable for shock loads (sudden change in strength and quality).

iv) In case of presence of detergents in influent, air diffusion plants may face foaming difficulties.

v) Difficulties in handling of large sludge.

Problem: *Calculate volume and depth of a low rate trickling filter to treat sewage of a 7650 m*^{3}*/day having BOD of 200 mg/l. Assume organic loading rate of 0.3 kg/m*^{3}*-day and hydraulic loading rate of 3 m/day.*

Sol.

Q		=	7650 m^3/d	BOD_5	=200 mg/l	Dia = ?
Assume &	OLR	=	0.3 kg/m^3-day			
HLR		=	3 m /d			
	HLR	=	Q/As	3	=	7650/As
	As	=	2550 m^2			
Also						
OLR		=	BOD Load / Volume			
Solution						
BOD load		=	(200 x 7650x1000)/1000000	Kg/day		
Volume		=	1530/0.3 = 5100 m^3			
As		=	$A = \pi D2/4$			
	D^2	=	(4x2550)/3.14	D	= 56.99	
	Depth	=	5100/2550	=	2 m	

Problem: *Design a high rate trickling filter to treat a flow of 10,000 m3 day having a BOD of 400 mg/l. Use Organic Loading Rate of 2.5 kg/m*^{3}*-day and Hydraulic Loading Rate of 25 m/d and employ a 1:1 recirculation ratio.*

Sol.

Q		=	1000 m^3/d	BOD_5	=400 mg/l	Dia = ?
Assume &	OLR	=	2.5 kg/m^3-day			
HLR		=	25 m /d			
	HLR	=	Q/As		=	
	As	=	m^2			
Also						
OLR		=	BOD Load / Volume			
Solution						
BOD load		=		Kg/day		
Volume		=				
As		=	$A = \pi d2/4$			
	d^2	=				
	Depth	=				

Problem: *Calculate Effluent BOD of Trickling Filter having volume 830 m^3 and discharge of 3.15 m^3 / min. The depth of filter is 2m and influent BOD is 290 mg/l. Take r-1.25.*

Example: Calculate the effluent BOD_5 of a 2 stage Trickling filter with the following flows, BOD_5 and dimensions

Q	=	3.15 m^3/min	BOD_5	=	290 mg/l
Vol. of Filter No.1	=	830 m^3			
Vol. of Filter No.2	=	830 m^3			
Filter Depth	=	2 m			
Qr for Filter No.1	=	1.25Q			
Qr for Filter No.2	=	Q			

Solution

Ci = 290 mg/l

We have

$$\frac{Ci - Ce}{Ci} = \frac{1}{1 + 0.532\sqrt{\dfrac{QCi}{VF}}}$$

And for 1st stage TF

$$F = \text{Recirculation Factor} = \frac{\left(1 + \dfrac{Qr}{Q}\right)}{\left(1 + 0.1\dfrac{Qr}{Q}\right)^2} \qquad \textbf{Where:} \quad r = \frac{Qr}{Q}$$

F = (1 + 1.25 Q) / [1 + 0.1 (1.25)]^2 = 1.778

$$\frac{\underline{\mathbf{290 - Ce)}}}{\mathbf{290}} = \frac{1}{[1 + 0.5323\{ (315x290)/830x1.778\}^{1/2}}$$

Ce = 85.57 mg/l

Now for 2nd stage

$$\frac{Ci - Ce'}{Ci} = \frac{1}{1 + 0.532\sqrt{\dfrac{QCi}{\bar{V}\bar{F}}}}$$

Qr/Q = 1

$\bar{\mathbf{F}}$ = (1 + 1)/(1 + 0.1)^2 = 1.653

Ce = 85.57 mg/l

$\bar{\mathbf{V}}$ = 830

$$\frac{85.57 - Ce'}{85.57} = \frac{1}{[1 + 0.5323\{(315 \times 85.57)/830 \times 1.654\}^{1/2}}$$

Ce' = 38.01 mg/l

34. Sludge Treatment and Disposal

In all biological waste treatment processes some surplus sludge is produced. This sludge is usually

a) huge in volume
b) Contain pathogens
c) Contain Nutrients
d) Sludge produces 55% to 75% methane (CH4) gas when subjected to biological degradation

34.1 Sludge Handling

The ***objective of sludge handling /residual management*** is:

- Removal of pathogens
- Reduction in sludge solids volume.
- Reduction of water content.
- Recovery of methane (CH_4)

3.4.1.1 Sludge Digestion

34.1.2 Sludge Drying

34.1.1 Sludge Digestion

i) Anaerobic Digestion

Sludge digestion involves the treatment of highly concentrated organic wastes in the absence of oxygen by anaerobic bacteria.The anaerobic treatment of organic wastes resulting in the production of carbon dioxide and methane, involves two distinct stages.

a) First stage (Acid Fermentation)

Ist stage is referred as "***acid fermentation***". In this stage, complex waste components including fats, proteins, and polysaccharides are first hydrolyzed by a heterogeneous group of facultative and anaerobic bacteria. These bacteria then subject the products of hydrolysis to fermentations, b-oxidations, and other metabolic processes leading to the formation of simple organic compounds, mainly short-chain (volatile) acids and alcohols. The bacteria involved in acid fermentation are are relatively tolerant to changes in pH and temperature and have a much higher rate of growth.

b) Second Stage (Methane Fermentation)

The second stage is referred as "***methane fermentation***". In this stage the end products of the first stage are converted to gases (mainly methane and carbon dioxide) by several different species of strictly anaerobic bacteria.

c) pH Adjustment

The bacteria responsible for acid fermentation are relatively tolerant to changes in pH and temperature and have a much higher rate of growth than the bacteria responsible for methane fermentation. They work much better in pH ranges 6.5 to 8.0, preferably pH from 7.2 to 7.4.If the pH drops below 6.0, methane formation essentially ceases, and more acid accumulates, thus bringing the digestion process to a standstill. Generally 2-5 kg lime for 1000 person is used daily for adjustment of pH.

The methane bacteria are highly active in mesophilic (27-43°C) with digestion period of four weeks and thermophilic range (35-40°C) with digestion period of 15-18 days. But thermophilic range is not practised because of odour and operational difficulties.

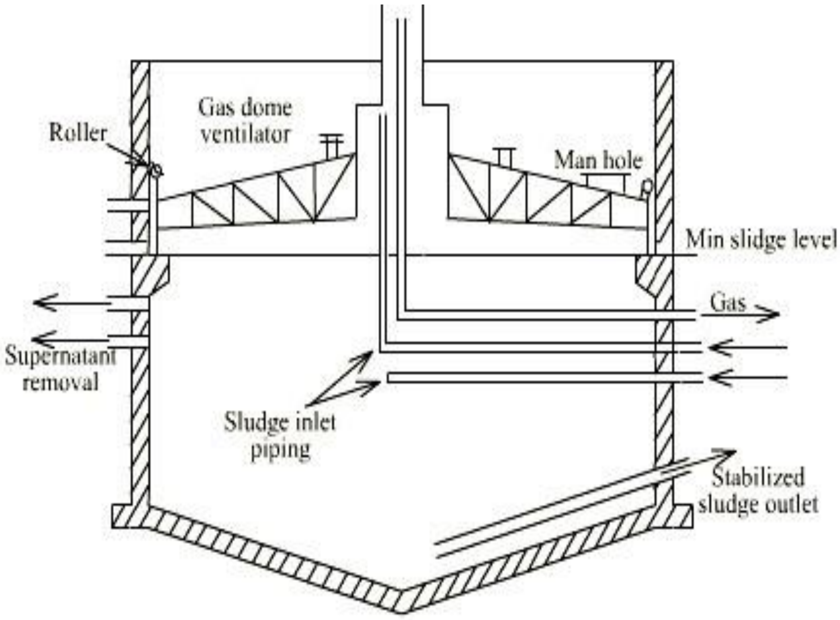
d) Water Content (W/C) of Sludges

Sr.No.	Type of Unit	**Water content**	
		Raw Sludge	Digested Sludge
1.	Primary Sedimentation Tank	94 – 96 %	88 – 94 %
2.	Activated Sludge Process	98.5 – 99.5 %	94 – 96%
3.	Trickling Filter	96 – 97 %	90 – 94 %

ii) Sludge Digesters

A sludge digestion tank is a RCC or steel tank of cylindrical shape with hopper bottom and is covered with fixed or floating type of roofs. It normally has following operational features

- Heating of the tank content at 35^0C
- Mixing of contents through propellers or draft tubes
- Sludge is loaded normally twice a day.
- Gas produced in the digester is drawn from top arrangements



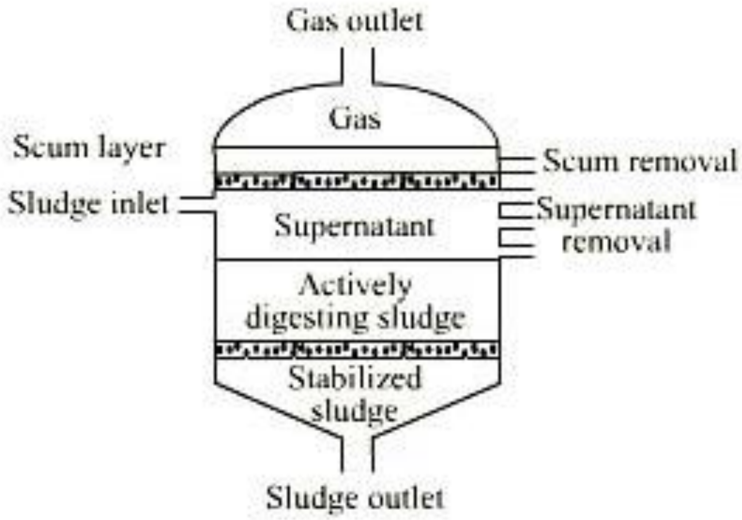
Anaerobic sludge digester

iii) Types of Anaerobic Digesters

The anaerobic digesters are of two types:

a) Standard Rate Sludge Digester

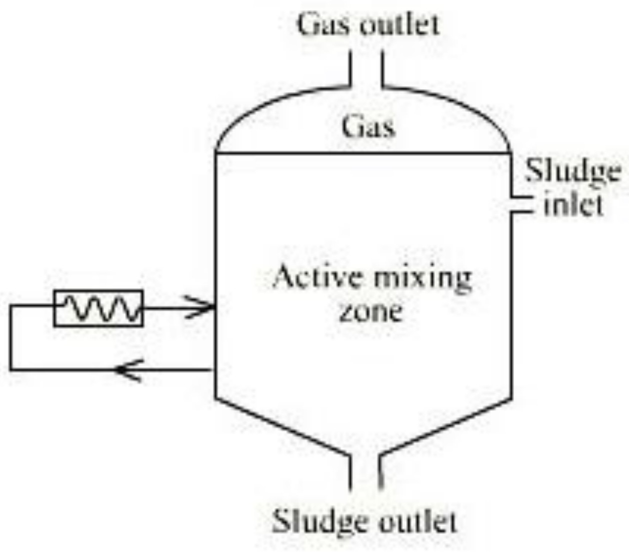
In the standard rate digestion process, the digester contents are usually unheated and unmixed (Quiescent). The digestion period may vary from 30 to 60 d. They act as sludge thickener.



Standard-rate sludge digester

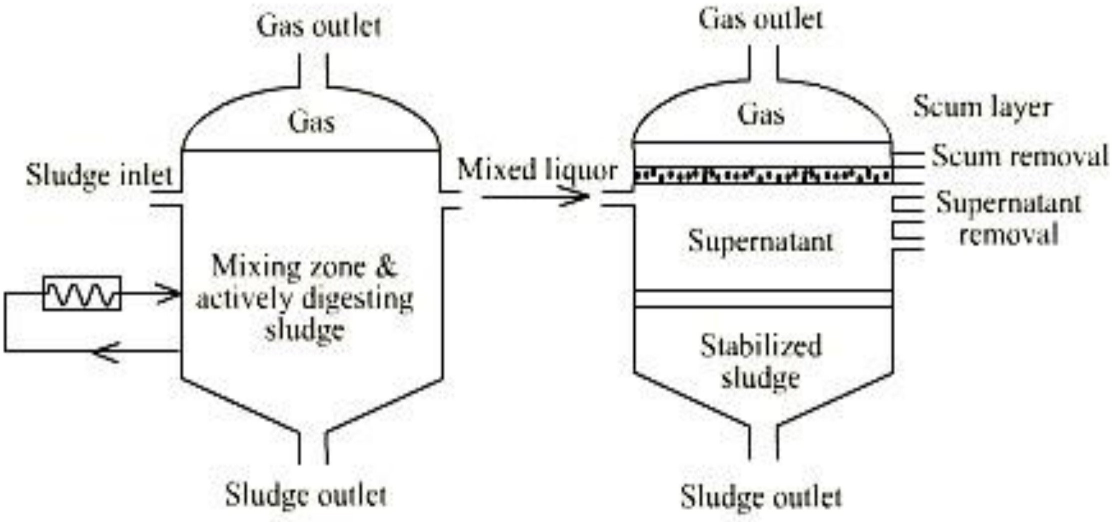
b) High Rate Sludge Digester

In a high rate digestion process, the digester contents are heated and completely mixed. The required detention period is 10 to 20 d.



High-rate sludge digester

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



Two stage sludge digester

iv) Design Criteria for Digesters

Sr.No.	Parameter	Unit	Value
1.	Solids Retention Time	Days	10-20
2.	Volume (1st Digester)	m^3/person	• 0.1 for Activated Sludge Process and Trickling Filter • 0.05 for Primary sedimentation tank
3.	VSS loading	$Kg/m^3.d$	2.4 – 6.4
4.	Digester Depth	m	6 – 15
5.	Bottom Slope	Nil	1 V : 3 H
6.	Digester Diameter	m	6 – 40