Raw Water Source

The various sources of water can be classified into two categories:

1. Surface sources, such as
    a. Ponds and lakes;
    b. Streams and rivers;
    c. Storage reservoirs; and
    d. Oceans, generally not used for water supplies, at present.
2. Sub-surface sources or underground sources, such as
    a. Springs;
    b. Infiltration wells ; and
    c. Wells and Tube-wells.

Water Quality

The raw or treated water is analysed by testing their physical, chemical and bacteriological characteristics:

Physical Characteristics:

Turbidity
Colour
Taste and Odour
Temperature

Chemical Characteristics:

pH
Acidity
Alkalinity
Hardness
Chlorides
Sulphates
Iron
Solids
Nitrates

Bacteriological Characteristics:

Bacterial examination of water is very important, since it indicates the degree of pollution. Water polluted by sewage contain one or more species of disease producing pathogenic bacteria. Pathogenic organisms cause water borne diseases, and many non pathogenic bacteria such as *E.Coli*, a member of coliform group, also live in the intestinal tract of human beings. *Coliform* itself is not a harmful group but it has more resistance to adverse condition than any other group. So, if it is ensured to minimize the number of coliforms, the harmful species will be very less. So, coliform group serves as indicator of contamination of water with sewage and presence of pathogens.

The methods to estimate the bacterial quality of water are:

Standard Plate Count Test
Most Probable Number
Membrane Filter Technique

Water Quantity Estimation

The quantity of water required for municipal uses for which the water supply scheme has to be designed requires following data:

1. Water consumption rate *(Per Capita Demand in litres per day per head)*
2. Population to be served.

Quantity= Per capita demand x Population

Water Consumption Rate

It is very difficult to precisely assess the quantity of water demanded by the public, since there are many variable factors affecting water consumption. The various types of water demands, which a city may have, may be broken into following classes:

Water Consumption for Various Purposes:

	Types of Consumption	Normal Range (lit/capita/day)	Average	%
1	Domestic Consumption	65-300	160	35
2	Industrial and Commercial Demand	45-450	135	30
3	Public Uses including Fire Demand	20-90	45	10
4	Losses and Waste	45-150	62	25

Fire Fighting Demand:

The per capita fire demand is very less on an average basis but the rate at which the water is required is very large. The rate of fire demand is sometimes traeted as a function of population and is worked out from following empirical formulae:

	Authority	Formulae (P in thousand)	Q for 1 lakh Population)
1	American Insurance Association	Q (L/min)=4637 $\sqrt{P}$ (1-0.01 $\sqrt{P}$)	41760
2	Kuchling's Formula	Q (L/min)=3182 $\sqrt{P}$	31800
3	Freeman's Formula	Q (L/min)= 1136.5(P/5+10)	35050
4	Ministry of Urban Development Manual Formula	Q (kilo liters/d)=100 $\sqrt{P}$ for P>50000	31623

Factors affecting per capita demand:

a. Size of the city: Per capita demand for big cities is generally large as compared to that for smaller towns as big cities have sewered houses.

b. Presence of industries.
c. Climatic conditions.
d. Habits of people and their economic status.
e. Quality of water: If water is aesthetically $ medically safe, the consumption will increase as people will not resort to private wells, etc.
f. Pressure in the distribution system.
g. Efficiency of water works administration: Leaks in water mains and services; and unauthorised use of water can be kept to a minimum by surveys.
h. Cost of water.
i. Policy of metering and charging method: Water tax is charged in two different ways: on the basis of meter reading and on the basis of certain fixed monthly rate.

Fluctuations in Rate of Demand

Average Daily Per Capita Demand
= Quantity Required in 12 Months/ (365 x Population)

If this average demand is supplied at all the times, it will not be sufficient to meet the fluctuations.

- ***Seasonal variation***: The demand peaks during summer. Firebreak outs are generally more in summer, increasing demand. So, there is seasonal variation .
- ***Daily variation*** depends on the activity. People draw out more water on Sundays and Festival days, thus increasing demand on these days.
- ***Hourly variations*** are very important as they have a wide range. During active household working hours i.e. from six to ten in the morning and four to eight in the evening, the bulk of the daily requirement is taken. During other hours the requirement is negligible. Moreover, if a fire breaks out, a huge quantity of water is required to be supplied during short duration, necessitating the need for a maximum rate of hourly supply.

So, an adequate quantity of water must be available to meet the peak demand. To meet all the fluctuations, the supply pipes, service reservoirs and distribution pipes must be properly proportioned. The water is supplied by pumping directly and the pumps and distribution system must be designed to meet the peak demand. The effect of monthly variation influences the design of storage reservoirs and the hourly variations influences the design of pumps and service reservoirs. As the population decreases, the fluctuation rate increases.

Maximum daily demand = 1.8 x average daily demand
Maximum hourly demand of maximum day i.e. Peak demand
= 1.5 x average hourly demand
= 1.5 x Maximum daily demand/24
= 1.5 x (1.8 x average daily demand)/24
= 2.7 x average daily demand/24
= 2.7 x annual average hourly demand

Design Periods & Population Forecast

This quantity should be worked out with due provision for the estimated requirements of the future . The future period for which a provision is made in the water supply scheme is known as the **design period**.

Design period is estimated based on the following:

- Useful life of the component, considering obsolescence, wear, tear, etc.
- Expandability aspect.
- Anticipated rate of growth of population, including industrial, commercial developments & migration-immigration.
- Available resources.
- Performance of the system during initial period.

Population Forecasting Methods

The various methods adopted for estimating future populations are given below. The particular method to be adopted for a particular case or for a particular city depends largely on the factors discussed in the methods, and the selection is left to the discretion and intelligence of the designer.

1. *Arithmetic Increase Method*
2. *Geometric Increase Method*
3. *Incremental Increase Method*
4. *Decreasing Rate of Growth Method*
5. *Simple Graphical Method*
6. *Comparative Graphical Method*
7. *Ratio Method*
8. *Logistic Curve Method*

Intake Structure

The basic function of the intake structure is to help in safely withdrawing water from the source over predetermined pool levels and then to discharge this water into the withdrawal conduit (normally called intake conduit), through which it flows up to water treatment plant.

Factors Governing Location of Intake

1. As far as possible, the site should be near the treatment plant so that the cost of conveying water to the city is less.
2. The intake must be located in the purer zone of the source to draw best quality water from the source, thereby reducing load on the treatment plant.
3. The intake must never be located at the downstream or in the vicinity of the point of disposal of wastewater.
4. The site should be such as to permit greater withdrawal of water, if required at a future date.
5. The intake must be located at a place from where it can draw water even during the driest period of the year.
6. The intake site should remain easily accessible during floods and should noy get flooded. Moreover, the flood waters should not be concentrated in the vicinity of the intake.

Design Considerations

1. sufficient factor of safety against external forces such as heavy currents, floating materials, submerged bodies, ice pressure, etc.
2. should have sufficient self weight so that it does not float by upthrust of water.

Types of Intake

Depending on the source of water, the intake works are classified as follows:

Pumping

A pump is a device which converts mechanical energy into hydraulic energy. It lifts water from a lower to a higher level and delivers it at high pressure. Pumps are employed in water supply projects at various stages for following purposes:

1. To lift raw water from wells.
2. To deliver treated water to the consumer at desired pressure.
3. To supply pressured water for fire hydrants.
4. To boost up pressure in water mains.
5. To fill elevated overhead water tanks.
6. To back-wash filters.
7. To pump chemical solutions, needed for water treatment.

Classification of Pumps

Based on principle of operation, pumps may be classified as follows:

1. Displacement pumps (reciprocating, rotary)
2. Velocity pumps (centrifugal, turbine and jet pumps)
3. Buoyancy pumps (air lift pumps)
4. Impulse pumps (hydraulic rams)

Capacity of Pumps

Work done by the pump,

$H.P. = \gamma QH/75$

where, γ= specific weight of water kg/m^3, Q= discharge of pump, m^3/s; and H= total head against which pump has to work.

$H = H_s + H_d + H_f$ + (losses due to exit, entrance, bends, valves, and so on)

where, H_s=suction head, H_d = delivery head, and H_f = friction loss.

Efficiency of pump (E) = γQH/Brake H.P.

Total brake horse power required = γQH/E

Provide even number of motors say 2,4,... with their total capacity being equal to the total BHP and provide half of the motors required as stand-by.

Conveyance

There are two stages in the transportation of water:

1. Conveyance of water from the source to the treatment plant.
2. Conveyance of treated water from treatment plant to the distribution system.

In the first stage water is transported by gravity or by pumping or by the combined action of both, depending upon the relative elevations of the treatment plant and the source of supply.
In the second stage water transmission may be either by pumping into an overhead tank and then supplying by gravity or by pumping directly into the water-main for distribution.

Free Flow System

In this system, the surface of water in the conveying section flows freely due to gravity. In such a conduit the hydraulic gradient line coincide with the water surface and is parallel to the bed of the conduit. It is often necessary to construct very long conveying sections, to suit the slope of the existing ground. The sections used for free-flow are: Canals, flumes, grade aqueducts and grade tunnels.

Pressure System

In pressure conduits, which are closed conduits, the water flows under pressure above the atmospheric pressure. The bed or invert of the conduit in pressure flows is thus independant of the grade of the hydraulic gradient line and can, therefore, follow the natural available ground surface thus requiring lesser length of conduit. The pressure aqueducts may be in the form of closed pipes or closed aqueducts and tunnels called *pressure aqueducts or pressure tunnels* designed for the pressure likely to come on them. Due to their circular shapes, every pressure conduit is generally termed as a *pressure pipe*. When a pressure pipe drops beneath a valley, stream, or some other depression, it is called a depressed pipe or an *inverted siphon*.
Depending upon the construction material, the pressure pipes are of following types: Cast iron, steel, R.C.C, hume steel, vitrified clay, asbestos cement, wrought iron, copper, brass and lead, plastic, and glass reinforced plastic pipes.

Hydraulic Design

The design of water supply conduits depends on the resistance to flow, available pressure or head, and allowable velocities of flow. Generally, Hazen-William's formula for pressure conduits and Manning's formula for freeflow conduits are used.

Hazen-William's formula

$$U = 0.85\, C\, r_H^{0.63} S^{0.54}$$

Manning's formula

$$U = \frac{1}{n}\, r_H^{2/3} S^{1/2}$$

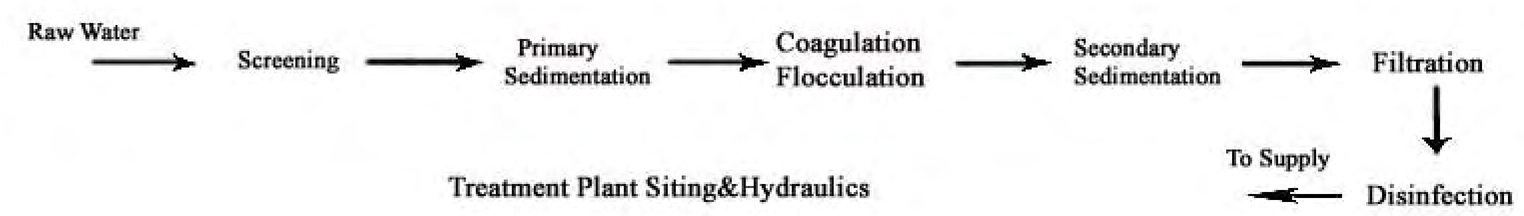
where, U= velocity, m/s; r_H= hydraulic radius,m; S= slope, C= Hazen-William's coefficient, and n = Manning's coefficient.

Darcy-Weisbach formula

$$h_L = (fLU^2)/(2gd)$$

The available raw waters must be treated and purified before they can be supplied to the public for their domestic, industrial or any other uses. The extent of treatment required to be given to the particular water depends upon the characteristics and quality of the available water, and also upon the quality requirements for the intended use..

The layout of conventional water treatment plant is as follows:



Depending upon the magnitude of treatment required, proper unit operations are selected and arranged in the proper sequential order for the purpose of modifying the quality of raw water to meet the desired standards. Indian Standards for drinking water are given in the table below.

Indian Standards for drinking water

Parameter	**Desirable-Tolerable**	***If no alternative source available, limit extended upto***
Physical		
Turbidity (NTU unit)	< 10	25
Colour (Hazen scale)	< 10	50
Taste and Odour	Un-objectionable	Un-objectionable
Chemical		
pH	7.0-8.5	6.5-9.2
Total Dissolved Solids mg/l	500-1500	3000
Total Hardness mg/l (as $CaCO_3$)	200-300	600
Chlorides mg/l (as Cl)	200-250	1000
Sulphates mg/l (as SO_4)	150-200	400
Fluorides mg/l (as F)	0.6-1.2	1.5
Nitrates mg/l (as NO_3)	45	45
Calcium mg/l (as Ca)	75	200
Iron mg/l (as Fe)	0.1-0.3	1.0

The typical functions of each unit operations are given in the following table:

Functions of Water Treatment Units

Unit treatment	Function (removal)
Aeration, chemicals use	Colour, Odour, Taste
Screening	Floating matter
Chemical methods	Iron, Manganese, etc.
Softening	Hardness
Sedimentation	Suspended matter
Coagulation	Suspended matter, a part of colloidal matter and bacteria
Filtration	Remaining colloidal dissolved matter, bacteria
Disinfection	Pathogenic bacteria, Organic matter and Reducing substances

The types of treatment required for different sources are given in the following table:

Source	Treatment required
1. Ground water and spring water fairly free from contamination	No treatment or Chlorination
2. Ground water with chemicals, minerals and gases	Aeration, coagulation (if necessary), filtration and disinfection
3. Lakes, surface water reservoirs with less amount of pollution	Disinfection
4. Other surface waters such as rivers, canals and impounded reservoirs with a considerable amount of pollution	Complete treatment

Aeration

- Aeration removes odour and tastes due to volatile gases like hydrogen sulphide and due to algae and related organisms.
- Aeration also oxidise iron and manganese, increases dissolved oxygen content in water, removes CO_2 and reduces corrosion and removes methane and other flammable gases.
- Principle of treatment underlines on the fact that volatile gases in water escape into atmosphere from the air-water interface and atmospheric oxygen takes their place in water, provided the water body can expose itself over a vast surface to the atmosphere. This process continues until an equilibrium is reached depending on the partial pressure of each specific gas in the atmosphere.

Types of Aerators

1. Gravity aerators
2. Fountain aerators
3. Diffused aerators
4. Mechanical aerators.

Gravity Aerators (Cascades): In gravity aerators, water is allowed to fall by gravity such that a large area of water is exposed to atmosphere, sometimes aided by turbulence.

Fountain Aerators : These are also known as spray aerators with special nozzles to produce a fine spray. Each nozzle is 2.5 to 4 cm diameter discharging about 18 to 36 l/h. Nozzle spacing should be such that each m^3 of water has aerator area of 0.03 to 0.09 m^2 for one hour.

Injection or Diffused Aerators : 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. Time of aeration is 10 to 30 min and 0.2 to 0.4 litres of air is required for 1 litre of water.

Mechanical Aerators : Mixing paddles as in flocculation are used. Paddles may be either submerged or at the surface.

Settling

Solid liquid separation process in which a suspension is separated into two phases –

- Clarified supernatant leaving the top of the sedimentation tank (overflow).
- Concentrated sludge leaving the bottom of the sedimentation tank (underflow).

Purpose of Settling

- To remove coarse dispersed phase.
- To remove coagulated and flocculated impurities.
- To remove precipitated impurities after chemical treatment.
- To settle the sludge (biomass) after activated sludge process / tricking filters.

Principle of Settling

- Suspended solids present in water having specific gravity greater than that of water tend to settle down by gravity as soon as the turbulence is retarded by offering storage.
- Basin in which the flow is retarded is called ***settling tank***.
- Theoretical average time for which the water is detained in the settling tank is called the ***detention period***.

Types of Settling

Type I: ***Discrete particle settling*** - Particles settle individually without interaction with neighboring particles.
Type II: ***Flocculent Particles*** – Flocculation causes the particles to increase in mass and settle at a faster rate.
Type III: ***Hindered or Zone settling*** –The mass of particles tends to settle as a unit with individual particles remaining in fixed positions with respect to each other.
Type IV: ***Compression*** – The concentration of particles is so high that sedimentation can only occur through compaction of the structure.

Type I Settling

- Size, shape and specific gravity of the particles do not change with time.
- Settling velocity remains constant.

If a particle is suspended in water, it initially has two forces acting upon it:
(1) ***force of gravity***: $F_g=\rho_p g V_p$
(2) the ***buoyant force*** quantified by Archimedes as: $F_b=\rho g V_p$
If the density of the particle differs from that of the water, a net force is exerted and the particle is accelaratd in the direction of the force:
$F_{net}=(\rho_p-\rho)gV_p$
This net force becomes the driving force.
Once the motion has been initiated, a third force is created due to viscous friction. This force, called the ***drag force***, is quantified by:
$F_d=C_D A_p \rho v^2/2$
C_D= drag coefficient.
A_p = projected area of the particle.
Because the drag force acts in the opposite direction to the driving force and increases as the square of the velocity, accelaration occurs at a decreasing rate until a steady velocity is reached at a point where the drag force equals the driving force:
$(\rho_p-\rho)gV_p = C_D A_p \rho v^2/2$
For spherical particles,
$V_p=\pi d^3/6$ and $A_p=\pi d^2/4$
Thus, $v^2 = \frac{4g(\rho_p-\rho)d}{3 \; C_D \rho}$
Expressions for C_D change with characteristics of different flow regimes. For laminar, transition, and turbulent flow, the values of C_D are:
$C_D = \frac{24}{R_e}$ (laminar)
$C_D = \frac{24}{R_e} + \frac{3}{R_e^{1/2}} + 0.34$ (transition)
$C_D = 0.4$ (turbulent)
where Re is the Reynolds number:
$R_e = \frac{\rho v d}{\mu}$

Reynolds number less than 1.0 indicate laminar flow, while values greater than 10 indicate turbulent flow. Intermediate values indicate transitional flow.

Stokes Flow

For laminar flow, terminal settling velocity equation becomes:

$$v = \frac{(\rho_p - \rho) g d^2}{18\mu}$$

which is known as the ***stokes equation***.

Transition Flow

Need to solve non-linear equations:

$$v^2 = \frac{4g(\rho_p - \rho)d}{3 \quad C_D \rho}$$

$$C_D = \frac{24}{R_e} + \frac{3}{R_e^{1/2}} + 0.34$$

$$R_e = \frac{\rho v d}{\mu}$$

- Calculate velocity using Stokes law or turbulent expression.
- Calculate and check Reynolds number.
- Calculate C_D.
- Use general formula.
- Repeat from step 2 until convergence.

Types of Settling Tanks

- Sedimentation tanks may function either intermittently or continuously.The intermittent tanks also called quiescent type tanks are those which store water for a certain period and keep it in complete rest. In a continuous flow type tank, the flow velocity is only reduced and the water is not brought to complete rest as is done in an intermittent type.
- Settling basins may be either long rectangular or circular in plan. Long narrow rectangular tanks with horizontal flow are generally preferred to the circular tanks with radial or spiral flow.

Long Rectangular Settling Basin

- Long rectangular basins are hydraulically more stable, and flow control for large volumes is easier with this configuration.
- A typical long rectangular tank have length ranging from 2 to 4 times their width. The bottom is slightly sloped to facilitate sludge scraping. A slow moving mechanical sludge scraper continuously pulls the settled material into a sludge hopper from where it is pumped out periodically.

A long rectangular settling tank can be divided into four different functional zones:
Inlet zone: Region in which the flow is uniformly distributed over the cross section such that the flow through settling zone follows horizontal path.
Settling zone: Settling occurs under quiescent conditions.
Outlet zone: Clarified effluent is collected and discharge through outlet weir.
Sludge zone: For collection of sludge below settling zone.

Inlet and Outlet Arrangement

Inlet devices: Inlets shall be designed to distribute the water equally and at uniform velocities. A baffle should be constructed across the basin close to the inlet and should project several feet below the water surface to dissipate inlet velocities and provide uniform flow;

Outlet Devices: Outlet weirs or submerged orifices shall be designed to maintain velocities suitable for settling in the basin and to minimize short-circuiting. Weirs shall be adjustable, and at least equivalent in length to the perimeter of the tank. However, peripheral weirs are not acceptable as they tend to cause excessive short-circuiting.

Weir Overflow Rates

Large weir overflow rates result in excessive velocities at the outlet. These velocities extend backward into the settling zone, causing particles and flocs to be drawn into the outlet. Weir loadings are generally used upto 300 m^3/d/m. It may be necessary to provide special inboard weir designs as shown to lower the weir overflow rates.

Inboard Weir Arrangement to Increase Weir Length

Circular Basins

- Circular settling basins have the same functional zones as the long rectangular basin, but the flow regime is different. When the flow enters at the center and is baffled to flow radially towards the perimeter, the horizontal velocity of the water is continuously decreasing as the distance from the center increases. Thus, the particle path in a circular basin is a parabola as opposed to the straight line path in the long rectangular tank.
- Sludge removal mechanisms in circular tanks are simpler and require less maintenance.

Settling Operations

- Particles falling through the settling basin have two components of velocity:

  1) Vertical component: $v_t = \dfrac{(\rho_p - \rho) g d^2}{18\mu}$

  2) Horizontal component: $v_h = Q/A$

  The path of the particle is given by the vector sum of horizontal velocity v_h and vertical settling velocity v_t.
- Assume that a settling column is suspended in the flow of the settling zone and that the column travels with the flow across the settling zone. Consider the particle in the batch analysis for type-1 settling which was initially at the surface and settled through the depth of the column Z_0, in the time t_0. If t_0 also corresponds to the time required for the column to be carried horizontally across the settling zone, then the particle will fall into the sludge zone and be removed from the suspension at the point at which the column reaches the end of the settling zone.
  All particles with $v_t > v_0$ will be removed from suspension at some point along the settling zone.
- Now consider the particle with settling velocity < v_0. If the initial depth of this particle was such that $Z_p/v_t = t_0$, this particle will also be removed. Therefore, the removal of suspended particles passing through the settling zone will be in proportion to the ratio of the individual settling velocities to the settling velocity v_0.
  The time t_0 corresponds to the retention time in the settling zone. $t = \dfrac{V}{Q} = \dfrac{LZ_0W}{Q}$

Also, $t_0 = \dfrac{Z_0}{v_0}$

Therefore, $\frac{Z_0}{v_0} = \frac{LZ_0W}{Q}$ and $v_0 = \frac{Q}{LW}$

or $v_0 = \frac{Q}{A_S}$

Thus, the depth of the basin is not a factor in determining the size particle that can be removed completely in the settling zone. The determining factor is the quantity Q/A_s, which has the units of velocity and is referred to as the overflow rate q_0. This overflow rate is the design factor for settling basins and corresponds to the terminal setting velocity of the particle that is 100% removed.

Design Details

1. Detention period: for plain sedimentation: 3 to 4 h, and for coagulated sedimentation: 2 to 2.5 h.
2. Velocity of flow: Not greater than 30 cm/min (horizontal flow).
3. Tank dimensions: L:B = 3 to 5:1. Generally L= 30 m (common) maximum 100 m. Breadth= 6 m to 10 m. Circular: Diameter not greater than 60 m. generally 20 to 40 m.
4. Depth 2.5 to 5.0 m (3 m).
5. Surface Overflow Rate: For plain sedimentation 12000 to 18000 L/d/m2 tank area; for thoroughly flocculated water 24000 to 30000 L/d/m2 tank area.
6. Slopes: Rectangular 1% towards inlet and circular 8%.

General Properties of Colloids

1. Colloidal particles are so small that their ***surface area*** in relation to mass is very large.
2. ***Electrical properties***: All colloidal particles are electrically charged. If electrodes from a D.C. source are placed in a colloidal dispersion, the particles migrate towards the pole of opposite charge.
3. Colloidal particles are in constant motion because of bombardment by molecules of dispersion medium. This motion is called ***Brownian motion*** (named after Robert Brown who first noticed it).
4. ***Tyndall effect:*** Colloidal particles have dimensioThese are reversible upon heating. e.g. organics in water.
5. ***Adsorption:*** Colloids have high surface area and hence have a lot of active surface for adsorption to occur. The stability of colloids is mainly due to preferential adsorption of ions. There are two types of colloids:
   i. Lyophobic colloids: that are solvent hating. These are irreversible upon heating. e.g. inorganic colloids, metal halides.
   ii. Lyophilic colloids: that are solvent loving. These are reversible upon heating. e.g. organics in water.

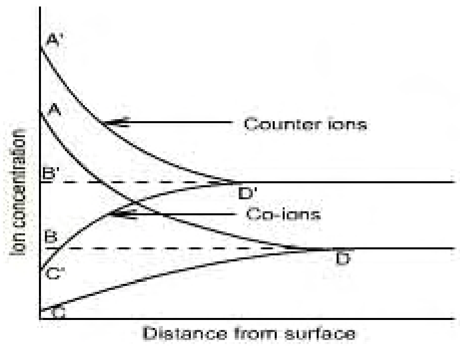
Coagulation and Flocculation

- Colloidal particles are difficult to separate from water because they do not settle by gravity and are so small that they pass through the pores of filtration media.
- To be removed, the individual colloids must aggregate and grow in size.
- The aggregation of colloidal particles can be considered as involving two separate and distinct steps:
  1. Particle transport to effect interparticle collision.
  2. Particle destabilization to permit attachment when contact occurs.

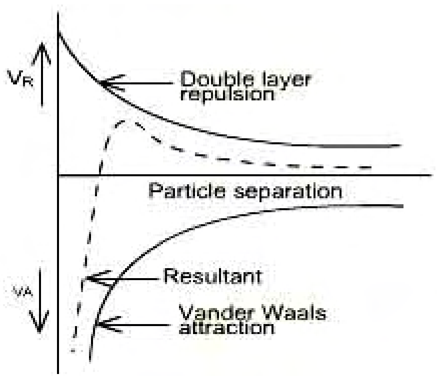
Transport step is known as ***flocculation*** whereas ***coagulation*** is the overall process involving destabilization and transport.

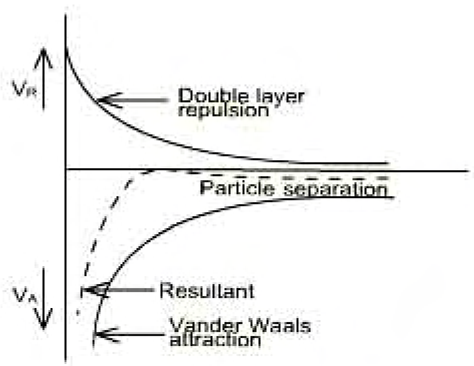
Electrical Double Layer

Although individual hydrophobic colloids have an electrical charge, a colloidal dispersion does not have a net electrical charge. The diffuse layer in a colloidal dispersion contains a quantity of counter ions sufficient to balance the electrical charge on the particle. The charge distribution in the diffuse layer of a negatively charged colloid can be represented by the curve ABCD in the figure. The ions involved in this electroneutrality are arranged in such a way as to constitute what is called ***electrical double layer***.



Net repulsion force, which may be considered as energy barrier must be overcome before aggregation occurs. The magnitude of energy barrier depends on (1) charge on the particle, and (2) ionic composition of water.





Destabilization of Colloidal Dispersion

Particle destabilization can be achieved by four mechanisms:

- Change characteristics of medium-*Compression of double layer.*
- Change characteristics of colloid particles-*Adsorption and charge neutralization*.
- Provide bridges-
  1. *Enmeshment in a precipitate.*
  2. *Adsorption and interparticle bridging*.

Flocculation is stimulation by mechanical means to agglomerate destabilised particles into compact, fast settleable particles (or flocs). Flocculation or gentle agitation results from velocity differences or gradients in the coagulated water, which causes the fine moving, destabilized particles to come into contact and become large, readily settleable flocs. It is a common practice to provide an initial rapid (or) flash mix for the dispersal of the coagulant or other chemicals into the water. Slow mixing is then done, during which the growth of the floc takes place.

Rapid or Flash mixing is the process by which a coagulant is rapidly and uniformly dispersed through the mass of water. This process usually occurs in a small basin immediately preceding or at the head of the coagulation basin. Generally, the detention period is 30 to 60 seconds and the head loss is 20 to 60 cms of water. Here colloids are destabilised and the nucleus for the floc is formed.

Slow mixing brings the contacts between the finely divided destabilised matter formed during rapid mixing.

Perikinetic and Orthokinetic Flocculation

The flocculation process can be broadly classified into two types, perikinetic and orthokinetic.

Perikinetic flocculation refers to flocculation (contact or collisions of colloidal particles) due to Brownian motion of colloidal particles. The random motion of colloidal particles results from their rapid and random bombardment by the molecules of the fluid.

Orthokinetic flocculation refers to contacts or collisions of colloidal particles resulting from bulk fluid motion, such as stirring. In systems of stirring, the velocity of the fluid varies both spatially (from point to point) and temporally (from time to time). The spatial changes in velocity are identified by a velocity gradient, G. G is estimated as $G=(P/\eta V)^{1/2}$, where P=Power, V=channel volume, and η= Absolute viscosity.

Mechanism of Flocculation

Gravitational flocculation: Baffle type mixing basins are examples of gravitational flocculation. Water flows by gravity and baffles are provided in the basins which induce the required velocity gradients for achieving floc formation.

Mechanical flocculation: Mechanical flocculators consists of revolving paddles with horizontal or vertical shafts or paddles suspended from horizontal oscillating beams, moving up and down.

Coagulation in Water Treatment

- Salts of Al(III) and Fe(III) are commonly used as coagulants in water and wastewater treatment.
- When a salt of Al(III) and Fe(III) is added to water, it dissociates to yield trivalent ions, which hydrate to form aquometal complexes $Al(H_2O)_6^{3+}$ and $Fe(H_2O)_6^{3+}$. These complexes then pass through a series of hydrolytic reactions in which H_2O molecules in the hydration shell are replaced by OH^- ions to form a variety of soluble species such as $Al(OH)^{2+}$ and $Al(OH)^{2+}$. These products are quite effective as coagulants as they adsorb very strongly onto the surface of most negative colloids.

Destabilization using Al(III) and Fe(III) Salts

- Al(III) and Fe(III) accomplish destabilization by two mechanisms:
  (1) Adsorption and charge neutralization.
  (2) Enmeshment in a sweep floc.
- Interrelations between pH, coagulant dosage, and colloid concentration determine mechanism responsible for coagulation.
- Charge on hydrolysis products and precipitation of metal hydroxides are both controlled by pH. The hydrolysis products possess a positive charge at pH values below iso-electric point of the metal hydroxide. Negatively charged species which predominate above iso-electric point, are ineffective for the destabilization of negatively charged colloids.
- Precipitation of amorphous metal hydroxide is necessary for sweep-floc coagulation.
- The solubility of $Al(OH)_3(s)$ and $Fe(OH)_3(s)$ is minimal at a particular pH and increases as the pH increases or decreases from that value. Thus, pH must be controlled to establish optimum conditions for coagulation.
- Alum and Ferric Chloride reacts with natural alkalinity in water as follows:

$$Al_2(SO_4)_3.14H_2O + 6\ HCO^{3-} \longrightarrow 2\ Al(OH)_3(s) + 6CO_2 + 14\ H_2O + 3\ SO_4^{2-}$$

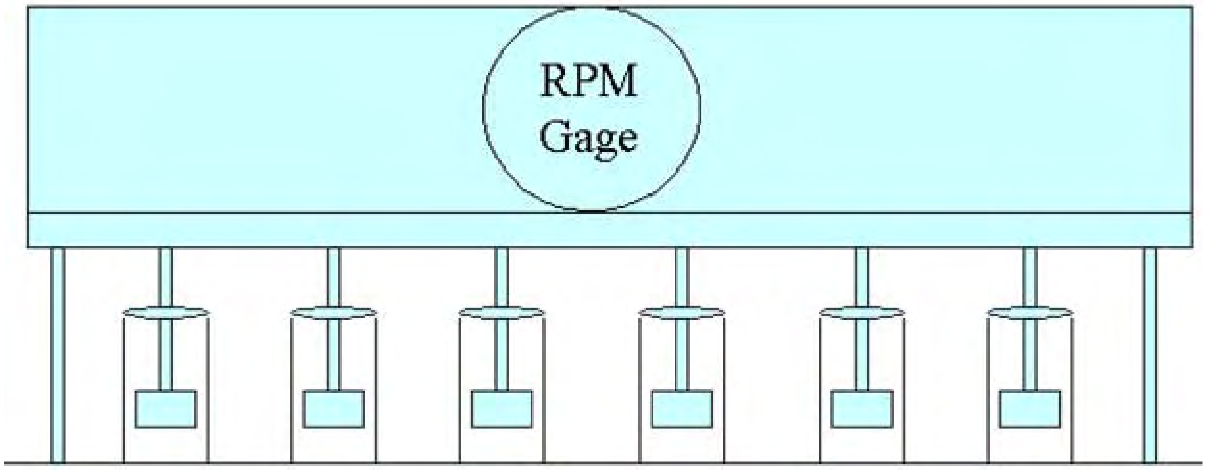
$$FeCl_3 + 3\ HCO^{3-} \longrightarrow Fe(OH)_3(S) + 3\ CO_2 + 3\ Cl^-$$

Jar Test

The jar test is a common laboratory procedure used to determine the optimum operating conditions for water or wastewater treatment. This method allows adjustments in pH, variations in coagulant or polymer dose, alternating mixing speeds, or testing of different coagulant or polymer types, on a small scale in order to predict the functioning of a large scale treatment operation.

Jar Testing Apparatus

The jar testing apparatus consists of six paddles which stir the contents of six 1 liter containers. One container acts as a control while the operating conditions can be varied among the remaining five containers. A rpm gage at the top-center of the device allows for the uniform control of the mixing speed in all of the containers.



Jar Test Procedure

- The jar test procedures involves the following steps:
  Fill the jar testing apparatus containers with sample water. One container will be used as a control while the other 5 containers can be adjusted depending on what conditions are being tested. For example, the pH of the jars can be adjusted or variations of coagulant dosages can be added to determine optimum operating conditions.
- Add the coagulant to each container and stir at approximately 100 rpm for 1 minute. The rapid mix stage helps to disperse the coagulant throughout each container.
- Turn off the mixers and allow the containers to settle for 30 to 45 minutes. Then measure the final turbidity in each container.
- Reduce the stirring speed to 25 to 35 rpm and continue mixing for 15 to 20 minutes. This slower mixing speed helps promote floc formation by enhancing particle collisions which lead to larger flocs.
- Residual turbidity vs. coagulant dose is then plotted and optimal conditions are determined. The values that are obtained through the experiment are correlated and adjusted in order to account for the actual treatment system.

Filtration

The resultant water after sedimentation will not be pure, and may contain some very fine suspended particles and bacteria in it. To remove or to reduce the remaining impurities still further, the water is filtered through the beds of fine granular material, such as sand, etc. The process of passing the water through the beds of such granular materials is known as Filtration.

How Filters Work: Filtration Mechanisms

There are four basic filtration mechanisms:
SEDIMENTATION : The mechanism of sedimentation is due to force of gravity and the associate settling velocity of the particle, which causes it to cross the streamlines and reach the collector.
INTERCEPTION : Interception of particles is common for large particles. If a large enough particle follows the streamline, that lies very close to the media surface it will hit the media grain and be captured.
BROWNIAN DIFFUSION : Diffusion towards media granules occurs for very small particles, such as viruses. Particles move randomly about within the fluid, due to thermal gradients. This mechanism is only important for particles with diameters < 1 micron.
INERTIA : Attachment by inertia occurs when larger particles move fast enough to travel off their streamlines and bump into media grains.

Filter Materials

Sand: Sand, either fine or coarse, is generally used as filter media. The size of the sand is measured and expressed by the term called effective size. *The effective size*, i.e. D_{10} may be defined as the size of the sieve in mm through which ten percent of the sample of sand by weight will pass. The uniformity in size or degree of variations in sizes of particles is measured and expressed by the term called *uniformity coefficient*. The uniformity coefficient, i.e. (D_{60}/D_{10}) may be defined as the ratio of the sieve size in mm through which 60 percent of the sample of sand will pass, to the effective size of the sand.

Gravel: The layers of sand may be supported on gravel, which permits the filtered water to move freely to the under drains, and allows the wash water to move uniformly upwards.

Other materials: Instead of using sand, sometimes, anthrafilt is used as filter media. Anthrafilt is made from anthracite, which is a type of coal-stone that burns without smoke or flames. It is cheaper and has been able to give a high rate of filtration.

Types of Filter

Slow sand filter: They consist of fine sand, supported by gravel. They capture particles near the surface of the bed and are usually cleaned by scraping away the top layer of sand that contains the particles.
Rapid-sand filter: They consist of larger sand grains supported by gravel and capture particles throughout the bed. They are cleaned by backwashing water through the bed to 'lift out' the particles.
Multimedia filters: They consist of two or more layers of different granular materials, with different densities. Usually, anthracite coal, sand, and gravel are used. The different layers combined may provide more versatile collection than a single sand layer. Because of the differences in densities, the layers stay neatly separated, even after backwashing.

Principles of Slow Sand Filtration

- In a slow sand filter impurities in the water are removed by a combination of processes: sedimentation, straining, adsorption, and chemical and bacteriological action.
- During the first few days, water is purified mainly by mechanical and physical-chemical processes. The resulting accumulation of sediment and organic matter forms a thin layer on the sand surface, which remains permeable and retains particles even smaller than the spaces between the sand grains.
- As this layer (referred to as “Schmutzdecke”) develops, it becomes living quarters of vast numbers of micro-organisms which break down organic material retained from the water, converting it into water, carbon dioxide and other oxides.
- Most impurities, including bacteria and viruses, are removed from the raw water as it passes through the filter skin and the layer of filter bed sand just below. The purification mechanisms extend from the filter skin to approx. 0.3-0.4 m below the surface of the filter bed, gradually decreasing in activity at lower levels as the water becomes purified and contains less organic material.
- When the micro-organisms become well established, the filter will work efficiently and produce high quality effluent which is virtually free of disease carrying organisms and biodegradable organic matter.
They are suitable for treating waters with low colors, low turbidities and low bacterial contents.

Sand Filters vs. Rapid Sand Filters

- ***Base material:*** In SSF it varies from 3 to 65 mm in size and 30 to 75 cm in depth while in RSF it varies from 3 to 40 mm in size and its depth is slightly more, i.e. about 60 to 90 cm.
- ***Filter sand:*** In SSF the effective size ranges between 0.2 to 0.4 mm and uniformity coefficient between 1.8 to 2.5 or 3.0. In RSF the effective size ranges between 0.35 to 0.55 and uniformity coefficient between 1.2 to 1.8.
- ***Rate of filtration:*** In SSF it is small, such as 100 to 200 L/h/sq.m. of filter area while in RSF it is large, such as 3000 to 6000 L/h/sq.m. of filter area.
- ***Flexibility:*** SSF are not flexible for meeting variation in demand whereas RSF are quite flexible for meeting reasonable variations in demand.
- ***Post treatment required:*** Almost pure water is obtained from SSF. However, water may be disinfected slightly to make it completely safe. Disinfection is a must after RSF.
- ***Method of cleaning:*** Scrapping and removing of the top 1.5 to 3 cm thick layer is done to clean SSF. To clean RSF, sand is agitated and backwashed with or without compressed air.
- ***Loss of head:*** In case of SSF approx. 10 cm is the initial loss, and 0.8 to 1.2m is the final limit when cleaning is required. For RSF 0.3m is the initial loss, and 2.5 to 3.5m is the final limit when cleaning is required.

Typical Rapid Gravity Filter Flow Operation

Isometric view of Rapid Sand Filter

Clean Water Headloss

Several equations have been developed to describe the flow of clean water through a porous medium. Carman-Kozeny equation used to calculate head loss is as follows:

$$h = \frac{f\,(1-\alpha)Lv_s^2}{\phi\alpha^3 dg}$$

$$h = \frac{f\,p(1-\alpha)Lv_s^2}{\phi\alpha^3 d_g g}$$

$$f = 150\,\frac{(1-\alpha)}{N_g} + 1.75$$

$$N_g = \frac{\phi d v_s \rho}{\mu}$$

where, h = headloss, m

f = friction factor

α = porosity

ϕ = particle shape factor (1.0 for spheres, 0.82 for rounded sand, 0.75 for average sand, 0.73 for crushed coal and angular sand)

L = depth of filter bed or layer, m

d = grain size diameter, m

v_s = superficial (approach) filtration velocity, m/s

g = accelaration due to gravity, 9.81 m/s^2

p = fraction of particles (based on mass) within adjacent sieve sizes

d_g = geometric mean diameter between sieve sizes d_1 and d_2

N_g = Reynolds number

μ = viscosity, $N\text{-}s/m^2$

Backwashing of Rapid Sand Filter

- For a filter to operate efficiently, it must be cleaned before the next filter run. If the water applied to a filter is of very good quality, the filter runs can be very long. Some filters can operate longer than one week before needing to be backwashed. However, this is not recommended as long filter runs can cause the filter media to pack down so that it is difficult to expand the bed during the backwash.
- Treated water from storage is used for the backwash cycle. This treated water is generally taken from elevated storage tanks or pumped in from the clear well.
- The filter backwash rate has to be great enough to expand and agitate the filter media and suspend the floc in the water for removal. However, if the filter backwash rate is too high, media will be washed from the filter into the troughs and out of the filter.

When is Backwashing Needed

The filter should be backwashed when the following conditions have been met:

- The head loss is so high that the filter no longer produces water at the desired rate; and/or
- Floc starts to break through the filter and the turbidity in the filter effluent increases; and/or
- A filter run reaches a given hour of operation.

Operational Troubles in Rapid Gravity Filters

Air Binding :

- When the filter is newly commissioned, the loss of head of water percolating through the filter is generally very small. However, the loss of head goes on increasing as more and more impurities get trapped into it.
- A stage is finally reached when the frictional resistance offered by the filter media exceeds the static head of water above the and bed. Most of this resistance is offered by the top 10 to 15 cm sand layer. The bottom sand acts like a vacuum, and water is sucked through the filter media rather than getting filtered through it.
- The negative pressure so developed, tends to release the dissolved air and other gases present in water. The formation of bubbles takes place which stick to the sand grains. This phenomenon is known as Air Binding as the air binds the filter and stops its functioning.
- To avoid such troubles, the filters are cleaned as soon as the head loss exceeds the optimum allowable value.

Formation of Mud Balls :

- The mud from the atmosphere usually accumulates on the sand surface to form a dense mat. During inadequate washing this mud may sink down into the sand bed and stick to the sand grains and other arrested impurities, thereby forming mud balls.

Cracking of Filters :

- The fine sand contained in the top layers of the filter bed shrinks and causes the development of shrinkage cracks in the sand bed. With the use of filter, the loss of head and, therefore, pressure on thesand bed goes on increasing, which further goes on widening these cracks.

Remedial Measures to Prevent Cracking of Filters and Formation of Mud Balls

- Breaking the top fine mud layer with rakes and washing off the particles.
- Washing the filter with a solution of caustic soda.
- Removing, cleaning and replacing the damaged filter sand.

Standard design practice of Rapid Sand filter: Maximum length of lateral = not less than 60 times its diameter. Spacing of holes = 6 mm holes at 7.5 cm c/c or 13 at 15 c/c. C.S area of lateral = not less than 2 times area of perforations. C.S area of manifold = 2 times total area of laterals. Maximum loss of head = 2 to 5 m. Spacing of laterals = 15 to 30 cm c/c. Pressure of wash water at perforations = not greater than 1.05 kg/cm^2. Velocity of flow in lateral = 2 m/s. Velocity of flow in manifold = 2.25 m/s. Velocity of flow in manifold for washwater= 1.8 to 2.5 m/s. Velocity of rising washwater= 0.5 to 1.0 m/min. Amount of washwater = 0.2 to 0.4% of total filtered water. Time of backwashing = 10 to 15 min. Head of water over the filter = 1.5 to 2.5 m. Free board = 60 cm. Bottom slope = 1 to 60 towards manifold.

$$Q = (1.71 \times b \times h^{3/2})$$

where Q is in m^3/s, b is in m, h is in m. L:B = 1.25 to 1.33:1 .

Disinfection

The filtered water may normally contain some harmful disease producing bacteria in it. These bacteria must be killed in order to make the water safe for drinking. The process of killing these bacteria is known as Disinfection or Sterilization.

Disinfection Kinetics

When a single unit of microorganisms is exposed to a single unit of disinfectant, the reduction in microorganisms follows a first-order reaction.

$dN/dt = -kN$ $\qquad$ $N = N_0 e^{-kt}$

This equation is known as Chick's Law:-

N = number of microorganism (N_0 is initial number)
k = disinfection constant
t = contact time

Methods of Disinfection

1. ***Boiling:*** The bacteria present in water can be destroyed by boiling it for a long time. However it is not practically possible to boil huge amounts of water. Moreover it cannot take care of future possible contaminations.
2. ***Treatment with Excess Lime:*** Lime is used in water treatment plant for softening. But if excess lime is added to the water, it can in addition, kill the bacteria also. Lime when added raises the pH value o water making it extremely alkaline. This extreme alkalinity has been found detrimental to the survival of bacteria. This method needs the removal of excess lime from the water before it can be supplied to the general public. Treatment like recarbonation for lime removal should be used after disinfection.
3. ***Treament with Ozone:*** Ozone readily breaks down into normal oxygen, and releases nascent oxygen. The nascent oxygen is a powerful oxidising agent and removes the organic matter as well as the bacteria from the water.
4. ***Chlorination:*** The germicidal action of chlorine is explained by the recent theory of *Enzymatic hypothesis*, according to which the chlorine enters the cell walls of bacteria and kill the enzymes which are essential for the metabolic processes of living organisms.

Chlorine Chemistry

Chlorine is added to the water supply in two ways. It is most often added as a gas, $Cl_2(g)$. However, it also can be added as a salt, such as sodium hypochlorite (NaOCl) or bleach. Chlorine gas dissolves in water following Henry's Law.

$$Cl_{2(g)} \longrightarrow Cl_{2(aq)} \quad K_H = 6.2 \times 10^{-2}$$

Once dissolved, the following reaction occurs forming hypochlorous acid (HOCl):

$$Cl_{2(aq)} + H_2O \longrightarrow HOCl + H^+ + Cl^-$$

Hypochlorous acid is a weak acid that dissociates to form hypochlorite ion (OCl-).

$$HOCl \longrightarrow OCl^- + H^+ \quad Ka = 3.2 \times 10^{-8}$$

All forms of chlorine are measured as mg/L of Cl_2 (MW = 2 x 35.45 = 70.9 g/mol)
Hypochlorous acid and hypochlorite ion compose what is called the free chlorine residual. These free chlorine compounds can react with many organic and inorganic compounds to form chlorinated compounds. If the products of these reactions posses oxidizing potential, they are considered the combined chlorine residual. A common compound in drinking water systems that reacts with chlorine to form combined residual is ammonia. Reactions between ammonia and chlorine form chloramines, which is mainly monochloramine (NH_2Cl), although some dichloramine ($NHCl_2$) and trichloramine (NCl_3) also can form. Many drinking water utilities use monochloramine as a disinfectant. If excess free chlorine exits once all ammonia nitrogen has been converted to monochloramine, chloramine species are oxidized through what is termed the breakpoint reactions. The overall reactions of free chlorine and nitrogen can be represented by two simplified reactions as follows:

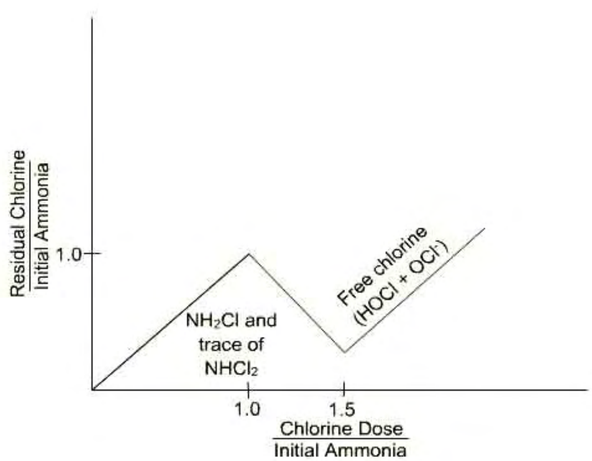
Monochloramine Formation Reaction. This reaction occurs rapidly when ammonia nitrogen is combined with free chlorine up to a molar ratio of 1:1.

$$HOCl + NH_3 \longrightarrow NH_2Cl + HOCl$$

Breakpoint Reaction: When excess free chlorine is added beyond the 1:1 initial molar ratio, monochloramine is removed as follows:

$$2NH_2Cl + HOCl \longrightarrow N_{2(g)} + 3H^+ + 3Cl^- + H_2O$$

The formation of chloramines and the breakpoint reaction create a unique relationship between chlorine dose and the amount and form of chlorine as illustrated below.



Free Chlorine, Chloramine, and Ammonia Nitrogen Reactions

Chlorine Demand

Free chlorine and chloramines readily react with a variety compounds, including organic substances, and inorganic substances like iron and manganese. The stoichiometry of chlorine reactions with organics can be represented as shown below:

HOCl:

$$1/10C_5H_7O_2N + HOCl \longrightarrow 4/10CO_2 + 1/10HCO^{3-} + 1/10NH^{4+} + H^+ + Cl^- + 1/10H_2O$$

OCl$^-$:

$$1/10C_5H_7O_2N + OCl^- \longrightarrow 4/10CO_2 + 1/10HCO^{3-} + 1/10NH^{4+} + Cl^- + 1/10H_2O$$

NH$_2$Cl:

$$1/10C_5H_7O_2N + NH_2Cl + 9/10H_2O \longrightarrow 4/10CO_2 + 1/10HCO^{3-} + 11/10NH^{4+} + Cl^-$$

Chlorine demand can be increased by oxidation reactions with inorganics, such as reduced iron at corrosion sites at the pipe wall. Possible reactions with all forms of chlorine and iron are as follows:

Treatment Plant Layout and Siting

Plant layout is the arrangement of designed treatment units on the selected site. ***Siting*** is the selection of site for treatment plant based on features as character, topography, and shoreline. Site development should take the advantage of the existing site topography. The following principles are important to consider:

1. A site on a side-hill can facilitate gravity flow that will reduce pumping requirements and locate normal sequence of units without excessive excavation or fill.
2. When landscaping is utilized it should reflect the character of the surrounding area. Site development should alter existing naturally stabilized site contours and drainage as little as possible.
3. The developed site should be compatible with the existing land uses and the comprehensive development plan.

Treatment Plant Hydraulics

Hydraulic profile is the graphical representation of the hydraulic grade line through the treatment plant. The head loss computations are started in the direction of flow using water surface in the influent of first treatment unit as the reference level. The ***total available head*** at the treatment plant is the difference in water surface elevations in the influent of first treatment unit and that in the effluent of last treatment unit. If the total available head is less than the head loss through the plant, flow by gravity cannot be achieved. In such cases pumping is needed to raise the head so that flow by gravity can occur.

There are many basic principles that must be considered when preparing the hydraulic profile through the plant. Some are listed below:

1. The hydraulic profiles are prepared at peak and average design flows and at minimum initial flow.
2. The hydraulic profile is generally prepared for all main paths of flow through the plant.
3. The head loss through the treatment plant is the sum of head losses in the treatment units and the connecting piping and appurtenances.
4. The head losses through the treatment unit include the following:
    a. Head losses at the influent structure.
    b. Head losses at the effluent structure.
    c. Head losses through the unit.
    d. Miscellaneous and free fall surface allowance.
5. The total loss through the connecting pipings, channels and appurtenances is the sum of following:
    a. Head loss due to entrance.
    b. Head loss due to exit.
    c. Head loss due to contraction and enlargement.
    d. Head loss due to friction.
    e. Head loss due to bends, fittings, gates, valves, and meters.
    f. Head required over weir and other hydraulic controls.
    g. Free-fall surface allowance.

Water Distribution Systems

The purpose of distribution system is to deliver water to consumer with appropriate quality, quantity and pressure. Distribution system is used to describe collectively the facilities used to supply water from its source to the point of usage.

Requirements of Good Distribution System

1. Water quality should not get deteriorated in the distribution pipes.
2. It should be capable of supplying water at all the intended places with sufficient pressure head.
3. It should be capable of supplying the requisite amount of water during fire fighting.
4. The layout should be such that no consumer would be without water supply, during the repair of any section of the system.
5. All the distribution pipes should be preferably laid one metre away or above the sewer lines.
6. It should be fairly water-tight as to keep losses due to leakage to the minimum.

Layouts of Distribution Network

The distribution pipes are generally laid below the road pavements, and as such their layouts generally follow the layouts of roads. There are, in general, four different types of pipe networks; any one of which either singly or in combinations, can be used for a particular place. They are:

Dead End System
Grid Iron System
Ring System
Radial System

Distribution Reservoirs

Distribution reservoirs, also called service reservoirs, are the storage reservoirs, which store the treated water for supplying water during emergencies (such as during fires, repairs, etc.) and also to help in absorbing the hourly fluctuations in the normal water demand.

Functions of Distribution Reservoirs:

- to absorb the hourly variations in demand.
- to maintain constant pressure in the distribution mains.
- water stored can be supplied during emergencies.

Location and Height of Distribution Reservoirs:

- should be located as close as possible to the center of demand.
- water level in the reservoir must be at a sufficient elevation to permit gravity flow at an adequate pressure.

Types of Reservoirs

1. Underground reservoirs.
2. Small ground level reservoirs.
3. Large ground level reservoirs.
4. Overhead tanks.

Storage Capacity of Distribution Reservoirs

The total storage capacity of a distribution reservoir is the summation of:

1. ***Balancing Storage:*** The quantity of water required to be stored in the reservoir for equalising or balancing fluctuating demand against constant supply is known as the balancing storage (or equalising or operating storage). The balance storage can be worked out by *mass curve method*.
2. ***Breakdown Storage:*** The breakdown storage or often called emergency storage is the storage preserved in order to tide over the emergencies posed by the failure of pumps, electricity, or any othe mechanism driving the pumps. A value of about 25% of the total storage capacity of reservoirs, or 1.5 to 2 times of the average hourly supply, may be considered as enough provision for accounting this storage.
3. ***Fire Storage:*** The third component of the total reservoir storage is the fire storage. This provision takes care of the requirements of water for extinguishing fires. A provision of 1 to 4 per person per day is sufficient to meet the requirement.

The total reservoir storage can finally be worked out by adding all the three storages.

Pipe Network Analysis

Analysis of water distribution system includes determining quantities of flow and head losses in the various pipe lines, and resulting residual pressures. In any pipe network, the following two conditions must be satisfied:

1. The algebraic sum of pressure drops around a closed loop must be zero, i.e. there can be no discontinuity in pressure.
2. The flow entering a junction must be equal to the flow leaving that junction; i.e. the law of continuity must be satisfied.

Based on these two basic principles, the pipe networks are generally solved by the methods of successive approximation. The widely used method of pipe network analysis is the Hardy-Cross method.

Hardy-Cross Method

This method consists of assuming a distribution of flow in the network in such a way that the principle of continuity is satisfied at each junction. A correction to these assumed flows is then computed successively for each pipe loop in the network, until the correction is reduced to an acceptable magnitude.

If Q_a is the assumed flow and Q is the actual flow in the pipe, then the correction δ is given by

$\delta = Q - Q_a$; or $Q = Q_a + \delta$

Now, expressing the head loss (H_L) as

$H_L = K.Q^x$

we have, the head loss in a pipe

$= K.(Q_a + \delta)^x$

$= K.[Q_a^x + x.Q_a^{x-1}\delta + ...$negligible terms]

$= K.[Q_a^x + x.Q_a^{x-1}\delta]$

Now, around a closed loop, the summation of head losses must be zero.

$\therefore \Sigma K.[Q_a^x + x.Q_a^{x-1}\delta] = 0$

or $\Sigma K.Q_a^x = -\Sigma Kx\, Q_a^{x-1}\delta$

Since, δ is the same for all the pipes of the considered loop, it can be taken out of the summation.

$\therefore \Sigma K.Q_a^x = -\delta.\Sigma Kx\, Q_a^{x-1}$

or $\delta = -\Sigma K.Q_a^x / \Sigma x.KQ_a^{x-1}$

Since δ is given the same sign (direction) in all pipes of the loop, the denominator of the above equation is taken as the absolute sum of the individual items in the summation. Hence,

or $\delta = -\Sigma K.Q_a^x / \Sigma\, |\, x.KQ_a^{x-1}\, |$

or $\delta = -\Sigma H_L \,/\, x.\Sigma\, |H_L/Q_a|$

where H_L is the head loss for assumed flow Q_a.

The numerator in the above equation is the algebraic sum of the head losses in the various pipes of the closed loop computed with assumed flow. Since the direction and magnitude of flow in these pipes is already assumed, their respective head losses with due regard to sign can be easily calculated after assuming their diameters. The absolute sum of respective KQ_a^{x-1} or H_L/Q_a

is then calculated. Finally the value of δ is found out for each loop, and the assumed flows are corrected. Repeated adjustments are made until the desired accuracy is obtained.

The value of x in Hardy- Cross method is assumed to be constant (i.e. 1.85 for Hazen-William's formula, and 2 for Darcy-Weisbach formula)

Raw Water Source

The various sources of water can be classified into two categories:

1. Surface sources, such as
   a. Ponds and lakes;
   b. Streams and rivers;
   c. Storage reservoirs; and
   d. Oceans, generally not used for water supplies, at present.
2. Sub-surface sources or underground sources, such as
   a. Springs;
   b. Infiltration wells ; and
   c. Wells and Tube-wells.

Water Quality

The raw or treated water is analysed by testing their physical, chemical and bacteriological characteristics:

Physical Characteristics:

Turbidity
Colour
Taste and Odour
Temperature

Chemical Characteristics:

pH
Acidity
Alkalinity
Hardness
Chlorides
Sulphates
Iron
Solids
Nitrates

Bacteriological Characteristics:

Bacterial examination of water is very important, since it indicates the degree of pollution. Water polluted by sewage contain one or more species of disease producing pathogenic bacteria. Pathogenic organisms cause water borne diseases, and many non pathogenic bacteria such as *E.Coli*, a member of coliform group, also live in the intestinal tract of human beings. *Coliform* itself is not a harmful group but it has more resistance to adverse condition than any other group. So, if it is ensured to minimize the number of coliforms, the harmful species will be very less. So, coliform group serves as indicator of contamination of water with sewage and presence of pathogens.

The methods to estimate the bacterial quality of water are:

Standard Plate Count Test
Most Probable Number
Membrane Filter Technique

Wastewater Quantity Estimation

The flow of sanitary sewage alone in the absence of storms in dry season is known as dry weather flow (DWF).

Quantity= Per capita sewage contributed per day x Population

Sanitary sewage is mostly the spent water of the community draining into the sewer system. It has been observed that a small portion of spent water is lost in evaporation, seepage in ground, leakage, etc. Usually 80% of the water supply may be expected to reach the sewers.

Fluctuations in Dry Weather Flow

Since dry weather flow depends on the quantity of water used, and as there are fluctuations in rate of water consumption, there will be fluctuations in dry weather flow also. In general, it can be assumed that (i) Maximum daily flow = 2 x average daily flow and (ii) Minimum daily flow = 2/3 x (average daily flow).

Population Equivalent

Population equivalent is a parameter used in the conversion of contribution of wastes from industrial establishments for accepting into sanitary sewer systems. The strength of industrial sewage is, thus, written as

Std. BOD_5 = (Std. BOD_5 of domestic sewage per person per day) x
(population equivalent)

Design Periods & Population Forecast

This quantity should be worked out with due provision for the estimated requirements of the future . The future period for which a provision is made in the water supply scheme is known as the **design period**. It is suggested that the construction of sewage treatment plant may be carried out in phases with an initial design period ranging from 5 to 10 years excluding the construction period.

Design period is estimated based on the following:

- Useful life of the component, considering obsolescence, wear, tear, etc.
- Expandability aspect.
- Anticipated rate of growth of population, including industrial, commercial developments & migration-immigration.
- Available resources.
- Performance of the system during initial period.

Population forecasting methods:

The various methods adopted for estimating future populations are given below. The particular method to be adopted for a particular case or for a particular city depends largely on the factors discussed in the methods, and the selection is left to the discrection and intelligence of the designer.

1. *Arithmetic Increase Method*
2. *Geometric Increase Method*
3. *Incremental Increase Method*
4. *Decreasing Rate of Growth Method*
5. *Simple Graphical Method*
6. *Comparative Graphical Method*
7. *Ratio Method*
8. *Logistic Curve Method*

Wastewater Characterization

To design a treatment process properly, characterization of wastewater is perhaps the most critical step. Wastewater characteristics of importance in the design of the activated sludge process can be grouped into the following categories:

Temperature
pH
Colour and Odour
Carbonaceous substrates
Nitrogen

Phosphorous
Chlorides
Total and volatile suspended solids (TSS and VSS)
Toxic metals and compounds

Design of Sewers

The hydraulic design of sewers and drains, which means finding out their sections and gradients, is generally carried out on the same lines as that of the water supply pipes. However, there are two major differences between characteristics of flows in sewers and water supply pipes. They are:

- The sewage contain particles in suspension, the heavier of which may settle down at the bottom of the sewers, as and when the flow velocity reduces, resulting in the clogging of sewers. To avoid silting of sewers, it is necessary that the sewer pipes be laid at such a gradient, as to generate self cleansing velocities at different possible discharges.
- The sewer pipes carry sewage as gravity conduits, and are therefore laid at a continuous gradient in the downward direction upto the outfall point, from where it will be lifted up, treated and disposed of.

Hazen-William's formula

$$U=0.85\ C\ r_H^{0.63}S^{0.54}$$

Manning's formula

$$U={}^{1}/_{n}\ r_H^{2/3}S^{1/2}$$

where, U= velocity, m/s; r_H= hydraulic radius,m; S= slope, C= Hazen-William's coefficient, and n = Manning's coefficient.

Darcy-Weisbach formula

$$h_L=(fLU^2)/(2gd)$$

Minimum Velocity

The flow velocity in the sewers should be such that the suspended materials in sewage do not get silted up; i.e. the velocity should be such as to cause automatic self-cleansing effect. The generation of such a minimum *self cleansing velocity* in the sewer, atleast once a day, is important, because if certain deposition takes place and is not removed, it will obstruct free flow, causing further deposition and finally leading to the complete blocking of the sewer.

Maximum Velocity

The smooth interior surface of a sewer pipe gets scoured due to continuous abrasion caused by the suspended solids present in sewage. It is, therefore, necessary to limit the maximum velocity in the sewer pipe. This limiting or non-scouring velocity will mainly depend upon the material of the sewer.

Effects of Flow Variation on Velocity in a Sewer

Due to variation in discharge, the depth of flow varies, and hence the hydraulic mean depth (r) varies. Due to the change in the hydraulic mean depth, the flow velocity (which depends directly on $r^{2/3}$) gets affected from time to time. It is necessary to check the sewer for maintaining a minimum velocity of about 0.45 m/s at the time of minimum flow (assumed to be 1/3rd of average flow). The designer should also ensure that a velocity of 0.9 m/s is developed atleast at the time of maximum flow and preferably during the average flow periods also. Moreover, care should be taken to see that at the time of maximum flow, the velocity generated does not exceed the scouring value.

Sewer Appurtenances

Sewer appurtenances are the various accessories on the sewerage system and are necessary for the efficient operation of the system. They include man holes, lamp holes, street inlets, catch basins, inverted siphons, and so on.

Man-holes: Man holes are the openings of either circular or rectangular in shape constructed on the alignment of a sewer line to enable a person to enter the sewer for inspection, cleaning and flushing. They serve as ventilators for sewers, by the provisions of perforated man-hole covers. Also they facilitate the laying of sewer lines in convenient length.

Man-holes are provided at all junctions of two or more sewers, whenever diameter of sewer changes, whenever direction of sewer line changes and when sewers of different elevations join together.

Special Man-holes:

Junction chambers: Man-hole constructed at the intersection of two large sewers.

Drop man-hole: When the difference in elevation of the invert levels of the incoming and outgoing sewers of the man-hole is more than 60 cm, the interception is made by dropping the incoming sewer vertically outside and then it is jointed to the man-hole chamber.

Flushing man-holes: They are located at the head of a sewer to flush out the deposits in the sewer with water.

Lamp-holes: Lamp holes are the openings constructed on the straight sewer lines between two man-holes which are far apart and permit the insertion of a lamp into the sewer to find out obstructions if any inside the sewers from the next man-hole.

Street inlets: Street inlets are the openings through which storm water is admitted and conveyed to the storm sewer or combined sewer. The inlets are located by the sides of pavement with maximum spacing of 30 m.

Catch Basins: Catch basins are small settling chambers of diameter 60 - 90 cm and 60 - 75 cm deep, which are constructed below the street inlets. They interrupt the velocity of storm water entering through the inlets and allow grit, sand, debris and so on to settle in the basin, instead of allowing them to enter into the sewers.

Inverted siphons: These are depressed portions of sewers, which flow full under pressure more than the atmospheric pressure due to flow line being below the hydraulic grade line. They are constructed when a sewer crosses a stream or deep cut or road or railway line. To clean the siphon pipe sluice valve is opened, thus increasing the head causing flow. Due to increased velocity deposits of siphon pipe are washed into the sump, from where they are removed.

Pumping of Sewage

Pumping of sewage is required when it is not possible to have a gravitational flow for the entire sewerage project.

Sufficient pumping capacity has to be provided to meet the peak flow, atleast 50% as stand by.

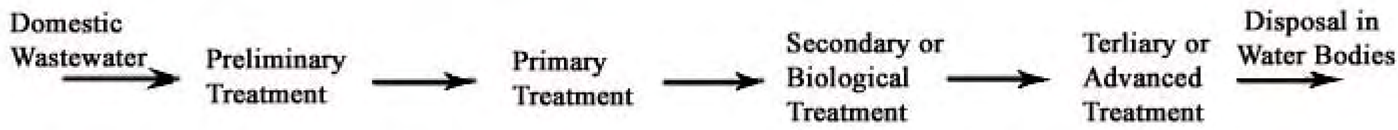
Types of pumps :

1. Centrifugal pumps either axial, mixed and radial flow.
2. Pneumatic ejector pumps.

Water Treatment

The raw sewage must be treated before it is discharged into the river stream. The extent of treatment required to be given depends not only upon the characteristics and quality of the sewage but also upon the source of disposal, its quality and capacity to tolerate the impurities

The layout of conventional wastewater treatment plant is as follows:



Siting and Hydraulics of Wastewater Treatment Plant

Indian Standards for discharge of sewage in surface waters are given in the table below.

Indian Standards for Discharge of Sewage in Surface Waters

Characteristic of the Effluent	Tolerance *limit for Discharge of Sewage in Suface Water Sources*
BOD_5	20 mg/L
TSS	30 mg/L

The unit operations and processes commonly employed in domestic wastewater treatment, their functions and units used to achieve these functions are given in the following table:

Unit Operations/Processes, Their Functions and Units Used for Domestic Wastewater Treatment

Unit Operations/Processes	Functions	Treatment Devices
Screening	Removal of large floating, suspended and settleable solids	Bar racks and screens of various description
Grit Removal	Removal of inorganic suspended solids	Grit chamber
Primary Sedimentation	Removal of organic/inorganic settleable solids	Primary sedimentation tank
Aerobic Biological Suspended Growth Process	Conversion of colloidal, dissolved and residual suspended organic matter into settleable biofloc and stable inorganics	Activated sludge process units and its modifications, Waste stabilisation ponds, Aerated lagoons
Aerobic Biological Attached Growth Process	same as above	Trickling filter, Rotating biological contactor
Anaerobic biological growth processes	Conversion of organic matter into CH_4 & CO_2 and relatively stable organic residue	Anaerobic filter, Fluid bed submerged media anaerobic reactor, Upflow anaerobic sludge blanket reactor, Anaerobic rotating biological contactor
Anaerobic Stabilization of Organic Sludges	same as above	Anaerobic digestor

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.

Types of Screens

Coarse Screens: Coarse screens also called racks, are usually bar screens, composed of vertical or inclined bars spaced at equal intervals across a channel through which sewage flows. Bar screens with relatively large openings of 75 to 150 mm are provided ahead of pumps, while those ahead of sedimentation tanks have smaller openings of 50 mm.

Bar screens are usually hand cleaned and sometimes provided with mechanical devices. These cleaning devices are rakes which periodically sweep the entire screen removing the solids for further processing or disposal. Hand cleaned racks are set usually at an angle of 45° to the horizontal to increase the effective cleaning surface and also facilitate the raking operations. Mechanical cleaned racks are generally erected almost vertically. Such bar screens have openings 25% in excess of the cross section of the sewage channel.

Medium Screens: Medium screens have clear openings of 20 to 50 mm.Bar are usually 10 mm thick on the upstream side and taper slightly to the downstream side. The bars used for screens are rectangular in cross section usually about 10 x 50 mm, placed with larger dimension parallel to the flow.

Fine Screens: Fine screens are mechanically cleaned devices using perforated plates, woven wire cloth or very closely spaced bars with clear openings of less than 20 mm. Fine screens are not normally suitable for sewage because of clogging possibilities.

The most commonly used bar type screen is shown in figure:

Velocity

The velocity of flow ahead of and through the screen varies and affects its operation. The lower the velocity through the screen, the greater is the amount of screenings that would be removed from sewage. However, the lower the velocity, the greater would be the amount of solids deposited in the channel. Hence, the design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of ***0.6 to 1.2 mps through the open area for the peak flows*** have been used satisfactorily. Further, the velocity at low flows in the approach channel should ***not be less than 0.3 mps*** to avoid deposition of solids.

Head loss

Head loss varies with the quantity and nature of screenings allowed to accumulate between cleanings. The head loss created by a clean screen may be calculated by considering the flow and the effective areas of screen openings, the latter being the sum of the vertical projections of the openings. The head loss through clean flat bar screens is calculated from the following formula:

$$h = 0.0729\ (V^2 - v^2)$$

where, h = head loss in m

V = velocity through the screen in mps

v = velocity before the screen in mps

Another formula often used to determine the head loss through a bar rack is Kirschmer's equation:

$$h = \beta\ (W/b)^{4/3}\ h_v \sin\theta$$

where h = head loss,m

β = bar shape factor (2.42 for sharp edge rectangular bar, 1.83 for rectangular bar with semicircle upstream, 1.79 for circular bar and 1.67 for rectangular bar with both u/s and d/s face as semicircular).

W = maximum width of bar u/s of flow, m

b = minimum clear spacing between bars, m

h_v = velocity head of flow approaching rack, m = $v^2/2g$

θ = angle of inclination of rack with horizontal

The head loss through fine screen is given by

$$h = (1/2g)\ (Q/CA)$$

where, h = head loss, m
Q = discharge, m^3/s

C = coefficient of discharge (typical value 0.6)
A = effective submerged open area, m^2

The quantity of screenings depends on the nature of the wastewater and the screen openings.

Equalization Tanks

The equalization tanks are provided (i) to balance fluctuating flows or concentrations, (ii) to assist self neutralization, or (iii) to even out the effect of a periodic "slug" discharge from a batch process.

Types of Equalization Tanks

Equalization tanks are generally of three types:

1. Flow through type
2. Intermittent flow type
3. Variable inflow/constant discharge type

The simple ***flow through type*** equalization tank is mainly useful in assisting self neutralization or evening out of fluctuating concentrations, not for balancing of flows since a flow through type tank once filled, gives output equal to input.

Flow balancing and self-neutralization are both achieved by using two tanks, intermittently one after another. One tank is allowed to fill up after which it is checked for pH (or any other parameter) and then allowed to empty out. The second tank goes through a similar routine. ***Intermittent flow type*** tanks are economic for small flows from industries.

When flows are large an equalization tank of such a size may have to be provided that ***inflow can be variable while outflow is at a constant rate***, generally by a pump.The capacity required is determined from a plot of the cumulative inflow and a plot of the constant rate outflow and measuring the gaps between the two plots. A factor of safety may be applied if desired.

Generally, ***detention time*** vary from 2 to 8 hours but may be even 12 hours or more in some cases. When larger detention times are required, the equalization unit is sometimes provided in the form of facultative aerated lagoon.

Grit Chambers

Grit chambers are basin to remove the inorganic particles to prevent damage to the pumps, and to prevent their accumulation in sludge digestors.

Types of Grit Chambers

Grit chambers are of two types: mechanically cleaned and manually cleaned. In ***mechanically cleaned*** grit chamber, scraper blades collect the grit settled on the floor of the grit chamber. The grit so collected is elevated to the ground level by several mechanisms such as bucket elevators, jet pump and air lift. The grit washing mechanisms are also of several designs most of which are agitation devices using either water or air to produce washing action. ***Manually cleaned*** grit chambers should be cleaned atleast once a week. The simplest method of cleaning is by means of shovel.

Aerated Grit Chamber

An aerated grit chamber consists of a standard spiral flow aeration tank provided with air diffusion tubes placed on one side of the tank. The grit particles tend to settle down to the bottom of the tank at rates dependant upon the particle size and the bottom velocity of roll of the spiral flow, which in turn depends on the rate of air diffusion through diffuser tubes and shape of aeration tank. The heavier particles settle down whereas the lighter organic particles are carried with roll of the spiral motion.

Principle of Working of Grit Chamber

Grit chambers are nothing but like sedimentation tanks, designed to separate the intended heavier inorganic materials (specific gravity about 2.65) and to pass forward the lighter organic materials. Hence, the flow velocity should neither be too low as to cause the settling of lighter organic matter, nor should it be too high as not to cause the settlement of the silt and grit present in the sewage. This velocity is called "differential sedimentation and differential scouring velocity". The scouring velocity determines the optimum ***flow through velocity***. This may be explained by the fact that the critical velocity of flow 'v_c' beyond which particles of a certain size and density once settled, may be again introduced into the stream of flow. It should always be less than the scouring velocity of grit particles. The critical velocity of scour is given by Schield's formula:

$V = 3 \text{ to } 4.5\ (g(S_s - 1)d)^{1/2}$

A horizontal velocity of flow of 15 to 30 cm/sec is used at peak flows. This same velocity is to be maintained at all fluctuation of flow to ensure that only organic solids and not the grit is scoured from the bottom.

Types of Velocity Control Devices

1. A sutro weir in a channel of rectangular cross section, with free fall downstream of the channel.
2. A parabolic shaped channel with a rectangular weir.
3. A rectangular shaped channel with a parshall flume at the end which would also help easy flow measurement.

Design of Grit Chambers

Settling Velocity

The settling velocity of discrete particles can be determined using appropriate equation depending upon Reynolds number.

- Stoke's law: $v = \frac{g(S_s-1)d^2}{18\upsilon}$

Stoke's law holds good for Reynolds number, R_e below 1.

$R_e = \frac{vd}{\upsilon}$

For grit particles of specific gravity 2.65 and liquid temperature at 10°C, $\upsilon = 1.01 \times 10^{-6} m^2/s$. This corresponds to particles of size less than 0.1 mm.

- Transition law: The design of grit chamber is based on removal of grit particles with minimum size of 0.15 mm and therefore Stoke's law is not applicable to determine the settling velocity of grit particles for design purposes.

$v^2 = \frac{4g(\rho_p-\rho)d}{3\ C_D\rho}$

where, C_D = drag coefficient Transition flow conditions hold good for Reynolds number, R_e between 1 and 1000. In this range C_D can be approximated by

$C_D = \frac{18.5}{R_e^{0.6}} = \frac{18.5}{(vd/\upsilon)^{0.6}}$

Substituting the value of C_D in settling velocity equation and simplifying, we get

$v = [0.707(S_s-1)d^{1.6}\upsilon^{-0.6}]^{0.714}$

Primary Sedimentation

Primary sedimentation in a municipal wastewater treatment plant is generally plain sedimentation without the use of chemicals. In treating certain industrial wastes chemically aided sedimentation may be involved. In either case, it constitutes ***flocculent settling***, and the particles do not remain discrete as in the case of grit, but tend to agglomerate or coagulate during settling. Thus, their diameter keeps increasing and settlement proceeds at an over increasing velocity. Consequently, they trace a curved profile.

The settling tank design in such cases depends on both ***surface loading*** and ***detention time***.

Long tube settling tests can be performed in order to estimate specific value of surface loading and detention time for desired efficiency of clarification for a given industrial wastewater using recommended methods of testing. Scale-up factors used in this case range from 1.25 to 1.75 for the overflow rate, and from 1.5 to 2.0 for detention time when converting laboratory results to the prototype design.

For primary settling tanks treating municipal or domestic sewage, laboratory tests are generally not necessary, and recommended design values given in table may be used. Using an appropriate value of surface loading from table, the required tank area is computed. Knowing the average depth, the detention time is then computed. Excessively high detention time (longer than 2.5 h) must be avoided especially in warm climates where anaerobicity can be quickly induced.

Design parameters for settling tank

Types of settling	Overflow rate m^3m^2/day		Solids 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 filters	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	-

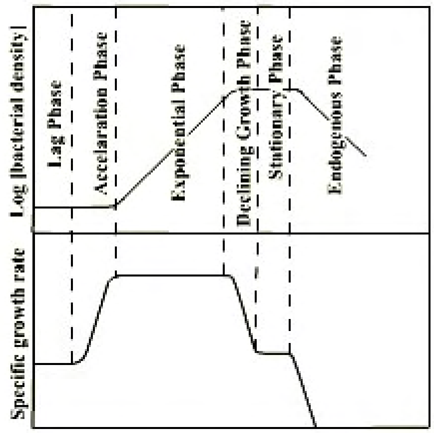
Classification of Micro organisms

1. ***Nutritional Requirements***: On the basis of chemical form of carbon required, microorganisms are classified as
   a. Autotrophic: organisms that use CO_2 or HCO_3^- as their sole source of carbon.
   b. Heterotrophic: organisms that use carbon from organic compounds.
2. ***Energy Requirements***: On the basis of energy source required, microorganisms are classified as
   . Phototrophs: organisms that use light as their energy source.
   a. Chemotrophs: organisms that employ oxidation-reduction reactions to provide energy. They are further classified on the basis of chemical compounds oxidized (i.e., electron donor)
      i. Chemoorganotrophs: Organisms that use complex organic molecules as their electron donor.
      ii. Chemoautotrophs: Organisms that use simple inorganic molecules such as hydrogen sulfide or ammonia as their electron donor.
3. ***Temperature Range***: On the basis of temperature range within which they can proliferate, microorganisms are classified as
   . Psychrophilic: organisms whose growth is optimum within 15 to 30°C.
   a. Mesophilic: organisms whose growth is optimum within 30 to 45°C.
   b. Thermophilic: organisms whose growth is optimum within 45 to 70°C.
4. ***Oxygen Requirements***: On the basis of oxygen requirement microorganisms are classified as
   . Aerobes: organisms that use molecular oxygen as electron acceptor.
   a. Anaerobes: organisms that use some molecule other than molecular oxygen as electron acceptor.
   b. Facultative organisms : organisms that can use either molecular oxygen or some other chemical compound as electron acceptor.

Growth Pattern of Micro organisms

When a small number of viable bacterial cells are placed in a close vessel containing excessive food supply in a suitable environment, conditions are established in which unrestricted growth takes place. However, growth of an organism do not go on indefinitely, and after a characteristic size is reached, the cell divides due to hereditary and internal limitations. The growth rate may follow a pattern similar to as shown in figure:

Characteristic Growth Curves of Cultures of Microorganisms



The curve shown may be divided into six well defined phases:

1. *Lag Phase*:adaptation to new environment, long generation time and null growth rate.
2. *Accelaration phase*: decreasing generation time and increasing growth rate.
3. *Exponential phase*: minimal and constant generation time, maximal and constant specific growth rate and maximum rate of substrate conversion.
4. *Declining growth phase*: increasing generation time and decreasing specific growth rate due to gradual decrease in substrate concentration and increased accumulation of toxic metabolites.
5. *Stationary phase*: exaustion of nutrients, high concentration of toxic metabolites, and cells in a state of suspended animation.
6. *Endogenous phase*: endogenous metabolism, high death rate and cell lysis.

Biomass Growth Rate

The most widely used expression for the growth rate of micro organisms is given by Monod:

Total rate of microbial growth, $\frac{dx}{dt} = \frac{\mu_m XS}{K_s + S}$

where,

μ_m = maximum specific growth rate

X = micro organism concentration

S = substrate concentration

K_s = substrate concentration at one half the maximum growth rate

Similarly, rate of substrate utilization,

$$\frac{dS}{dt} = \frac{k\,X\,S}{K_s + S}$$

where,

k = maximum specific substrate utilization rate

Maintenance as Endogenous Respiration

Net growth rate of micro organisms is computed by subtracting from the total growth rate, the rate of micro organisms endogenously decayed to satisfy maintenance energy requirement. Therefore,

$$\text{Net rate of microbial growth} = \frac{\mu_m X\, S}{K_s + S} - k_d X$$

where,

k_d = endogenous decay coefficient

Growth Yield

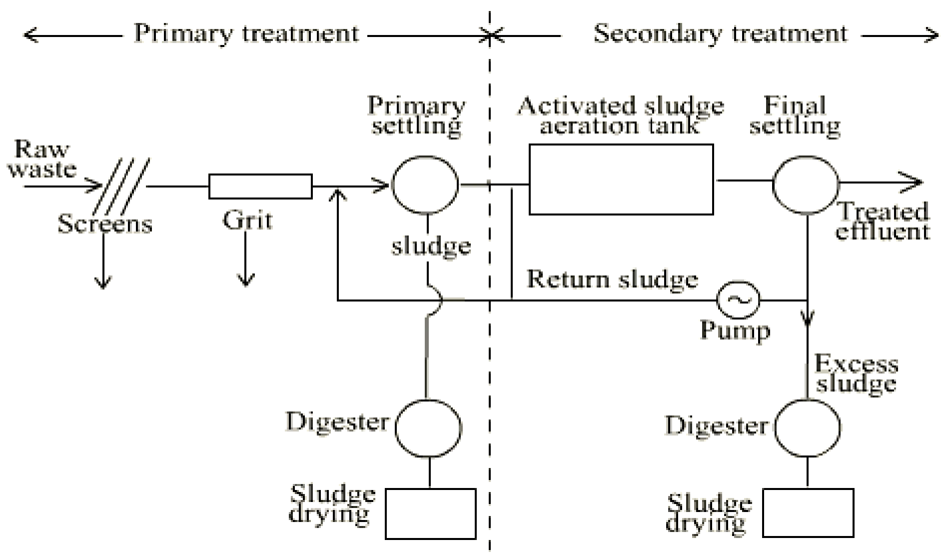
Growth yield is defined as the incremental increase in biomass which results from the utilization of the incremental amount of substrate. The maximum specific growth rate is given by: $\mu_m = Y.k$

where, Y is the maximum yield coefficient and is defined as the ratio of maximum mass of cells formed to the mass of substrate utilized. The coefficients Y, k_d, k and K_s are designated as kinetic coefficients. The values of kinetic coefficients depend upon the nature of wastewater and operational and environmental conditions in biological reactor. The biological reactors can be completely mixed flow or plug flow reactor with or without recycle.

Activated Sludge Process

The most common suspended growth process used for municipal wastewater treatment is the activated sludge process as shown in figure:

Flow sheet of an activated sludge system



Activated sludge plant involves:

1. wastewater aeration in the presence of a microbial suspension,
2. solid-liquid separation following aeration,
3. discharge of clarified effluent,
4. wasting of excess biomass, and
5. return of remaining biomass to the aeration tank.

In activated sludge process wastewater containing organic matter is aerated in an aeration basin in which micro-organisms metabolize the suspended and soluble organic matter. Part of organic matter is synthesized into new cells and part is oxidized to CO_2 and water to derive energy. In activated sludge systems the new cells formed in the reaction are removed from the liquid stream in the form of a flocculent sludge in settling tanks. A part of this settled biomass, described as activated sludge is returned to the aeration tank and the remaining forms waste or excess sludge.

Activated Sludge Process Variables

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

Mixing Regime

Generally two types of mixing regimes are of major interest in 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 no element of mixed liquor overtaking or mixing with any other element. There may be lateral mixing of mixed liquor but there must be no mixing along the path of flow.

In 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 contents.

The type of mixing regime is very important as it affects (1) oxygen transfer requirements in the aeration tank, (2) susceptibility of biomass to shock loads, (3) local environmental conditions in the aeration tank, and (4) the kinetics governing the treatment process.

Loading Rate

A loading parameter that has been developed over the years is the ***hydraulic retention time*** (HRT), θ, d

$$\theta = \frac{V}{Q}$$

V= volume of aeration 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 is ***specific substrate utilization rate***, q, per day.

$$q = \frac{Q\,(S_O - S_e)}{V\ X}$$

A similar loading parameter is ***mean cell residence time*** or ***sludge retention time*** (SRT), θ_c, d

$$\theta_c = \frac{V\,X}{Q_w X_r + (Q - Q_w X_e)}$$

where S_O 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 aeration tank, 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

$$Q_w X_r = YQ\,(S_O - S_e) - k_d\,XV$$

where Y= maximum yield coefficient (microbial mass synthesized / mass of substrate utilized) and k_d = endogenous decay rate (d^{-1}) .

From the above equation it is seen that $1/\theta_c = Yq - k_d$

If the value of S_e is small as compared S_O, q may also be expressed as ***Food to Microorganism ratio***, F/M

$$F/M = Q(S_O - S_e) / XV = QS_O / XV$$

The θ_c value adopted for design controls the effluent quality, and settleability and drainability of biomass, oxygen requirement and quantity of waste activated sludge.

Flow Scheme

The flow scheme involves:

- the pattern of sewage addition
- the pattern of sludge return to the aeration tank and
- the pattern of aeration.

Sewage addition may be at a single point at the inlet end or it may be at several points along the aeration tank. The sludge return may be directly from the settling tank to the aeration tank or through a sludge reaeration tank. Aeration may be at a uniform rate or it may be varied from the head of the aeration tank to its end.

Conventional System and its Modifications

The conventional system maintains a plug flow hydraulic regime. Over the years, several modifications to the conventional system have been developed to meet specific treatment objectives. In ***step aeration*** settled sewage is introduced at several points along the tank length which produces more uniform oxygen demand throughout. ***Tapered aeration*** attempts to supply air to match oxygen demand along the length of the tank. ***Contact stabilization*** provides for reaeration of return activated sludge from from the final clarifier, which allows a smaller aeration or contact tank. ***Completely mixed*** process aims at instantaneous mixing of the influent waste and return sludge with the entire contents of the aeration tank. Extended aeration process operates at a low organic load producing lesser quantity of well stabilized sludge.

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.

Aeration Tank

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

$$VX = \frac{YQqc(SO - S)}{1+ kdqc}$$

Alternately, the tank capacity may be designed from

$$F/M = QSO / XV$$

Hence, the **first step** in designing is to 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 characteristics of the sludge and the nitrification required. The choice generally lies between 5 days in warmer climates to 10 days in temperate ones where nitrification is desired alongwith good BOD removal, and complete mixing systems are employed.

The **second step** is to select two interrelated parameters **HRT, t and MLSS concentration**. It is seen that economy in reactor volume can be achieved by assuming a large value of X. However, it is seldom taken to be more than 5000 g/m3. For typical domestic sewage, the MLSS value of 2000-3000 mg/l if conventional plug flow type aeration system is provided, or 3000-5000 mg/l for completely mixed types. Considerations which govern the upper limit are: initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitations of oxygen transfer equipment to supply oxygen at required rate in small reactor volume, increased solids loading on secondary clarifier which may necessitate a larger surface area, design criteria for the tank and minimum HRT for the aeration tank.

The **length** of the tank depends upon the type of activated sludge plant. Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. The **width** and **depth** of the aeration tank depends on the type of aeration equipment employed. The depth control the aeration efficiency and usually ranges from 3 to 4.5 m. The width controls the mixing and is usually kept between 5 to 10 m. **Width-depth ratio** should be adjusted to be between 1.2 to 2.2. The length should not be less than 30 or not ordinarily longer than 100 m.

Oxygen Requirements

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

$$O2 \text{ required (g/d)} = \frac{Q(SO - S)}{f} - 1.42\, QwXr$$

where, f = ratio of BOD5 to ultimate BOD and 1.42 = oxygen demand of biomass (g/g)

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

Aeration Facilities

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

Secondary Settling

Secondary settling tanks, which receive the biologically treated flow undergo zone or compression settling. **Zone settling** occurs beyond a certain concentration when the particles are close enough together that interparticulate forces may hold the particles fixed relative to one another so that the whole mass tends to settle as a 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 whose results can be plotted as shown in figure.

Compression settling may occur at the bottom of a tank if particles are in such a concentration as to be in physical contact with one another. The weight of particles is partly supported by the lower layers of particles, leading to progressively greater compression with depth and thickening of sludge. From the settling column test, the limiting solids flux required to reach any desired underflow concentration can be estimated, from which the rquired tank area can be computed.

The solids load on the clarifier is estimated in terms of (Q+R)X, while the overflow rate or surface loading is estimated in terms of flow Q only (not Q+R) since the quantity R is withdrawn from the bottom and does not contribute to the overflow from the tank. The secondary settling tank is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. Beyond an MLSS concentration of 2000 mg/l the clarifier design is often controlled by the solids loading rate rather than the overflow rate. Recommended design values for treating domestic sewage in final clarifiers and mechanical thickeners (which also fall in this category of compression settling) are given in lecture 22.

Sludge Recycle

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settleability and thickening in the secondary sedimentation tank.

$$\frac{Q_r}{Q} = \frac{X}{X_r - X}$$

where Q_r = Sludge recirculation rate, m^3/d

The sludge settleability is determined by sludge volume index (SVI) defined as volume occupied in mL by one gram of solids in the mixed liquor after settling for 30 min. If it is assumed that sedimentation of suspended solids in the laboratory is similar to that in sedimentation tank, then $X_r = 10^6/SVI$. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids. The X_r value may not be taken more than 10,000 g/m^3 unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value.

Excess Sludge Wasting

The sludge in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature. Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter is preferred as the sludge concentration is fairly steady in that case. The excess sludge generated under steady state operation may be estimated by

$$\theta_c = \frac{VX}{Q_w X_r}$$

or $Q_w X_r = YQ(S_O - S) - k_d XV$

Design of Completely Mixed Activated Sludge System

Design a completely mixed activated sludge system to serve 60000 people that will give a final effluent that is nitrified and has 5-day BOD not exceeding 25 mg/l. The following design data is available.

Sewage flow = 150 l/person-day = 9000 m^3/day
BOD_5 = 54 g/person-day = 360 mg/l ; BOD_u = 1.47 BOD_5
Total kjeldahl nitrogen (TKN) = 8 g/person-day = 53 mg/l
Phosphorus = 2 g/person-day = 13.3 mg/l
Winter temperature in aeration tank = 18°C
Yield coefficient Y = 0.6 ; Decay constant K_d = 0.07 per day ; Specific substrate utilization rate = $(0.038\ \text{mg/l})^{-1}\ (\text{h})^{-1}$ at 18°C
Assume 30% raw BOD_5 is removed in primary sedimentation, and BOD_5 going to aeration is, therefore, 252 mg/l (0.7 x 360 mg/l).

Design:

(a) *Selection of θ_c, t and MLSS concentration*:

Considering the operating temperature and the desire to have nitrification and good sludge settling characteristics, adopt θ_c = 5d. As there is no special fear of toxic inflows, the HRT, t may be kept between 3-4 h, and MLSS = 4000 mg/l.

(b) *Effluent BOD_5*:

Substrate concentration, $S = \frac{1}{qY}(1/\theta_c + k_d) = \frac{1}{(0.038)(0.6)}(1/5 + 0.07)$

S = 12 mg/l.
Assume suspended solids (SS) in effluent = 20 mg/l and VSS/SS =0.8.
If degradable fraction of volatile suspended solids (VSS) =0.7 (check later), BOD_5 of VSS in effluent = 0.7(0.8x20) = 11mg/l.

Thus, total effluent BOD_5 = 12 + 11 = 23 mg/l (acceptable).

(c) *Aeration Tank*:

$VX = \frac{YQ\theta_c(S_O - S)}{1 + k_d\theta_c}$ where X = 0.8(4000) = 3200 mg/l

or $3200\ V = \frac{(0.6)(5)(9000)(252-12)}{[1 + (0.07)(5)]}$

$V = 1500\ m^3$

Detention time, $t = \frac{1500 \times 24}{9000} = 4h$

$$F/M = \frac{(252-12)(9000)}{(3200)\ (1500)} = 0.45 \text{ kg } BOD_5 \text{ per kg MLSS per day}$$

Let the aeration tank be in the form of four square shaped compartments operated in two parallel rows, each with two cells measuring 11m x 11m x 3.1m

(d) ***Return Sludge Pumping:***

If suspended solids concentration of return flow is 1% = 10,000 mg/l

$$R = \frac{MLSS}{(10000)-MLSS} = 0.67$$

$Q_r = 0.67 \times 9000 = 6000\ m^3/d$

(e) ***Surplus Sludge Production:***

$$\text{Net VSS produced } Q_wX_r = \frac{VX}{\theta_c} = \frac{(3200)(1500)}{(5)}(10^3/10^6) = 960 \text{ kg/d}$$

or SS produced =960/0.8 = 1200 kg/d

If SS are removed as underflow with solids concentration 1% and assuming specific gravity of sludge as 1.0,

Liquid sludge to be removed = 1200 x 100/1 = 120,000 kg/d
= 120 m^3/d

(f) ***Oxygen Requirement:***

1. For carbonaceous demand,
   oxygen required = (BOD_u removed) - (BOD_u of solids leaving)
   = 1.47 (2160 kg/d) - 1.42 (960 kg/d)
   = 72.5 kg/h
2. For nitrification,
   oxygen required = 4.33 (TKN oxidized, kg/d)
   Incoming TKN at 8.0 g/ person-day = 480 kg/day. Assume 30% is removed in primary sedimentation and the balance 336 kg/day is oxidized to nitrates. Thus, oxygen required
   = 4.33 x 336 = 1455 kg/day = 60.6 kg/h
3. Total oxygen required
   = 72.5 + 60.6 = 133 kg/h = 1.0 kg/kg of BOD_u removed.

Oxygen uptake rate per unit tank volume = 133/1500
= 90.6 mg/h/l tank volume

(g) ***Power Requirement:***

Assume oxygenation capacity of aerators at field conditions is only 70% of the capacity at standard conditions and mechanical aerators are capable of giving 2 kg oyxgen per kWh at standard conditions.

$$\text{Power required} = \frac{136}{0.7 \times 2} = 97 \text{ kW (130 hp)}$$

= (97 x 24 x 365) / 60,000 = 14.2 kWh/year/person

Theory of Aeration

Aeration is a gas-liquid mass transfer process in which the driving force in the liquid phase is the concentration gradient (C_s - C) for slightly soluble gases.

Mass transfer per unit time $= K_L.a\,(C_s - C)$

where, K_L = Liquid film coefficient

$= \dfrac{\text{Diffusion coefficient of liquid (D)}}{\text{Thickness of film (Y)}}$

a = Interficial area per unit volume

C_s = saturation concentration at the gas-liquid interface and C = some lower value in the body of the liquid.

The value of a increases as finer and finer droplets are formed, thus increasing the gas transfer. However, in practice, it is not possible to measure this area and hence the overall coefficient (K_L.a) per unit time, is determined by experimentation.

Adjustment for Field Conditions

The oxygen transfer capacity under field conditions can be calculated from the standard oxygen transfer capacity by the formula:

$$N = [N_s(C_s - C_L) \times 1.024^{T-20}\alpha\,]/9.2$$

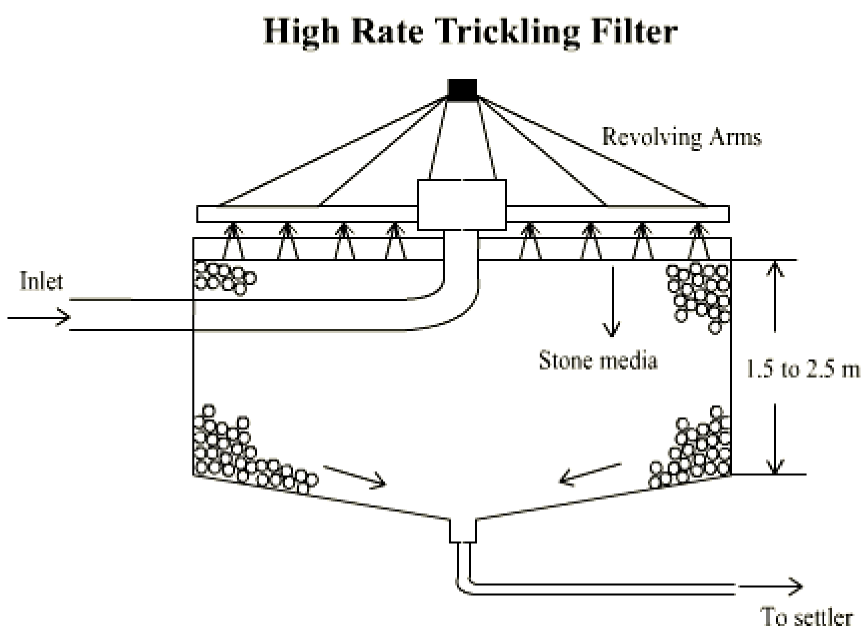
where,
N = oxygen transferred under field conditions, kg O_2/h.
N_s = oxygen transfer capacity under standard conditions, kg O_2/h.
C_s = DO saturation value for sewage at operating temperature.
C_L = operating DO level in aeration tank usually 1 to 2 mg/L.
T = Temperature, degree C.
α = Correction factor for oxygen transfer for sewage, usually 0.8 to 0.85.

Aeration Facilities

- Oxygen may be supplied either by surface aerators or diffused aerators employing fine or coarse diffusers.
- The aeration devices apart from supplying the required oxygen shall also provide adequate mixing in order that the entire MLSS present in the aeration tank will be available for biological activity.
- Aerators are rated based on the amount of oxygen they can transfer to tap water under standard conditions of 20°C, 760 mm Hg barometric pressure and zero DO.

Trickling Filters

Trickling filter is an ***attached growth process*** i.e. process in which microorganisms responsible for treatment are attached to an inert packing material. Packing material used in attached growth processes include rock, gravel, slag, sand, redwood, and a wide range of plastic and other synthetic materials.



Process Description

- The wastewater in trickling filter is distributed over the top area of a vessel containing non-submerged packing material.
- 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 as the substrate is metabolised before it can reach the micro-organisms near the medium face as a result of increased 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***.
- The sloughed off film and treated wastewater are collected by an underdrainage which also allows circulation of air through filter. The collected liquid is passed to a settling tank used for solid- liquid separation.

Types of Filters

Trickling filters are classified as high rate or low rate, based on the organic and hydraulic loading applied to the unit.

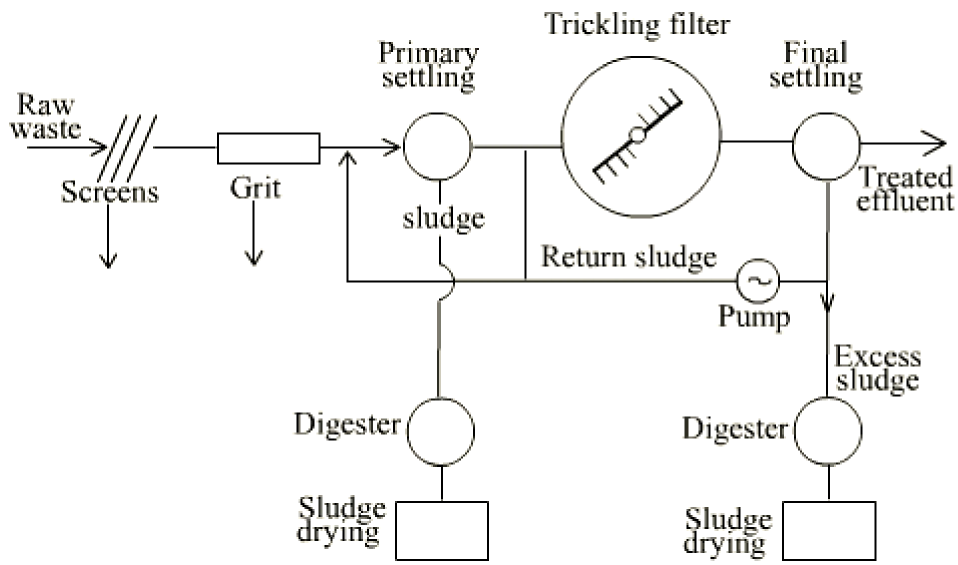
S.No.	Design Feature	Low Rate Filter	High Rate Filter
1.	Hydraulic loading, $m^3/m^2.d$	1 - 4	10 - 40
2.	Organic loading,kg BOD / $m^3.d$	0.08 - 0.32	0.32 - 1.0
3.	Depth, m.	1.8 - 3.0	0.9 - 2.5
4.	Recirculation ratio	0	0.5 - 3.0 (domestic wastewater) upto 8 for strong industrial wastewater.

- The hydraulic loading rate is the total flow including recirculation appied on unit area of the filter in a day, while the organic loading rate is the 5 day 20°C BOD, excluding the BOD of the recirculant, applied per unit volume in a day.
- Recirculation is generally not adopted in low rate filters.
- A well operated low rate trickling filter in combination with secondary settling tank may remove 75 to 90% BOD and produce highly nitrified effluent. It is suitable for treatment of low to medium strength domestic wastewaters.

- The high rate trickling filter, single stage or two stage are recommended for medium to relatively high strength domestic and industrial wastewater. The BOD removal efficiency is around 75 to 90% but the effluent is only partially nitrified.
- Single stage unit consists of a primary settling tank, filter, secondary settling tank and facilities for recirculation of the effluent. Two stage filters consist of two filters in series with a primary settling tank, an intermediate settling tank which may be omitted in certain cases and a final settling tank.

Process Design

Flow sheet of a trickling filter system



Generally trickling filter design is based on empirical relationships to find the required filter volume for a designed degree of wastewater treatment. Types of equations:

1. NRC equations (National Research Council of USA)
2. Rankins equation
3. Eckenfilder equation
4. Galler and Gotaas equation

NRC and Rankin's equations are commonly used. NRC equations give satisfactory values when there is no re-circulation, the seasonal variations in temperature are not large and fluctuations with high organic loading. Rankin's equation is used for high rate filters.

NRC equations: These equations are applicable to both low rate and high rate filters. The efficiency of single stage or first stage of two stage filters, E_2 is given by

$$E_2 = \frac{100}{1+0.44(F_{1.BOD}/V_1.Rf_1)^{1/2}}$$

For the second stage filter, the efficiency E_3 is given by

$$E_3 = \frac{100}{[(1+0.44)/(1-E_2)](F_{2.BOD}/V_2.Rf_2)^{1/2}}$$

where E_2= % efficiency in BOD removal of single stage or first stage of two-stage filter, E_3=% efficiency of second stage filter, $F_{1.BOD}$= BOD loading of settled raw sewage in single stage of the two-stage filter in kg/d, $F_{2.BOD}$= $F_{1.BOD}(1- E_2)$= BOD loading on second-stage filter in kg/d, V_1= volume of first stage filter, m^3; V_2= volume of second stage filter, m^3; Rf_1= Recirculation factor for first stage, R_1= Recirculation ratio for first stage filter, Rf_2= Recirculation factor for second stage, R_2= Recirculation ratio for second stage filter.

Rankins equation: This equation also known as Tentative Method of Ten States USA has been successfully used over wide range of temperature. It requires following conditions to be observed for single stage filters:

1. Raw settled domestic sewage BOD applied to filters should not exceed 1.2 kg BOD_5/day/ m^3 filter volume.
2. Hydraulic load (including recirculation) should not exceed 30 m^3/m^2 filter surface-day.
3. Recirculation ratio (R/Q) should be such that BOD entering filter (including recirculation) is not more than three times the BOD expected in effluent. This implies that as long as the above conditions are satisfied efficiency is only a function of recirculation and is given by:

$$E = \frac{(R/Q) + 1}{(R/Q) + 1.5}$$

Other Aerobic Treatment Units

1. ***Stabilization ponds***: The stabilization ponds are open flow through basins specifically designed and constructed to treat sewage and biodegradable industrial wastes. They provide long detention periods extending from a few to several days.
2. ***Aerated lagoons***: Pond systems, in which oxygen is provided through mechanical aeration rather than algal photosynthesis are called aerated lagoons.
3. ***Oxidation ditch***: The oxidation ditch is a modified form of "extended aeration" of activated sludge process. The ditch consists of a long continuous channel oval in shape with two surface rotors placed across the channel.

Anaerobic Treatment

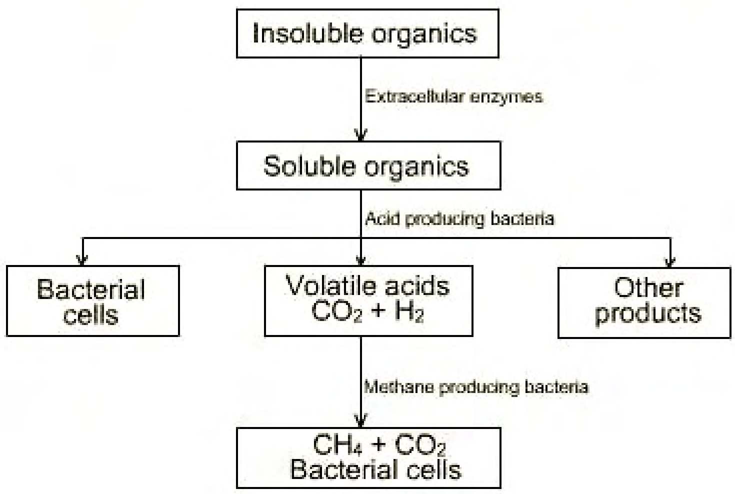
The anaerobic waste treatment process is an effective method for the treatment of many organic wastes. The treatment has a number of advantages over aerobic treatment process, namely,

- the energy input of the system is low as no energy is requred for oxygenation,
- lower production of excess sludge(biological synthesis) per unit mass of substrate utilized,
- lower nutrient requirement due to lower biological synthesis, and
- degradation leads to production of biogas which is a valuable source of energy.

Fundamental Microbiology

The anaerobic treatment of organic wastes resulting in the production of carbon dioxide and methane, involves two distinct stages. In the first 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, β-oxidations, and other metabolic processes leading to the formation of simple organic compounds, mainly short-chain (volatile) acids and alcohols. The first stage is commonly referred to as "***acid fermentation***". However in the second 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. This stage is generally referred to as "***methane fermentation***".

Sequential Mechanism of Anaerobic Waste Treatment



The primary acids produced during acid fermentation are propionic and acetic acid. It is reported that only one group of methane bacteria is necessary for methane fermentation of acetic acid, whereas propionic acid, which is fermented through acetic acid requires two different groups of methane bacteria. The methane fermentation reactions for these two acids are:

Acetic acid:

$$CH_3COOH \longrightarrow CH_4 + CO_2$$

Propionic acid:

Ist Step:

$$CH_3CH_2COOH + 0.5\ H_2O \longrightarrow CH_3COOH + 0.75CH_4 + 0.25CO_2$$

IInd Step:

$$CH_3COOH \longrightarrow CH_4 + CO_2$$

Overall:

$$CH_3CH_2COOH + 0.5\ H_2O \longrightarrow 1.75CH_4 + 1.25CO_2$$

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. As a result, methane fermentation is generally assumed to be the rate limiting step in anaerobic wastewater treatment.

Anaerobic Reactor

Various types of anaerobic units that have been developed are as follows:

- **Upflow anaerobic filters** packed with either pebbles, stones, PVC sheets, etc. as media to support submerged biological growths (fixed film). The units are reported to work well but a likely problem is accumulation of solids in the interstices.
- **Downflow anaerobic filters** packed with similar media as above but not to be confused with usual trickling filters which are aerobic. In the anaerobic units, the inlet and outlet are so placed that the media and fixed film stay submerged.
- **UASB type units** in which no special media have to be used since the sludge granules themselves act as the 'media' and stay in suspension. These are commonly preffered.
- **Fluidized bed units** filled with sand or plastic granules are used with recirculation under required pressure to keep the entire mass fluidized and the sludge distributed over the entire reactor volume. Their power consumption is higher.

UASB Units

UASB type units are one in which no special media have to be used since the sludge granules themselves act as the 'media' and stay in suspension. UASB system is not patented. A typical arrangement of a UASB type treatment plant for municipal sewage would be as follows:

1. Initial pumping
2. Screening and degritting
3. Main UASB reactor
4. Gas collection and conversion or conveyance
5. Sludge drying bed
6. Post treatment facility

In the UASB process, the whole waste is passed through the anaerobic reactor in an upflow mode, with a hydraulic retention time (HRT) of only about 8-10 hours at average flow. No prior sedimentation is required. The anaerobic unit does not need to be filled with stones or any other media; the upflowing sewage itself forms millions of small "granules" or particles of sludge which are held in suspension and provide a large surface area on which organic matter can attach and undergo biodegradation. A high solid retention time (SRT) of 30-50 or more days occurs within the unit. No mixers or aerators are required. The gas produced can be collected and used if desired. Anaerobic systems function satisfactorily when temperatures inside the reactor are above 18-20°C. Excess sludge is removed from time to time through a separate pipe and sent to a simple sand bed for drying.

Design Approach

Size of Reactor: Generally, UASBs are considered where temperature in the reactors will be above 20°C. At equilibrium condition, sludge withdrawn has to be equal to sludge produced daily. The sludge produced daily depends on the characteristics of the raw wastewater since it is the sum total of (i) the new VSS produced as a result of BOD removal, the yield coefficient being assumed as 0.1 g VSS/ g BOD removed, (ii) the non-degradable residue of the VSS coming in the inflow assuming 40% of the VSS are degraded and residue is 60%, and (iii) Ash received in the inflow, namely TSS-VSS mg/l. Thus, at steady state conditions,

$$\text{SRT} = \frac{\text{Total sludge present in reactor, kg}}{\text{Sludge withdrawn per day, kg/d}}$$

$$= 30 \text{ to } 50 \text{ days.}$$

Another parameter is HRT which is given by:

$$\text{HRT} = \frac{\text{Reactor volume, m}^3}{\text{Flow rate, m}^3\text{/h}}$$

$$= 8 \text{ to } 10 \text{ h or more at average flow.}$$

The reactor volume has to be so chosen that the desired SRT value is achieved. This is done by solving for HRT from SRT equation assuming (i) depth of reactor (ii) the effective depth of the sludge blanket, and (iii) the average concentration of sludge in the blanket (70 kg/m^3). The full depth of the reactor for treating low BOD municipal sewage is often 4.5 to 5.0 m of which the sludge blanket itself may be 2.0 to 2.5 m depth. For high BOD wastes, the depth of both the sludge blanket and the reactor may have to be increased so that the organic loading on solids may be kept within the prescribed range.

Once the size of the reactor is fixed, the upflow velocity can be determined from

$$\text{Upflow velocity m/h} = \frac{\text{Reactor height}}{\text{HRT, h}}$$

Using average flow rate one gets the average HRT while the peak flow rate gives the minimum HRT at which minimum exposure to treatment occurs. In order to retain any flocculent sludge in reactor at all times, experience has shown that the upflow velocity should not be more than 0.5 m/h at average flow and not more than 1.2 m/h at peak flow. At higher velocities, carry over of solids might occur and effluent quality may be deteriorated. The feed inlet system is next designed so that the required length and width of the UASB reactor are determined.

The settling compartment is formed by the sloping hoods for gas collection. The depth of the compartment is 2.0 to 2.5 m and the surface overflow rate kept at 20 to 28 m^3/m^2-day (1 to 1.2 m/h) at peak flow. The flow velocity through the aperture connecting the reaction zone with the settling compartment is limited to not more than 5 m/h at peak flow. Due attention has to be paid to the geometry of the unit and to its hydraulics to ensure proper working of the "Gas-Liquid-Solid-Separator (GLSS)" the gas collection hood, the incoming flow distribution to get spatial uniformity and the outflowing effluent.

Physical Parameters

A single module can handle 10 to 15 MLD of sewage. For large flows a number of modules could be provided. Some physical details of a typical UASB reactor module are given below:

Reactor configuration	Rectangular or circular. Rectangular shape is preferred
Depth	4.5 to 5.0 m for sewage.
Width or diameter	To limit lengths of inlet laterals to around 10-12 m for facilitating uniform flow distribution and sludge withdrawal.
Length	As necessary.
Inlet feed	gravity feed from top (preferred for municipal sewage) or pumped feed from bottom through manifold and laterals (preferred in case of soluble industrial wastewaters).
Sludge blanket depth	2 to 2.5 m for sewage. More depth is needed for stronger wastes.
Deflector/GLSS	This is a deflector beam which together with the gas hood (slope 60) forms a "gas-liquid-solid-separator" (GLSS) letting the gas go to the gas collection channel at top, while the liquid rises into the settler compartment and the sludge solids fall back into the sludge compartment. The flow velocity through the aperture connecting the reaction zone with the settling compartmentt is generally limited to about 5m/h at peak flow.
Settler compartment	2.0-2.5 m in depth. Surface overflow rate equals 20-28 $m^3/m^2/d$ at peak flow.

Process Design Parameters

A few process design parameters for UASBs are listed below for municipal sewages with BOD about 200-300 mg/l and temperatures above 20°C.

HRT	8-10 hours at average flow (minimum 4 hours at peak flow)
SRT	30-50 days or more
Sludge blanket concentration (average)	15-30 kg VSS per m^3. About 70 kg TSS per m^3.
Organic loading on sludge blanket	0.3-1.0 kg COD/kg VSS day (even upto 10 kg COD/ kg VSS day for agro-industrial wastes).
Volumetric organic loading	1-3 kg COD/m^3 day for domestic sewage (10-15 kg COD/m^3 day for agro-industrial wastes)
BOD/COD removal efficiency	Sewage 75-85% for BOD. 74-78% for COD.
Inlet points	Minimum 1 point per 3.7-4.0 m^2 floor area.
Flow regime	Either constant rate for pumped inflows or typically fluctuating flows for gravity systems.
Upflow velocity	About 0.5 m/h at average flow, or 1.2 m/h at peak flow, whichever is low.
Sludge production	0.15-0.25 kg TS per m^3 sewage treated.
Sludge drying time	Seven days (in India)
Gas production	Theoretical 0.38 m^3/kg COD removed. Actual 0.1-0.3 m^3 per kg COD removed.
Gas utilization	Method of use is optional. 1 m^3 biogas with 75% methane content is equivalent to 1.4 kWh electricity.
Nutrients nitrogen and phosphorus removal	5 to 10% only.

Nitrification-Denitrification Systems

A certain amount of nitrogen removal (20-30%) occurs in conventional activated sludge systems. Nitrogen removal ranging from 70 to 90 % can be obtained by use of nitrification-denitrification method in plants based on activated sludge and other suspended growth systems. Biological denitrification requires prior nitrification of all ammonia and organic nitrogen in the incoming waste.

Nitrification

There are two groups of chemoautotrophic bacteria that can be associated with the process of nitrification. One group (*Nitrosomonas*) derives its energy through the oxidation of ammonium to nitrite, whereas the other group (*Nitrobacter*) obtains energy through the oxidation of nitrite to nitrate. Both the groups, collectively called *Nitrifiers*, obtain carbon required, from inorganic carbon forms. Nitrification of ammonia to nitrate is a two step process:

$$NH_3 \longrightarrow NH_4 \xrightarrow{\text{Nitrosomonas}} NO_2 \xrightarrow{\text{Nitrobacter}} NO_3$$

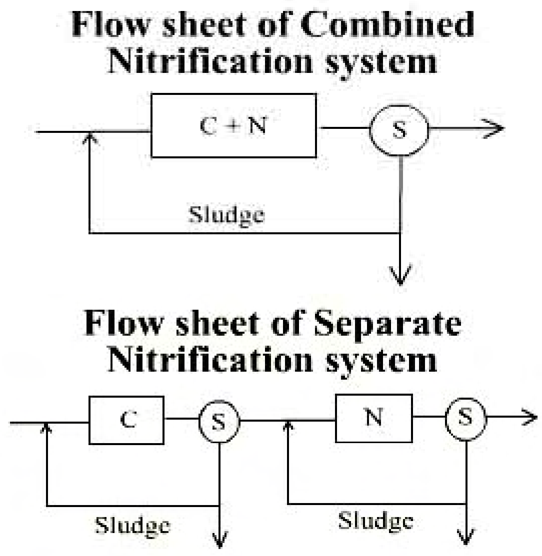
Stoichiometrically, 4.6 kg of oxygen is required for nitrifying 1 kg of nitrogen. Under steady state conditions, experimental evidence has shown nitrite accumulation to be insignificant. This suggests that the rate-limiting step for the conversion of ammonium to nitrate is the oxidation of ammonium to nitrite by the genus Nitrosomonas.

$$\theta_c = \frac{1}{\mu}$$

where μ is the growth rate of nitrosomonas at the worst operating temperature. Sludge age (or mean cell residence time), θ_c in a treatment plant must be sufficiently high if nitrification is desired.

Combined and Separate Systems of Biological Oxidation & Nitrification

Following figure shows flow sheets for combined and separate systems for biological oxidation and nitrification.



Combined system is favoured method of operation as it is less sensitive to load variations - owing to larger sized aeration tank - generally produces a smaller volume of surplus sludge owing to higher values of θ_c adopted, and better sludge settleability.

Care should be taken to ensure that the oxygenation capacity of aeration tank is sufficient to meet oxygen uptake due to carbonaceous demand and nitrification. Recycling of sludge must be rapid enough to prevent denitrification (and rising sludge) owing to anoxic conditions in the settling tank.

In ***separate system***, the first tank can be smaller in size since a higher F/M ratio can be used, but this makes the system somewhat more sensitive to load variations and also tends to produce more sludge for disposal. An additional settling tank is also necessary between the two aeration tanks to keep the two sludges separate. A principal advantage of this system is its higher efficiency of nitrification and its better performance when toxic substances are feared to be in the inflow.

Biological Denitrification

When a treatment plant discharges into receivingstream with lowavailable nitrogen concentration and with a flow much larger than the effluent, the presence of nitrate in the effluent generally does not adversely affect stream quality. However,if the nitrate concentration in the stream is significant, it may be desirable to control the nitrogen contentof the effluent, as highly nitrified effluents can stillaccelarate algal blooms. Even more critical is the case where treatment plant effluent is dischargeddirectly into relatively still bodies of water such as lakes or reservoirs. Another argument for the controlof nitrogen in the aquatic environment is theoccurence of infantile methemoglobinemia,which results from high concentration of nitrates indrinking water.

The four basic processes that are used are: (1) ammonia stripping,
(2) selective ion exchange, (3) break point chlorination, and
(4) biological nitrification/denitrification.

Biological nitrification/denitrification is a two step process. The first step is nitrification, which is conversion of ammonia to nitrate through the action of nitrifying bacteria. The second step is nitrate conversion (denitrification), which is carried out by facultative heterotrophic bacteria under anoxic conditions.

Microbiological Aspects of Denitrification

- Nitrate conversion takes place through both assimilatory and dissimilatory cellular functions. In ***assimilatory denitrification***, nitrate is reduced to ammonia, which then serves as a nitrogen source for cell synthesis. Thus, nitrogen is removed from the liquid stream by incorporating it into cytoplasmic material.
- In ***dissimilatory denitrification***, nitrate serves as the electron acceptor in energy metabolism and is converted to various gaseous end products but principally molecular nitrogen, N_2, which is then stripped from the liquid stream.
- Because the microbial yield under anoxic conditions is considerably lower than under aerobic conditions, a relatively small fraction of the nitrogen is removed through assimilation. Dissimilatory denitrification is, therfore, the primary means by which nitrogen removal is achieved.
- A carbon source is also essential as electron donor for denitrification to take place. This source may be in the form of carbon internally available in sewage or artificially added (eg. as methanol). Since most community wastewaters have a higher ratio of BOD:N, the internally available carbon becomes attractive and economical for denitrification.

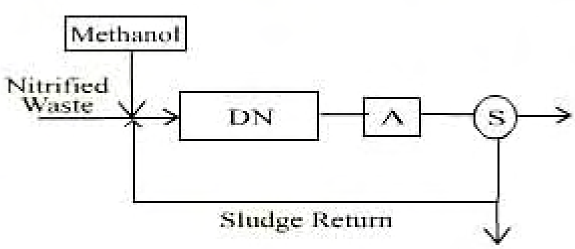
  Denitrification releases nitrogen which escapes as an inert gas to the atmosphere while oxygen released stays dissolved in the liquid and thus reduces the oxygen input needed into the system. Each molecule of nitrogen needs 4 molecules of oxygen during nitrification but releases back 2.5 molecules in denitrification. Thus, theoretically, 62.5% of the oxygen used is released back in denitrification.

Typical Flowsheets for Denitrification

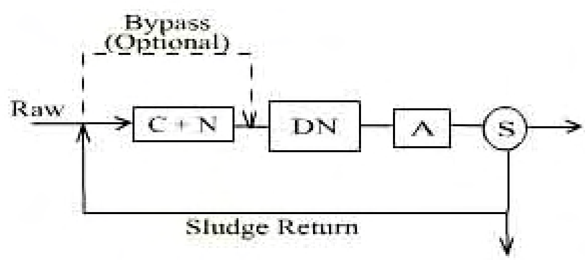
Denitrification in suspended growth systems can be achieved using anyone of the typical flowsheets shown in the figure.

- The use of methanol or any other artificial carbon source should be avoided as far as possible since it adds to the cost of treatment and also some operating difficulties may arise fro dosing rate of methanol. Too much would introduce an unnecessary BOD in the effluent while too little would leave some nitrates undernitrified.
- A more satisfactory arrangement would be to use the carbon contained in the waste itself. However, the anoxic tank has to be of sufficient detention time for denitrification to occur which, has a slower rate; since the corresponding oxygen uptake rate of the mixed liquor is mainly due to endogenous respiration and is thus low. The denitrification rate, therefore, in a way also depends on the F/M ratio in the prior aeration tank.
- Consequently, if desired, a portion of the raw waste may be bypassed to enter directly into the anoxic tank and thus contribute to an increased respiration rate. This reduces the sizes of both the anoxic and aeration tanks, but the denitrification efficiency is reduced as the bypassed unnitrified ammonia can not be denitrified.
- By reversing the relative positions of anoxic and aerobic tanks, the oxygen requirement of the waste in its anoxic state is met by the release of oxygen from nitrates in the recycled flow taken from the end of nitrification tank. Primary settling of the raw waste may be omitted so as to bring more carbon into the anoxic tank.
- More complete nitrification-denitrification can be achieved by Bardenpho arrangement. The first anoxic tank has the advantage of higher denitrification rate while the nitrates remaining in the liquor passing out of the tank can be denitrified further in a second anoxic tank through endogenous respiration.
- The flow from anoxic tank is desirable to reaerate for 10-15 minutes to drive off nitrogen gas bubbles and add oxygen prior to sedimentation.

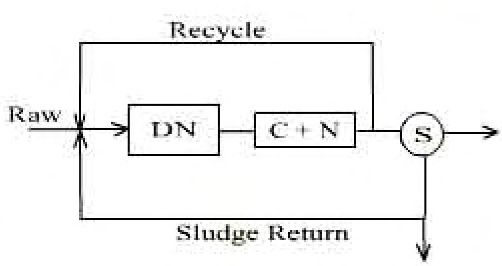
Flow sheet for Separate Denitrification of Nitrified Wastewater Using Methanol



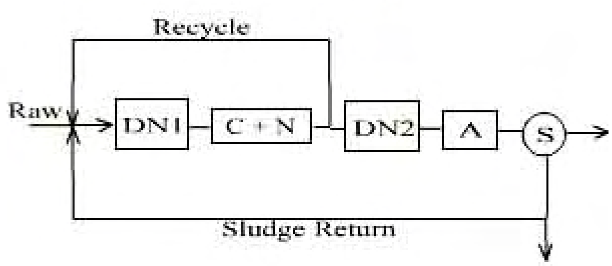
Separate Denitrification of Nitrified Wastewater Using Methanol

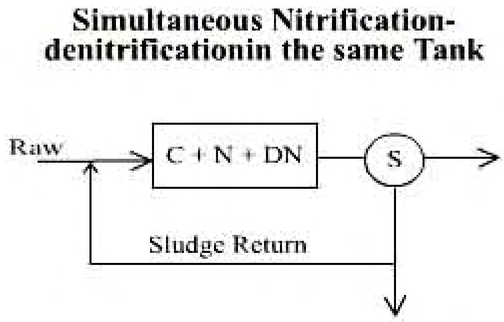


Pre-denitrification with Recycle of Nitrified Effluent to Anoxic Tank



"Bardenpho" Arrangement With two Anoxic Tanks to Give Higher Degree of Denitrification





Phosphorus Removal

Phosphorus precipitation is ususally achieved by addition of chemicals like calcium hydroxide, ferrous or ferric chloride, or alum, either in the primary or the final settling tank.

Alum is more expensive and generates more hydroxide, which creates extra sludge, that is difficult to dewater. Use of lime results in an increase of approximately 50% in surplus sludge, but the sludge is reported to have good dewatering properties. When using iron salts, a molar ratio of 1.0:1.4 of iron to phosphorus is reported to give 91-96% removal of total phosphorus using ferrous chloride dosed directly beneath the aerator.

Chemical addition prior to biological treatment is feasible if a primary settling tank exists as in the case of the conventional activated sludge process. The dose requirement then increases, but chemical precipitation also improves organic removal, thus reducing BOD load on the biological treatment. For extended aeration plants there is no primary settling; chemical addition has to be done in the final settling tank.

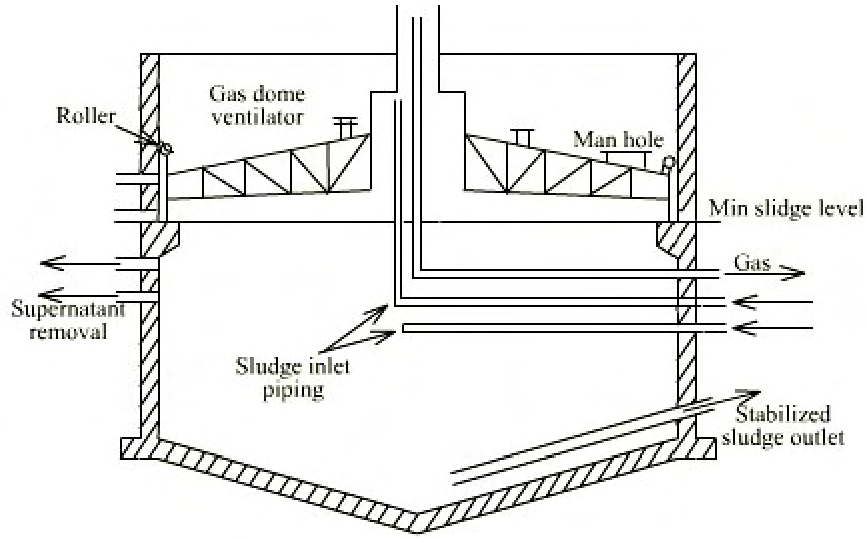
Sludge Digestion

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. In the first stage, referred to as "***acid fermentation***", 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, β-oxidations, and other metabolic processes leading to the formation of simple organic compounds, mainly short-chain (volatile) acids and alcohols. However in the second stage, referred to as "***methane fermentation***", the end products of the first stage are converted to gases (mainly methane and carbon dioxide) by several different species of strictly anaerobic bacteria.

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. If the pH drops below 6.0, methane formation essentially ceases, and more acid accumulates, thus bringing the digestion process to a standstill. As a result, methane fermentation is generally assumed to be the rate limiting step in anaerobic wastewater treatment. 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.

Digestion Tanks or Digesters

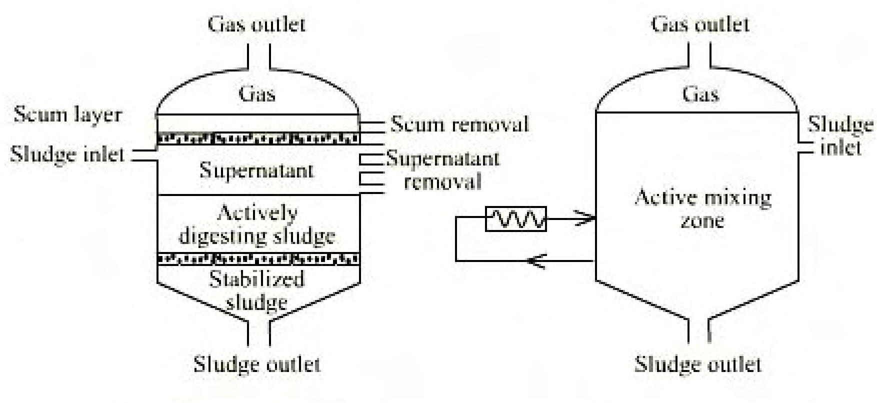
A sludge digestion tank is a RCC or steel tank of cylindrical shape with hopper bottom and is covered with fixed or floating type of roofs.



Anaerobic sludge digester

Types of Anaerobic Digesters

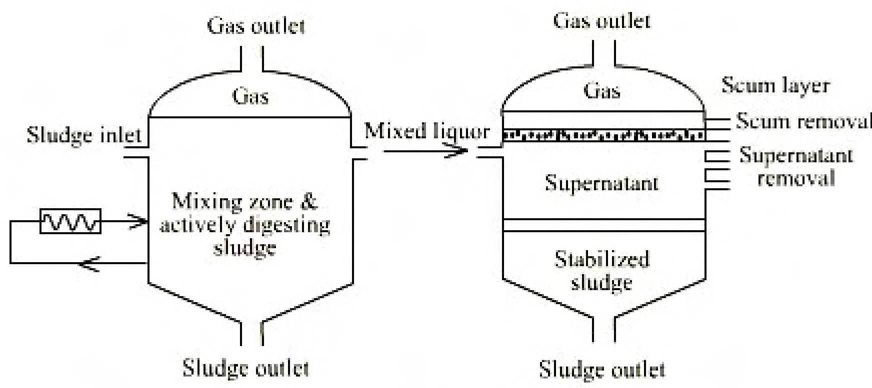
The anaerobic digesters are of two types: standard rate and high rate. In the standard rate digestion process, the digester contents are usually unheated and unmixed. The digestion period may vary from 30 to 60 d. In a high rate digestion process, the digester contents are heated and completely mixed. The required detention period is 10 to 20 d.



Standard-rate sludge digester

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

Design Details

Generally digesters are designed to treat for a capacity upto 4 MLD.

1. Tank sizes are not less than 6 m 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. The digester capacity may be determined from the relationship

   $$V = [V_f - 2/3\ (V_f - V_d)]t_1 + V_d t_2$$

   where V = capacity of digester in m^3, V_f = volume of fresh sludge m^3/d, V_d = volume of daily digested sludge accumulation in tank m^3/d, t_1= digestion time in days required for digestion, d, and t_2 = period of digested sludge storage.

Gas Collection

The amount of sludge gas produced varies from 0.014 to 0.028 m^3 per capita. The sludge gas is normally composed of 65% methane and 30% carbondioxide and remaining 5% of nitrogen and other inert gases, with a calorific value of 5400 to 5850 kcal/m^3.

Treatment Plant Layout and Siting

Plant layout is the arrangement of designed treatment units on the selected site. The components that need to be included in a treatment plant, should be so laid out as to optimize land requirement, minimize lengths of interconnecting pipes and pumping heads. Access for sludge and chemicals transporting, and for possible repairs, should be provided in the layout.

Siting is the selection of site for treatment plant based on features as character, topography, and shoreline. Site development should take the advantage of the existing site topography. The following principles are important to consider:

1. A site on a side-hill can facilitate gravity flow that will reduce pumping requirements and locate normal sequence of units without excessive excavation or fill.
2. When landscaping is utilized it should reflect the character of the surrounding area. Site development should alter existing naturally stabilized site contours and drainage as little as possible.
3. The developed site should be compatible with the existing land uses and the comprehensive development plan.

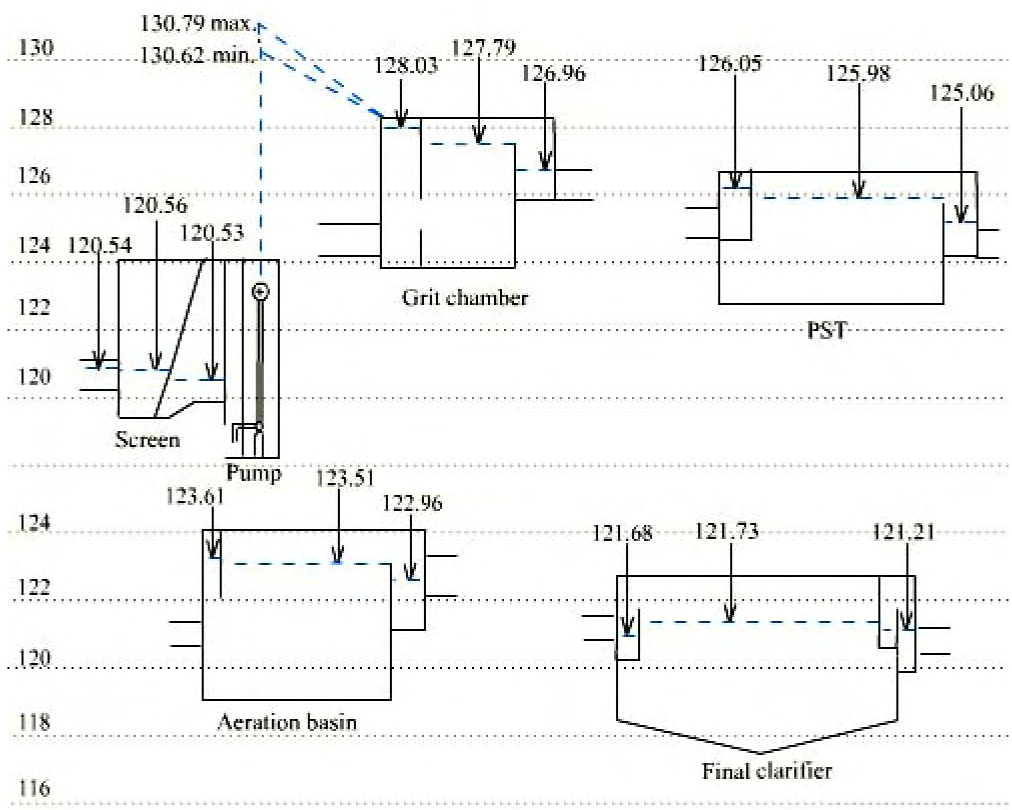
Treatment Plant Hydraulics

Hydraulic profile is the graphical representation of the hydraulic grade line through the treatment plant. If the high water level in the receiving water is known, this level is used as a control point, and the head loss computations are started backward through the plant. The ***total available head*** at the treatment plant is the difference in water surface elevations in the interceptor and the water surface elevation in the receiving water at high flood level. If the total available head is less than the head loss through the plant, flow by gravity cannot be achieved. In such cases pumping is needed to raise the head so that flow by gravity can occur.

There are many basic principles that must be considered when preparing the hydraulic profile through the plant. Some are listed below:

1. The hydraulic profiles are prepared at peak and average design flows and at minimum initial flow.
2. The hydraulic profile is generally prepared for all main paths of flow through the plant.
3. The head loss through the treatment plant is the sum of head losses in the treatment units and the connecting piping and appurtenances.
4. The head losses through the treatment unit include the following:
    a. Head losses at the influent structure.
    b. Head losses at the effluent structure.
    c. Head losses through the unit.
    d. Miscellaneous and free fall surface allowance.
5. The total loss through the connecting pipings, channels and appurtenances is the sum of following:
    a. Head loss due to entrance.
    b. Head loss due to exit.
    c. Head loss due to contraction and enlargement.
    d. Head loss due to friction.
    e. Head loss due to bends, fittings, gates, valves, and meters.
    f. Head required over weir and other hydraulic controls.
    g. Free-fall surface allowance.

Typical Hydraulic Profile Through Treatment Facility



Treated Effluent Disposal

The proper disposal of treatment plant effluent or reuse requirements is an essential part of planning and designing wastewater treatment facilities. Different methods of ultimate disposal of secondary effluents are discussed as follows.

Natural Evaporation

The process involves large impoundments with no discharge. Depending on the climatic conditions large impoundments may be necessary if precipitation exceeds evaporation. Therefore, considerations must be given to net evaporation, storage requirements, and possible percolation and groundwater pollution. This method is particularly beneficial where recovery of residues is desirable such as for disposal of brines.

Groundwater Recharge

Methods for groundwater recharge include rapid infiltration by effluent application or impoundment, intermittent percolation, and direct injection. In all cases risks for groundwater pollution exists. Furthermore, direct injection implies high costs of treating effluent and injection facilities.

Irrigation

Irrigation has been practiced primarily as a substitute for scarce natural waters or sparse rainfall in arid areas. In most cases food chain crops (i.e. crops consumed by humans and those animals whose products are consumed by humans) may not be irrigated by effluent. However, field crops such as cotton, sugar beets, and crops for seed production are grown with wastewater effluent.

Wastewater effluent has been used for watering parks, golf courses and highway medians.

Recreational Lakes

The effluent from the secondary treatment facility is stored in a lagoon for approximately 30 days. The effluent from the lagoon is chlorinated and then percolated through an area of sand and gravel, through which it travels for approximately 0.5 km and is collected in an interceptor trench. It is discharged into a series of lakes used for swimming, boating and fishing.

Aquaculture

Aquaculture, or the production of aquatic organisms (both flora and fauna), has been practiced for centuries primarily for production of food, fiber and fertilizer. Lagoons are used for aquaculture, although artificial and natural wetlands are also being considered. However, the uncontrolled spread of water hyacinths is itself a great concern because the flora can clog waterways and ruin water bodies.

Municipal Uses

Technology is now available to treat wastewater to the extent that it will meet drinking water quality standards. However, direct reuse of treated wastewater is practicable only on an emergency basis. Many natural bodies of water that are used for municipal water supply are also used for effluent disposal which is done to supplement the natural water resources by reusing the effluent many times before it finally flows to the sea.

Industrial Uses

Effluent has been successfully used as a cooling water or boiler feed water. Deciding factors for effluent reuse by the industry include (1) availability of natural water, (2) quality and quantity of effluent, and cost of processing, (3) pumping and transport cost of effluent, and (4) industrial process water that does not involve public health considerations.

Discharge into Natural Waters

Discharge into natural waters is the most common disposal practice. The self-purification or assimilative capacity of natural waters is thus utilized to provide the remaining treatment.

Stabilization Ponds

secondary effluents and removal of bacteria are called ***maturation ponds***.

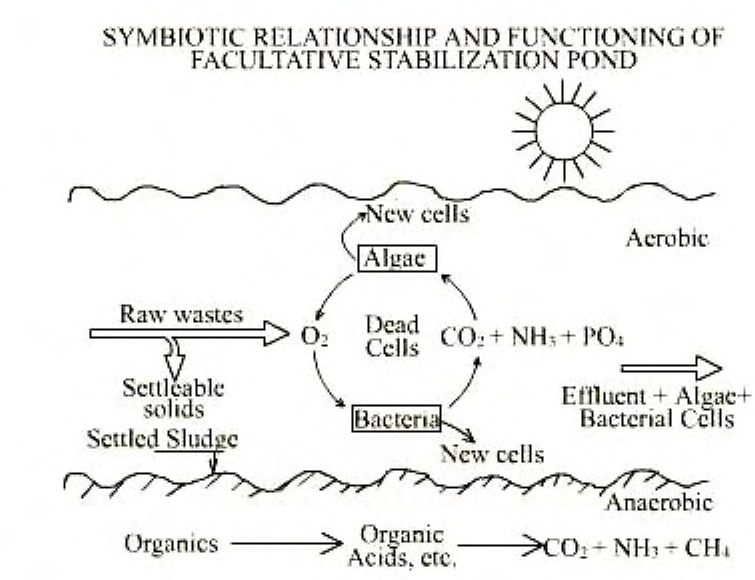
Classification of Stabilization Ponds

Stabilization ponds may be aerobic, anaerobic or facultative.

- ***Aerobic ponds*** are shallow ponds with depth less than 0.5 m and BOD loading of 40-120 kg/ha.d so as to maximize penetration of light throughout the liquid depth. Such ponds develop intense algal growth.
- ***Anaerobic ponds*** are used as pretreatment of high strength wastes with BOD load of 400-3000 kg/ha.d Such ponds are constructed with a depth of 2.5-5m as light penetration is unimportant.
- ***Facultative pond*** functions aerobically at the surface while anaerobic conditions prevail at the bottom.They are often about 1 to 2 m in depth. The aerobic layer acts as a good check against odour evolution from the pond.

Mechanism of Purification

The functioning of a facultative stabilization pond and symbiotic relationship in the pond are shown below. Sewage organics are stabilized by both aerobic and anaerobic reactions. In the top aerobic layer, where oxygen is supplied through algal photosynthesis, the non-settleable and dissolved organic matter is oxidized to CO_2 and water. In addition, some of the end products of partial anaerobic decomposition such as volatile acids and alcohols, which may permeate to upper layers are also oxidized periodically. The settled sludge mass originating from raw waste and microbial synthesis in the aerobic layer and dissolved and suspended organics in the lower layers undergo stabilization through conversion to methane which escapes the pond in form of bubbles.



Factors Affecting Pond Reactions

Various factors affect pond design:

- wastewater characteristics and fluctuations.
- environmental factors (solar radiation, light, temperature)
- algal growth patterns and their diurnal and seasonal variation)
- bacterial growth patterns and decay rates.
- solids settlement, gasification, upward diffusion, sludge accumulation.

The depth of aerobic layer in a facultative pond is a function of solar radiation, waste characteristics, loading and temperature. As the organic loading is increased, oxygen production by algae falls short of the oxygen requirement and the depth of aerobic layer decreases. Further, there is a decrease in the photosynthetic activity of algae because of greater turbidity and inhibitory effect of higher concentration of organic matter.

Gasification of organic matter to methane is carried out in distinct steps of acid production by acid forming bacteria and acid utilization by methane bacteria. If the second step does not proceed satisfactorily, there is an accumulation of organic acids resulting in decrease of pH which would result in complete inhibition of methane bacteria. Two possible reasons for imbalance between activities of methane bacteria are: (1) the waste may contain inhibitory substances which would retard the activity of methane bacteria and not affect the activity of acid producers to the same extent. (2) The activity of methane bacteria decreases much more rapidly with fall in temperature as compared to the acid formers.

Thus, year round warm temperature and sunshine provide an ideal environment for operation of facultative ponds.

Algal Growth and Oxygen Production

Algal growth converts solar energy to chemical energy in the organic form. Empirical studies have shown that generally about 6% of visible light energy can be converted to algal energy.

The chemical energy contained in an algal cell averages 6000 calories per gram of algae.

Depending on the sky clearance factor for an area, the average visible radiation received can be estimated as follows:

Avg. radiation= Min. radiation + [(Max. radiation - Min. radiation)x sky clearance factor]

Oxygen production occurs concurrently with algal production in accordance with following equation:

$$106CO_2 + 16NO_3 + HPO_4 + 122H_2O + 18H^+ \longrightarrow C_{106}H_{263}O_{110}N_{16}P_1 + 138O_2$$

On weight basis, the oxygen production is 1.3 times the algal production.

Areal Organic Loading

The permissible areal organic loading for the pond expressed as kg BOD/ha.d will depend on the minimum incidence of sunlight that can be expected at a location and also on the percentage of influent BOD that would have to be satisfied aerobically. The Bureau of Indian Standards has related the permissible loading to the latitude of the pond location to aerobically stabilize the organic matter and keep the pond odour free. The values are applicable to towns at sea levels and where sky is clear for nearly 75% of the days in a year. The values may be modified for elevations above sea level by dividing by a factor (1 + 0.003 EL) where EL is the elevation of the pond site above MSL in hundred meters.

Detention Time

The flow of sewage can approximate either plug flow or complete mixing or dispersed flow. If BOD exertion is described by first order reaction, the pond efficiency is given by:

for plug flow: $L_e/L_i = e^{-k1t}$

for complete mixing: $L_e/L_i = \dfrac{1}{1+k_1t}$

For dispersed flow the efficiency of treatment for different dgrees of intermixing is characterized by dispersion numbers.Choice of a larger value for dispersion number or assumption of complete mixing would give a conservative design and is recommended.

Depth

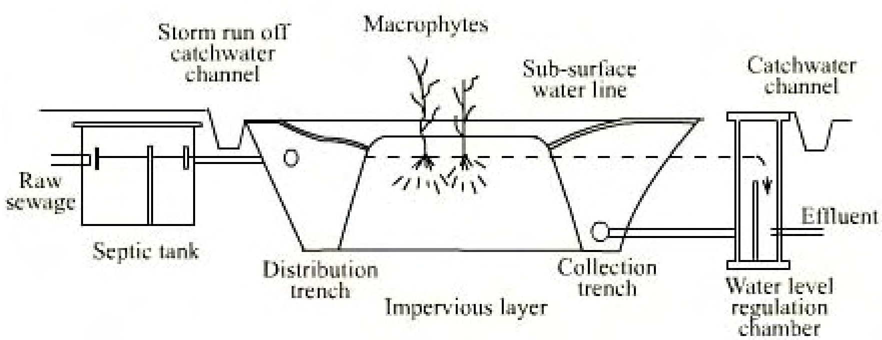
Having determined the surface area and detention capacity, it becomes necessary to consider the depth of the pond only in regard to its limiting value. The optimum range of depth for facultative ponds is 1.0 - 1.5 m.

Aquatic Plant Systems

Aquatic systems in waste treatment are either free floating growths harnessed in the form of built-up ponds for waste treatment such as ***duckweed and hyacinth ponds*** or rooted vegetations (reeds) which emerge out of shallow waters cultivated in ***constructed wetlands***.

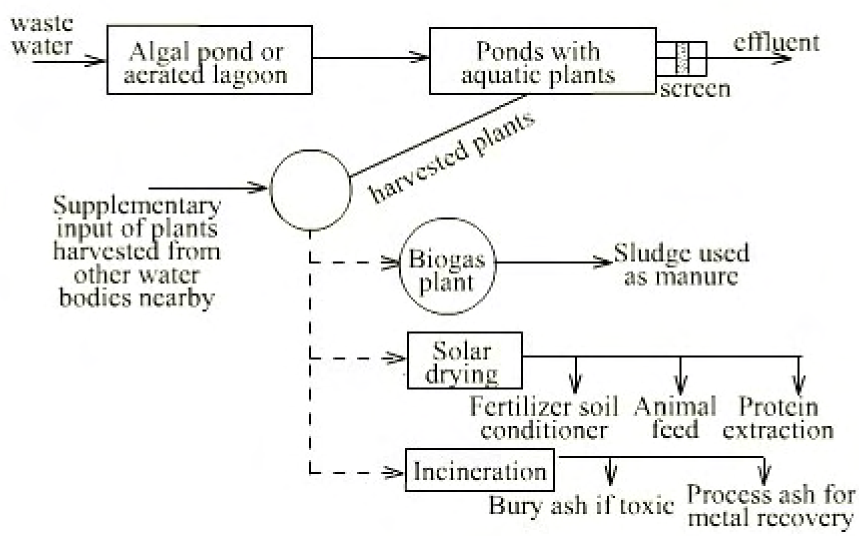
- ***Natural wetlands*** exists all over the world. They generally have saturated soil conditions and abound in rooted vegetation which emerges out of shallow waters in the euphotic zone. They may also have phytoplankton. Natural wetlands can be integrated with wastewater treatment systems.
- ***Constructed wetlands*** are man-made for treatment of wastewater, mine drainage, storm drainage, etc. They have rooted vegetation.

Longitudinal Section Through a Typical Reed Bed With Gravel, Sand or Selected Soil With Horizontal Flow of Wastewater



- ***Aquatic plant ponds*** consisting of free floating macrophytes, such as water hyacinths, duckweeds, etc. have been cultured in ponds either for their ability to remove heavy metals, phenols, nutrients, etc. from wastewaters or to assist in giving further treatment to pretreated wastewaters to meet stringent discharge standards while at the same time producing new plant growths for their gas production or food value.

Conceptual flowsheet showing waste treatment using an aquatic plant pond



Septic Tank

Septic tanks are horizontal continuous flow, small sedimentation tanks through which sewage is allowed to flow slowly to enable the sewage solids to settle to the bottom of the tank, where they are digested anaerobically. The tank is de-sludged at regular intervals usually once every 1-5 years.

Cesspool

It is a pit excavated in soil with water tight lining and loose lining by stone or brick to provide for leaching of wastewater by sides and the pit is covered. The leaching type is suitable for porous soils. The capacity should not be less than one day's flow into the pit. If all the water in a test pit of one meter diameter and 2 m deep, disappears in 24 hours, such soil is best suitable for cesspools. The bottom of the cesspool must be well above the ground water level. After sometime the sides of pit get clogged by the sewage solids, reducing the leaching capacity. At overflow level, an outlet is provided to take-off unleached liquid into a seepage pit. The settled matter is removed at intervals. Water tight cesspools are cleaned every 6 months and their capacity must not be less than 70 l/person/month.

Seepage Pit

The seepage pit is needed to discharge the effluent of cesspool, aquaprivy, septic tank or sullage from bathrooms and kitchens. The difference between seepage pit and cesspool is that the seepage pit is completely filled up with stones. The fine suspended solids adhere to the surface of stones and get decomposed by the zoogleal film, which are on the stones and the effluent is leached into the side walls.

Water & Wastewater Engineering

Worked-out Examples:

Population Forecast by Different Methods

Problem: Predict the population for the years 1981, 1991, 1994, and 2001 from the following census figures of a town by different methods.

Year	1901	1911	1921	1931	1941	1951	1961	1971
Population: (thousands)	60	65	63	72	79	89	97	120

Solution:

Year	Population: (thousands)	Increment per Decade	Incremental Increase	Percentage Increment per Decade
1901	60	-	-	-
1911	65	+5	-	(5+60) x100=+8.33
1921	63	-2	-3	(2+65) x100=-3.07
1931	72	+9	+7	(9+63) x100=+14.28
1941	79	+7	-2	(7+72) x100=+9.72
1951	89	+10	+3	(10+79) x100=+12.66
1961	97	+8	-2	(8+89) x100=8.98
1971	120	+23	+15	(23+97) x100=+23.71
Net values	1	+60	+18	+74.61
Averages	-	8.57	3.0	10.66

+=increase; - = decrease

Arithmetical Progression Method:

$$P_n = P + ni$$

Average increases per decade = i = 8.57

Population for the years,

1981= population 1971 + ni, here n=1 decade

= 120 + 8.57 = 128.57

1991= population 1971 + ni, here n=2 decade

= 120 + 2 x 8.57 = 137.14

2001= population 1971 + ni, here n=3 decade

= 120 + 3 x 8.57 = 145.71

1994= population 1991 + (population 2001 – 1991) x 3/10

= 137.14 + (8.57) x 3/10 = 139.71

Incremental Increase Method:

Population for the years,

1981= population 1971 + average increase per decade + average incremental increase

= 120 + 8.57 + 3.0 = 131.57

1991= population 1981 + 11.57

= 131.57 + 11.57 = 143.14

2001= population 1991 + 11.57

= 143.14 + 11.57 = 154.71

1994= population 1991 + 11.57 x 3/10

= 143.14 + 3.47 = 146.61

Geometric Progression Method:

Average percentage increase per decade = 10.66

$$P_n = P (1+i/100)^n$$

Population for 1981 = Population 1971 x $(1+i/100)^n$

= 120 x (1+10.66/100), *i = 10.66, n = 1*

= 120 x 110.66/100 = 132.8

Population for 1991 = Population 1971 x $(1+i/100)^n$

= 120 x $(1+10.66/100)^2$, *i = 10.66, n = 2*

= 120 x 1.2245 = 146.95

Population for 2001 = Population 1971 x $(1+i/100)^n$

$= 120 \times (1+10.66/100)^3$, $i = 10.66$, $n = 3$

$= 120 \times 1.355 = 162.60$

Population for 1994 = 146.95 + (15.84 x 3/10) = 151.70

Sedimentation Tank Design

Problem: Design a rectangular sedimentation tank to treat 2.4 million litres of raw water per day. The detention period may be assumed to be 3 hours.

Solution: Raw water flow per day is 2.4×10^6 l. Detention period is 3h.

Volume of tank = Flow x Detention period = $2.4 \times 10^3 \times 3/24 = 300\ m^3$

Assume depth of tank = 3.0 m.

Surface area = 300/3 = 100 m^2

L/B = 3 (assumed). L = 3B.

$3B^2 = 100\ m^2$ i.e. B = 5.8 m

L = 3B = 5.8 X 3 = 17.4 m

Hence surface loading (Overflow rate) = $\frac{2.4 \times 10^6}{100} = 24,000\ l/d/m^2 < 40,000\ l/d/m^2$ (OK)

Rapid Sand Filter Design

Problem: Design a rapid sand filter to treat 10 million litres of raw water per day allowing 0.5% of filtered water for backwashing. Half hour per day is used for bakwashing. Assume necessary data.

Solution: Total filtered water = $\frac{10.05 \times 24 \times 10^6}{24 \times 23.5} = 0.42766$ Ml / h

Let the rate of filtration be 5000 l / h / m^2 of bed.

Area of filter = $\frac{10.05 \times 10^6}{23.5} \times \frac{1}{5000} = 85.5\ m^2$

Provide two units. Each bed area 85.5/2 = 42.77. L/B = 1.3; $1.3B^2 = 42.77$

B = 5.75 m ; L = 5.75 x 1.3 = 7.5 m

Assume depth of sand = 50 to 75 cm.

Underdrainage system:

Total area of holes = 0.2 to 0.5% of bed area.

Assume 0.2% of bed area = $\frac{0.2}{100}$ x 42.77 = 0.086 m²

Area of lateral = 2 (Area of holes of lateral)

Area of manifold = 2 (Area of laterals)

So, area of manifold = 4 x area of holes = 4 x 0.086 = 0.344 = 0.35 m^2 .

∴ Diameter of manifold = $(4 \times 0.35/\pi)^{1/2}$ = 66 cm

Assume c/c of lateral = 30 cm. Total numbers = 7.5/ 0.3 = 25 on either side.

Length of lateral = 5.75/2 - 0.66/2 = 2.545 m.

C.S. area of lateral = 2 x area of perforations per lateral. Take dia of holes = 13 mm

Number of holes: $n \frac{\pi}{4} (1.3)^2 = 0.086 \times 10^4 = 860\ cm^2$

$\therefore n = \frac{4 \times 860}{\pi (1.3)^2} = 648$, say 650

Number of holes per lateral = 650/50 = 13

Area of perforations per lateral = 13 x $\pi (1.3)^2 /4$ = 17.24 cm^2

Spacing of holes = 2.545/13 = 19.5 cm.

C.S. area of lateral = 2 x area of perforations per lateral = 2 x 17.24 = 34.5 cm^2.

∴ Diameter of lateral = $(4 \times 34.5/\pi)^{1/2}$ = 6.63 cm

Check: Length of lateral < 60 d = 60 x 6.63 = 3.98 m. l = 2.545 m (Hence acceptable).

Rising washwater velocity in bed = 50 cm/min.

Washwater discharge per bed = (0.5/60) x 5.75 x 7.5 = 0.36 m^3/s.

Velocity of flow through lateral = $\frac{0.36}{\text{Total lateral area}} = \frac{0.36 \times 10^4}{50 \times 34.5}$ = 2.08 m/s (ok)

Manifold velocity = $\frac{0.36}{0.345}$ = 1.04 m/s < 2.25 m/s (ok)

<u>Washwater gutter</u>

Discharge of washwater per bed = 0.36 m^3/s. Size of bed = 7.5 x 5.75 m.

Assume 3 troughs running lengthwise at 5.75/3 = 1.9 m c/c.

Discharge of each trough = Q/3 = 0.36/3 = 0.12 m^3/s.

$Q = 1.71 \times b \times h^{3/2}$

Assume b =0.3 m

$$h^{3/2} = \frac{0.12}{1.71 \times 0.3} = 0.234$$

$\therefore$ h = 0.378 m = 37.8 cm = 40 cm

= 40 + (free board) 5 cm = 45 cm; slope 1 in 40

Clear water reservoir for backwashing

For 4 h filter capacity, Capacity of tank = $\frac{4 \times 5000 \times 7.5 \times 5.75 \times 2}{1000}$ = 1725 m^3

Assume depth d = 5 m. Surface area = 1725/5 = 345 m^2

L/B = 2; $2B^2$ = 345; B = 13 m & L = 26 m.

Dia of inlet pipe coming from two filter = 50 cm.

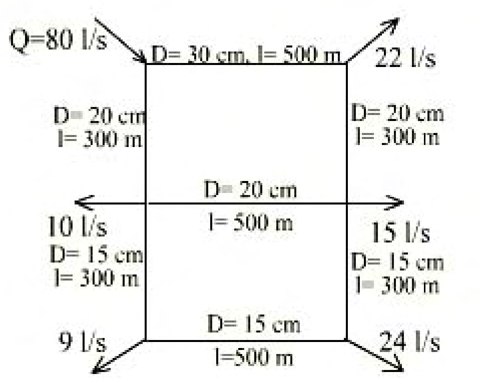
Velocity < 0.6 m/s. Diameter of washwater pipe to overhead tank = 67.5 cm.

Air compressor unit = 1000 l of air/ min/ m^2 bed area.

For 5 min, air required = 1000 x 5 x 7.5 x 5.77 x 2 = 4.32 m^3 of air.

Flow in Pipes of a Distribution Network by Hardy Cross Method

Problem: Calculate the head losses and the corrected flows in the various pipes of a distribution network as shown in figure. The diameters and the lengths of the pipes used are given against each pipe. Compute corrected flows after one corrections.



Solution: First of all, the magnitudes as well as the directions of the possible flows in each pipe are assumed keeping in consideration the law of continuity at each junction. The two closed loops, ABCD and CDEF are then analyzed by Hardy Cross method as per tables 1 & 2 respectively, and the corrected flows are computed.

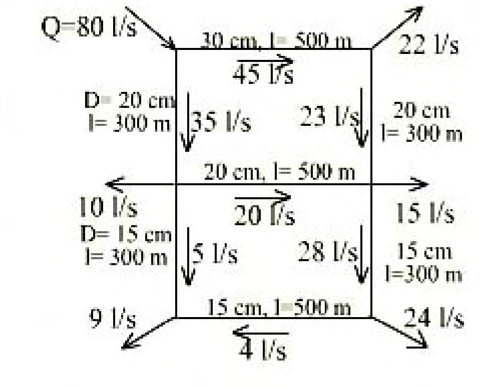


Table 1

Consider loop ABCD

Pipe	Assumed flow		Dia of pipe		Length of pipe (m)	K = L / 470 $d^{4.87}$	$Q_a^{1.85}$	H_L= K. $Q_a^{1.85}$	$\lvert H_L/Q_a \rvert$
	in l/ sec	in cumecs	d in m	$d^{4.87}$					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
AB	(+) 43	+0.043	0.30	2.85×10^{-3}	500	373	3×10^{-3}	+1.12	26
BC	(+) 23	+0.023	0.20	3.95×10^{-4}	300	1615	9.4×10^{-4}	+1.52	66
CD	(-) 20	-0.020	0.20	3.95×10^{-4}	500	2690	7.2×10^{-4}	-1.94	97
DA	(-) 35	-0.035	0.20	3.95×10^{-4}	300	1615	2×10^{-3}	-3.23	92
Σ								-2.53	281

* $H_L = (Q_a^{1.85} L)/(0.094 \times 100^{1.85} \times d^{4.87})$

or $K.Q_a^{1.85} = (Q_a^{1.85} L)/(470 \times d^{4.87})$

or $K = (L)/(470 \times d^{4.87})$

For loop ABCD, we have $\delta = -\Sigma H_L \ / \ x.\Sigma \lvert H_L/Q_a \rvert$

$= (-)\ -2.53/(1.85 \times 281)$ cumecs

$= (-)\ (-2.53 \times 1000)/(1.85 \times 281)$ l/s

$= 4.86$ l/s $= 5$ l/s (say)

Hence, corrected flows after first correction are:

Pipe	AB	BC	CD	DA
Corrected flows after first correction in l/s	+ 48	+ 28	- 15	- 30

Table 2

Consider loop DCFE

Pipe	Assumed flow		Dia of pipe		Length of pipe (m)	K = L / 470 $d^{4.87}$	$Q_a^{1.85}$	H_L= K. $Q_a^{1.85}$	lH_L/Q_al
	in l/ sec	in cumecs	d in m	$d^{4.87}$					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DC	(+) 20	+0.020	0.20	3.95 X10^{-4}	500	2690	7.2 X10^{-4}	+1.94	97
CF	(+) 28	+0.028	0.15	9.7 X10^{-5}	300	6580	1.34 X10^{-3}	+8.80	314
FE	(-) 8	-0.008	0.15	9.7 X10^{-5}	500	10940	1.34 X10^{-4}	-1.47	184
ED	(-) 5	-0.005	0.15	9.7 X10^{-5}	300	6580	5.6 X10^{-5}	-0.37	74
Σ								+8.9	669

For loop ABCD, we have $\delta = -\Sigma H_L$ / x.Σ lH_L/Q_al

=(-) +8.9/(1.85 X 669) cumecs

=(-) (+8.9 X 1000)/(1.85 X 669)) l/s

= -7.2 l/s

Hence, corrected flows after first correction are:

Pipe	DC	CF	FE	ED
Corrected flows after first correction in l/s	+ 12.8	+ 20.8	- 15.2	- 12.2

Trickling Filter Design

Problem: Design a low rate filter to treat 6.0 Mld of sewage of BOD of 210 mg/l. The final effluent should be 30 mg/l and organic loading rate is 320 g/m^3/d.

Solution: Assume 30% of BOD load removed in primary sedimentation i.e., = 210 x 0.30 = 63 mg/l. Remaining BOD = 210 - 63 = 147 mg/l.
Percent of BOD removal required = (147-30) x 100/147 = 80%

BOD load applied to the filter = flow x conc. of sewage (kg/d) = 6 x 10^6 x 147/10^6 = 882 kg/d

To find out filter volume, using NRC equation

$$E_2 = \frac{100}{1+0.44(F_{1.BOD}/V_1.Rf_1)^{1/2}}$$

$80 = \frac{100}{\quad}$ Rf_1= 1, because no circulation.

$1+0.44(\mathit{882}/V_1)^{1/2}$

$V_1 = 2704\ m^3$

Depth of filter = 1.5 m, Fiter area = 2704/1.5 = 1802.66 m^2, and Diameter = 48 m < 60 m

Hydraulic loading rate = 6 x $10^6/10^3$ x 1/1802.66 = $3.33 m^3/d/m^2$ < 4 hence o.k.

Organic loading rate = 882 x 1000 / 2704 = 326.18 $g/d/m^3$ which is approx. equal to 320.