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.

   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.

   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.

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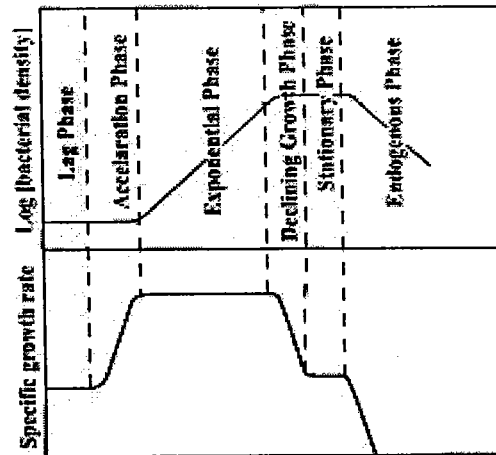
The curve shown may be divided into six well defined phases:

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:

Totalrate of microbial growth, $\dfrac{dx}{t} = \dfrac{\mu_m XS}{K_s + S}$

where,

$\mu_m = \ominus$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,

$\dfrac{dS}{dt} = \dfrac{kX\,S}{K_s + S}$

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,

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

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
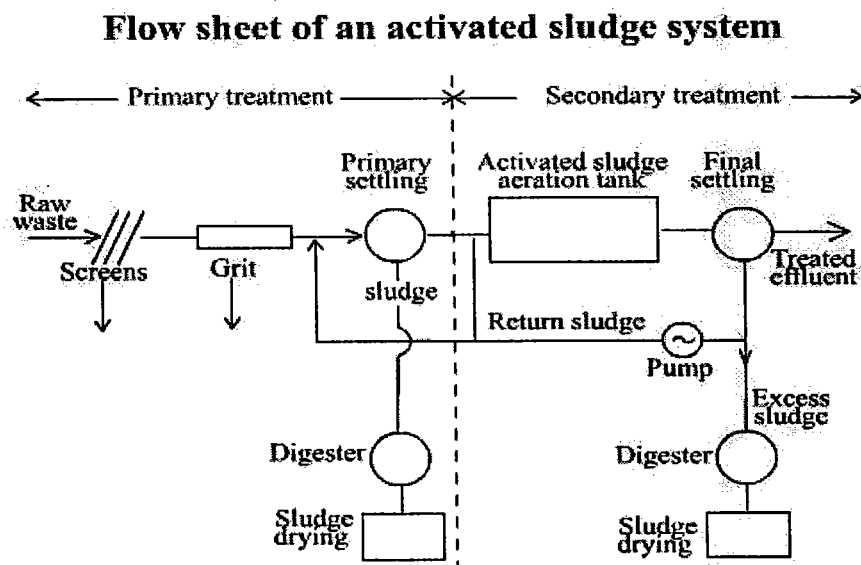
Activated sludge plant involves:

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.

$$q = \frac{Q\,(S_0 - S_e)}{V\,X}$$

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 Θ_c by assuming a suitable value of MLSS concentration, X.

$$VX = \frac{Y Q \Theta_c (S_O - S)}{1 + \Theta k_d \Theta_c}$$

Alternately, the tank capacity may be designed from

$$F/M = Q S_O / XV$$

Hence, the first step in designing is to choose a suitable value of Θ_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/m^3. 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:

$$O_2 \text{ required (g/d)} = \frac{Q(S_0 - S)}{f} - 1.42 \, Q_w X_r$$

where, f = ratio of BOD_5 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).

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<u>Sludge Recycle</u>

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.

<u>Excess Sludge Wasting</u>

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

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

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

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.

High Rate Trickling Filter
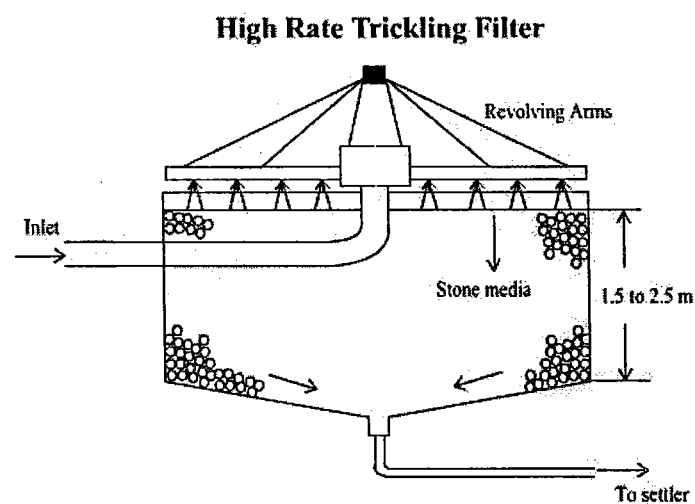
Process Description

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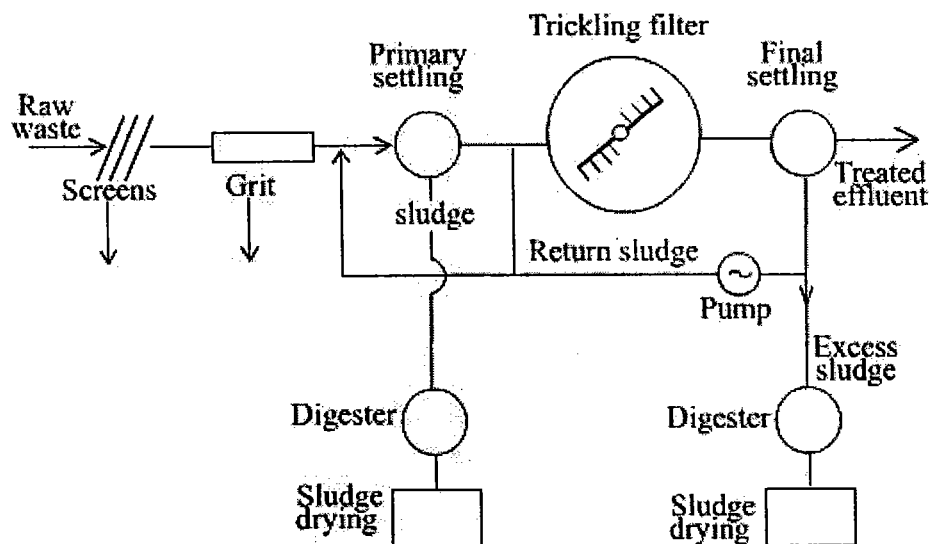
S.No.	Design Feature	Low Rate Filter	High Rate Filter
1.	Hydraulic loading, $m^3/m^2.d$	1 - 4	10 - 40
2.	Organic loading,kg BOD / $m^3.d$	0.08 - 0.32	0.32 - 1.0
3.	Depth, m	1.8 - 3.0	0.9 - 2.5
4.	Recirculation ratio	0	0.5 - 3.0 (domestic wastewater) upto 8 for strong industrial wastewater.

- 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

Trickling filter

Primary settling

Final settling

Raw waste

Screens   Grit

sludge

Treated effluent

Return sludge

Pump

Excess sludge

Digester

Digester

Sludge drying

Sludge drying

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)

   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:

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

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.
- Pond systems, in which oxygen is provided through mechanical aeration rather than algal photosynthesis are called _aerated lagoons_.
- Lightly loaded ponds used as tertiary step in waste treatment for polishing of secondary effluents and removal of bacteria are called *maturation ponds*.

lassification 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.

SYMBIOTIC RELATIONSHIP AND FUNCTIONING OF
FACULTATIVE STABILIZATION POND
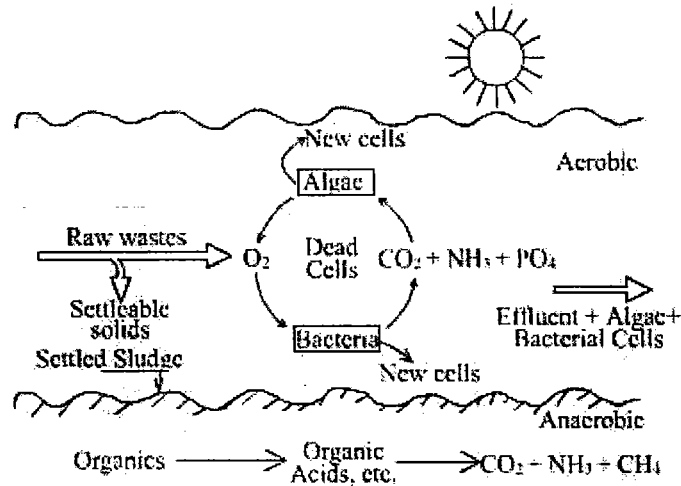
Factors Affecting Pond Reactions

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:

$$106CO_2 + 16NO_3 + HPO_4 + 122H_2O + 18H^+ \longrightarrow C_{106}H_{263}O_{110}N_{16}P_1 + 138O_{2s}$$
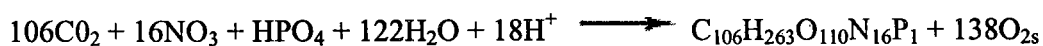
On weight basis, the oxygen production is 1.3 times the algal production.

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: $L_e/L_i = \dfrac{1}{1+k_1 t}$

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.

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:

$$NH_3 \xrightarrow{\text{Nitrosomonas}} NH_4 \xrightarrow{} NO_2 \xrightarrow{\text{Nitrobacter}} NO_3$$
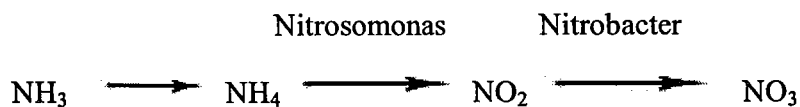
Stoichiometrically, 4.6 kg of oxygen is required for nitrifying 1 kg of nitrogen. Under steady state conditions, experimental evidence has shown nitrite accumulation to be insignificant. This suggests that the rate-limiting step for the conversion of ammonium to nitrate is the oxidation of ammonium to nitrite by the genus Nitrosomonas.

$$q_c = \frac{1}{m}$$

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

The quality of effluent provided by secondary treatment may not be always sufficient to meet discharge requirements. i.e.

- When large quantities are discharged into small streams

- Delicate ecosystems are encountered

Further treatment may be required to remove nutrients (N, P), suspended solids, dissolved inorganic salts and refractory organics

2.1 Nutrient Removal

a. Nitrogen Removal

    -Nitrification-denitrification

    -Air Stripping

b. Phosphorus Removal

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When large quantities are discharged into small

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Further treatment may be required to remove nutrients (N, P), suspended solids, dissolved inorganic salts and refractory organics

Nitrogen Removal using Nitrification-Denitrification

Ammonification

Nitrogen compounds results in wastewater from biological decomposition of proteins and from urea discharged in body waste.

This nitrogen is bound in complex organic molecules and is called **Organic Nitrogen**. While traveling through sewer pipes, the majority of organic-nitrogen is converted to ammonia through the process of hydrolysis.

Biological Characteristics

Microorganisms may be classified according to nutrient requirements All organisms require:

- An Energy source– for (1) maintenance and (2) biosynthesis

- A Carbon Source– for growth of microbes

Heterotrophic – these are microorganisms that uses organic compounds as BOTH a carbon source and as an energy source. These organisms are mostly employed in WWT

Chem-Autotrophs – these are organisms that uses inorganic compounds as BOTH an energy source and a carbon source.

Nitrification

Typical wastewater influent can contain 85 mg/L total

Nitrogen. Though conventional treatment can remove 20 –

30 % , Nitrification- Denitrification can remove 70 – 90%

Ammonia Nitrogen is the most reduced nitrogen compound found in wastewater. This compound can be converted to Nitrogen by biological processes.This process is done in two (2) steps:

Ammonia is first oxidized to

Nitrate Nitrate is reduced to

molecular Nitrogen

The organisms responsible for nitrification are chem-autotrophic bacteria, *nitrosomonas* and *nitrobacter*. These are aerobic bacteria and therefore need free oxygen to work.

Ammonia Nitrogen can be biologically oxidized by chem-autotrophic bacteria to nitrates if molecular oxygen is present:

These reactions require a great supply of oxygen. Contact time in secondary treatment may be sufficient to convert organic nitrogen to ammonia nitrogen but not sufficient to convert ammonia nitrogen to nitrates.

This reaction consumes about 4.6 mg of O2 7.1 mg alkalinity per mg ammonia nitrogen. Under favourable conditions this process can be accomplished in combination with carbonaceous removal in secondary systems. e.g. Extended Aeration System **or** done more efficiently, using a separate nitrification reactor.

De-Nitrification

Nitrate is reduced to nitrogen gas by the same facultative, heterotrophic bacteria involved in oxidation of carbonaceous material.

- Denitrification occurs when oxygen levels are depleted and nitrate becomes the primary oxygen source for microorganisms.

- The process is performed under anoxic conditions, when the dissolved oxygen concentration is less than 0.5 mg/L, ideally less than 0.2.

- When bacteria break apart nitrate (NO3-) to gain the oxygen (O2), the nitrate is reduced to nitrous oxide (N2 O), and, in turn, nitrogen gas (N2).

- For the process to proceed, the bacteria needs a carbon source. This can be obtained from carbon within the waste or a small amount of primary effluent

can be added. Alternatively, an source of carbon can be provided (Methanol). After leaving the anoxic tank, the wastewater is aerated for 10 to 15 minutes to drive off the Nitrogen gas and add oxygen to the wastewater before sedimentation
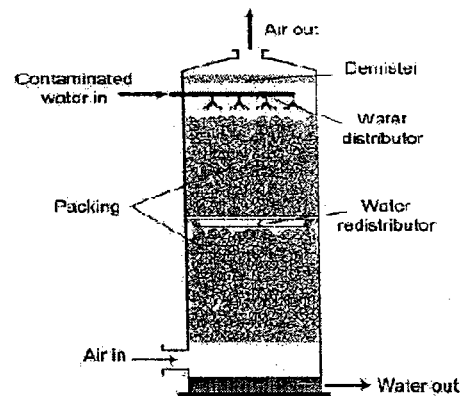
The Air Stripping Process

- The process consist of converting the ammonium to the gaseous phase and then dispersing the liquid in air

- The gaseous phase NH_3 and the aqueous phase NH_4+ exist together in equilibrium and the dominance of any one is dependent on pH and Temperature. A pH of >11 is required for complete conversion to NH_3

The Operation

- Lime is used to raise the pH to >11

- Stripping of de-gasification is most efficiently done using a counter current spray tower.

Design Parameters are:

➢ 2000-6000 m3 of air / m3 wastewater

➢ Tower Depths > 7.5 m

➢ HRL 40– 46 L/min/m2 of tower

Advantagesand Disadvantages of the air Stripping

➢ Air stripping is the most economical means of removing nitrogen, however, as temperature approaches freezing the efficiency drops significantly.

➢ Noise pollution by roaring fans.

➢ Air pollution by odor caused by release of ammonia gas.

➢ Addition of lime cause softening of WW of alkalinity.

➢ The precipitation of calcium carbonate on the packed media therefore requires continuous cleaning.

Phosphorus Removal

Characteristics of Phosphates in WW

➢ Phosphorus is a constituent of municipal wastewater, averaging around 15 – 10 mg/L. It exist in 3 forms

➢ Organically bound phosphorus–Body waste and food waste

➢ Polyphosphates- Used extensively in detergents and contributes to about half the phosphorus in WW

➢ Orthophosphates – Results due to biological decomposition of organically bound phosphates and hydrolysis of polyphosphates

➢ Thus, the principal phosphate found in WW is Orthophosphates

➢ Orthophosphates consist of (phosphate) $PO3-4$, $HPO42-$ and $H2$ $PO4-$ and form chemical bonds with cations and positive radicals.

➢ These compounds are highly soluble, thus negligible removal occurs in primary treatment. However, < 3mg/L is removed in biomass from secondary treatment due to utilization by microorganisms.

At Slightly Acidic pH

➢ Chemical precipitation is the principal method used to remove phosphorus. At slightly acidic pH, orthophosphates combine with trivalent aluminum or iron cations to form a ppt.

➢ Since domestic wastewater only contains trace amounts of iron and aluminum, thus,

➢ Alum (aluminum sulphate) or Ferric Chloride will have to be added.

At Higher pH

➢ Calcium forms an insoluble complex with phosphate at pH >9.0.

➢ The addition of lime can provide both the calcium and Ph adjustments necessary.

Process Selection

➢ The removal of phosphorus can occur as part of the primary or secondary treatment process or as a tertiary process. The choice of process depends on efficiency requirements,

a. If up to 1mg/L is acceptable for discharge, iron or aluminum salts added to the primary or secondary process is often done.

b. If greater efficiency is needed, tertiary system is employed with the addition of lime.

SolidsRemoval-SuspendedSolidsRemoval

> The removal of suspended solids from wastewater refers to the removal of particles and floc to small or too lightweight to be removed in gravity settling.

> These particles may have been brought over from secondary treatment or ppt in tertiary treatment.

Methods of removal

    1. Centrifugation

    2. Air Floatation

    3. Mechanical Micro straining

    4. Filtration (most common)

Filtration

Slow Sand Filters

> This method is most successful as a polishing step in oxidation ponds.

> (Not suitable for effluent from conventional treatment due to clogging)

Granular-media Filtration

The bed comprise duel or multimedia beds and is most suited for effluent from secondary treatment

Moving Bed Filters

These are continuous cleaned, with the rate of cleaning adjusted to match the solids loading rate. This system as the ability to filter raw sewage.

Pulse-bed Filters

Compressed air is periodically injected to break up the thin surface mat of deposits.This system as the ability to filter raw sewage.

Solids Removal-Dissolved Solids Removal

Secondary treatment as well as nutrient removal decreases the dissolved organic solids present in WW. However, neither process completely removes ALL organic dissolved solids OR significant amounts of inorganic dissolved solids.

If substantial reduction in dissolved solids is required, further treatment would be needed.

These techniques are similar to that used in the advanced treatment of Water for removal:

- Ion Exchange
- MicroporousMembrane Filtration
- Ads

orption

Chemica

Oxidation

Chemical

Oxidation

This technique can be used as an alternative to adsorption for the removal of refractory organic compounds from water and wastewater treatment system The target contaminants include; large complex organic, ring-structured detergents, phenolic & humic compounds. These are broken down into simple compounds by strong oxidants e.g. Ozone, Chlorine.

Advantage and Disadvantages

Advantages

- Removal of ammonia
- Oxidation of inorganic substances as iron and manganese
- Disinfection

Disadvantage

- Chlorine reacts with some organics to form halo form
- High doses of ozone is required 3:1

Wastewater Disposal

- The most common method of disposal is by dilution. Disposal to a stream is dependent of the
- level of dilution capable by the stream as well as the sensitivity of the stream to small changes
- Otherwise, tertiary treatment may be needed before discharge. This is normally in the form of nutrient removal.
- **Natural Evaporation**
- The process is most useful in climates where evaporation exceeds precipitation.
- The system is essentially large oxidation ponds with a surface area suited to

the rate of inflow.

Ocean Disposal

This is a efficient and cost effective method. The effluent is transported out to sea by pipelines along the ocean floor and discharged at multiple points. The length of the outfall depends on the ocean currents and volume of wastewater.

Land Application

- Land application can be a form of disposal as well as a method of reuse. These include
- Irrigation and Rapid Infiltration

Irrigation

1. Wastewater is applied to land surface to provide both water and nutrients for plant growth.

2. Applications include agriculture, civil culture; maintain vegetation in parks, golf courses, along roadways and airport runways.

3. 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.

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,

SRT= Total sludge present in reactor, kg
      Sludge withdrawn per day, kg/d

    = 30 to 50 days.

Another parameter is HRT which is given by:

HRT= Reactor volume, m^3
      Flow rate, m^3/h

    = 8 to 10 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 up flow velocity can be determined from

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

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.
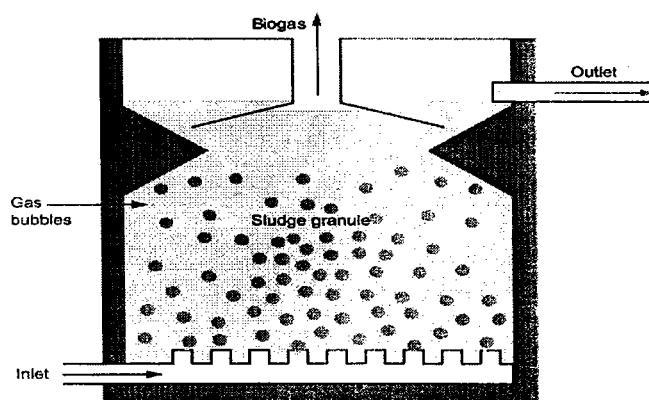
Septic tank

A septic tank can be defined as primary sedimentation tank with large detention time (12 to 36hrs against a period of 2hrs in an ordinary sedimentation tank).

In un-sewered rural and urban areas septic tanks are suitable for disposal of night soil. But sufficient water should be available as water is required for flow of the night soil from latrine to the septic tank and for proper functioning of the septic tank.

The size of the septic tank is so designed that the sewage is retained in the tank for 24hrs during which certain biological decomposition by the action of anaerobic bacteria takes place which liquefies and breaks the night soil leaving small quantity of soil which is known as sludge and settles at the bottom of the tank and clear water known as effluent flows out of the tank.

The effluent from the septic tank is usually disposed by absorption in the soil through soak pit, if no municipal drainage system is prevailing in the area. If municipal drainage line exists in the area, the effluent is discharged to the drain.

It is to be noted that disinfection agent such as bleaching powder, phenyl etc. should not be used in cleaning latrines as disinfectant entering the septic tank kills the bacteria growth as a result of which rate of biological decomposition is retarded.
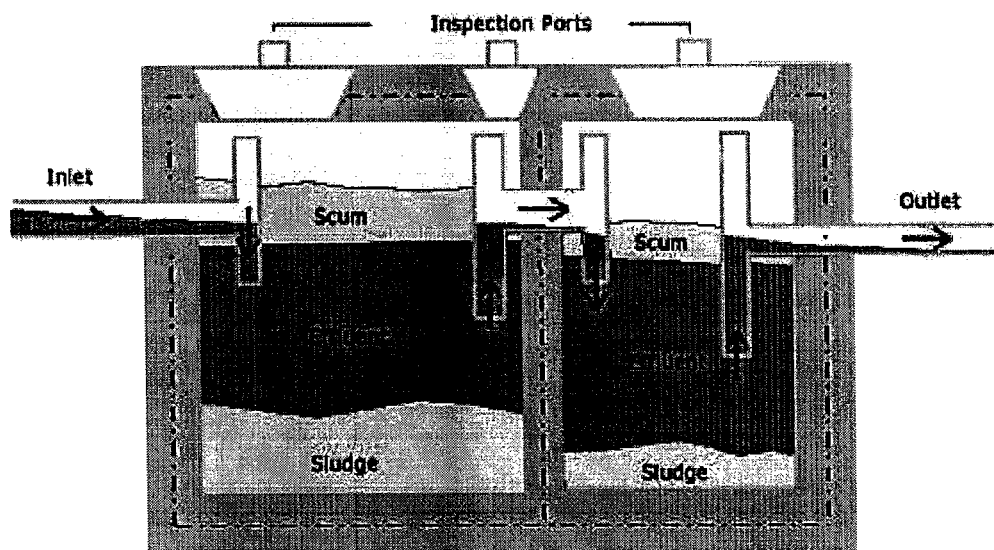
Design of Septic Tank

The capacity of septic tank depends on number of users and interval of sludge removal. Normally sludge should be removed every 2 years. The liquid capacity of tank is taken as 130 liters to 70 liters per head. For small number of users 130ltr per head is taken.

A septic tank is usually provided with brick wall in which cement mortar [not less than 20cm (9 inch)] thick and the foundation floor is of cement concrete 1:2:4. Both inside and outside faces of the wall and top of the floor are plastered with minimum thickness of 12mm (one-half inch) thick cement mortar 1:3 mix.

All inside corners of septic tank are rounded. Water proofing agent such as Impermo, Cem-seal or Accoproof etc. is added to the mortar at the rate of 2% of the cement weight. Water proofing agent is to be added in similar proportion in to the concrete also for making floor of the tank.

For proper convenience in collection and removal of the sludge, the floor of septic tank is given a slope of 1:10 to 1:20 towards the inlet side. Which means that floor of the outlet side will be on the higher elevation than the floor at inlet side.

Dimensions of Septic Tank Components

i) Length, Width and Depth of Septic Tank

Width = 750mm(min)

Length = 2 to 4 times width

Depth = 1000 to 1300mm. (min below water level) + 300 to 450mm free board

Maximum depth = 1800mm + 450 mm free board

Capacity = 1 cubic meter (10 cubic feet) minimum

ii) Detention period

Detention period of 24hrs (mostly) considered in septic tank design. The rate of flow of effluent must be equal to the rate of flow of influent.

iii) Inlet and outlet pipes

An elbow or T pipe of 100mm diameter is submerged to a depth of 250-600mm below the liquid level. For outlet pipe an elbow or T type of 100mm diameter pipe is submerged to a depth of 200-500mm below the liquid level. Pipes may be of stone ware or asbestos.

iv) Baffle Walls of Septic Tank

For small tanks, RCC hanging type scum baffle walls are provided in septic tanks. Baffle walls are provided near the inlet. It is optional near the outlet.

The inlet baffle wall is placed at a distance of L/5 from the wall, where L is the length of the wall. The baffle wall is generally extended 150mm above to scum level and 400-700mm below it.

Scum being light, generally floats at the water level in the tank. Thickness of the wall varies from 50mm to 100mm. for large tanks lower portion are having holes for flow of sludge.

v) Roofing Slab of Septic Tank

The top of the septic tank is covered with a RCC slab of thickness of 75-100mm depending upon the size of the tank. Circular manholes of 500mm clear diameter are provided for inspection and desludging. In case of rectangular opening clear size is kept as 600X450mm.

vi) Ventilation Pipe

For outlet of foul gases and ventilation purpose cast iron or asbestos pipe of 50-100mm diameter is provided which should extend 2m (min) above ground level. Top of the ventilation pipe is provided with a mosquito proof wire mess or cowl.
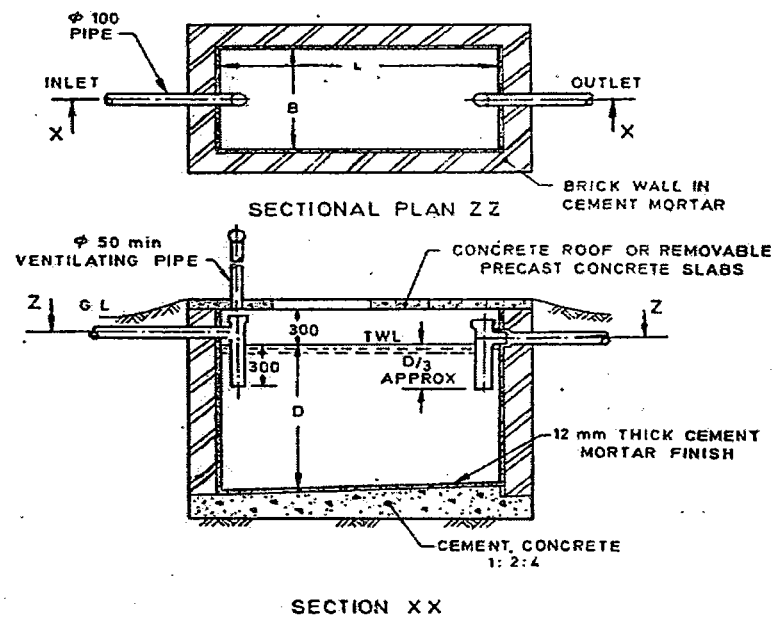
Fig: Sectional plan ZZ shoes the typical layout of the septic tank. Section XX shows the Cross-Sectional detail of septic tank.

Example – Design of Septic Tank for 20 Users

  Liquid capacity of the tank:

@120lts per user = 0.12 X 20 =2.4cum

Take liquid depth as 1.3meter.

Therefore Floor area of the tank = 24/1.3 = 1.85m^2

Taking length as 2.5times the breath

L X B=1.85

2.5B X B = 1.85

B = Sqrt(1.85/2.5) = 0.86 say 0.9m

Therefore, the dimension of the tank is **22.5 X 0.9m**

Biological Treatment I:
Sewage Filtration

3.1. INTRODUCTION

The objectives of the biological treatment of wastewater are to coagulate and remove the nonsettleable colloidal solids and to stabilise the organic matter. *Biological treatment systems are living systems which rely on mixed biological cultures to break down waste organics and remove organic matter from the solution.* Biological treatment systems are designed to maintain a large active mass of bacteria within the system confines. A treatment unit provides a controlled environment for the desired biological process. Domestic wastewater (*i.e.* sewage) supplies the biological food, growth nutrients, and inoculum.

Biological processes are classified by oxygen dependence of primary micro-organisms responsible for waste treatment. Based on this, biological processes may be (*a*) aerobic, (*b*) anaerobic, and (*c*) aerobic-anaerobic. *Aerobic processes* are those which occur in the presence of dissolved oxygen. The aerobic processes include the following : (*i*) Trickling filters ; (*ii*) Activated sludge processes ; (*iii*) Aerobic stabilization ponds ; (*iv*) Aerated lagoons.

Anaerobic waste treatment involves the decomposition of organic and/or inorganic matter in absence of molecular oxygen. Anaerobic processes consist of the followings : (*i*) Anaerobic sludge digestion; (*ii*) Anaerobic contact processes ; (*iii*) Anaerobic filters, and (*iv*) Anaerobic lagoons and ponds.

(331)

Aerobic-anaerobic processes are those in which stabilization of waste is brought about by a combination of aerobic, anaerobic and facultative bacteria. Most of the biological treatment processes are *preferred* to work on *aerobic bacterial decomposition* because such decomposition does not produce bad smells and gases as produced by anaerobic decomposition, and also because aerobic bacteria are about three times more active than anaerobic bacteria at 30°C.

13.2. BIOLOGICAL TREATMENT TECHNIQUES

The *biological treatment techniques* used may be classified under the following three heads : (*i*) Attached growth processes (or fixed film processes), (*ii*) Suspended growth processes, and (*iii*) Combined processes.

(*a*) *Attached growth processes (or fixed film processes)* : These are the biological treatment processes in which the microorganisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissue are attached to some inert medium, such as rock, slag or specially designed ceramic or plastic materials. Such processes include the followings :

(*i*) Intermittent sand filters; (*ii*) Trickling filters ; (*iii*) Rotating biological contactors ; (*iv*) Packed bed reactors ; (*v*) Anaerobic lagoons (ponds) ; (*vi*) Fixed film denitrification. ·

(*b*) *Suspended growth processes* : These are the biological treatment processes in which the micro-organisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissue are maintained in suspension within the liquid in the reactor by employing either natural or mechanical mixing. In most processes, the required volume is reduced by returning bacteria from the secondary clarifier in order to maintain a high solids concentration. The suspended growth processes include the following:

(*i*) Activated sludge processes; (*ii*) Aerated lagoons; (*iii*) Sludge digestion systems; (*iv*) Suspended growth nitrification and suspended growth denitrification.

(*c*) *Combined processes* : These consist of both attached growth processes as well as suspended growth processes. They include the following in sequence :

(*i*) Trickling filter, activated sludge ; (*ii*) Activated sludge, trickling filter ; (*iii*) Facultative lagoons.

It should be noted that various types of biological processes only help in changing the unstable organic matter into stable forms which are then removed in the secondary settling tanks. *Thus, in the biological treatment processes, secondary settling tanks are essential.* Out of various biological treatment processes enumerated above, *sewage filtration* and *activated sludge processes* are the one which are most

sludge...

9.38. Wasting of Excess Sludge (Q_w)

We know that the sludge generated in the aeration tank has to be partly discharged and wasted out of the plant to maintain a steady level of MLSS in the system. *The excess sludge quantity will increase with the increasing F/M ratio, and decrease with temperature.* In the case of domestic sewage, Q_w will be about 0.50—0.75 kg per kg BOD removed for the conventional sludge plants (having F/M ratio varying between 0.4 to 0.3).

Excess sludge may be wasted either from the sludge return line, or directly from the aeration tank as mixed liquor. The latter procedure is usually preferred, since the concentration of suspended solids will then be fairly steady in the waste discharge making the control easy.

In conventional plants, the wasted sludge is taken directly to a sludge thickner and digestor, or to the primary settling tank for its disposal along with the primary sludge. In extended aeration plants, however, the excess sludge is directly taken to the sludge drying beds.

9.39. Modifications of the Basic Activated Sludge Process

In the basic activated sludge process, also called *conventional aeration process,* the recirculated activated sludge is added to the inlet end of the aeration tank as a single dose. The regime flow employed in the aeration tank is *plug flow* and not *mixed flow.* Plug flow implies that the sewage moves down progressively along the aeration tank, essentially unmixed with the rest of the tank con-

tents. The other type of flow regime, called **complete mixed flow**, involves the rapid dispersal of the incoming sewage throughout the tank, and is adopted in the *extended aeration process* (described a little later).

In a conventional aeration tank (of plug flow type), the F/M ratio and the oxygen demand will be the highest at the inlet end, and will then progressively decrease. In the complete mix system on the other hand, the F/M ratio and oxygen demand will be uniform throughout the tank.

The plug flow regime is achieved in such an activated process by employing a long and narrow configuration of the aeration tank with length equal to 5 to 50 times the width. The sewage and the mixed liquor are let in at the head of the tank and withdrawn at its end. Because of the plug flow regime, the oxygen demand at the head of the aeration tank is high and then tapers down. However, air is supplied in the process at a uniform rate along the length of the tank. *This leads to either oxygen deficiency in the initial zone or wasteful application of air in the subsequent reaches.*

The conventional system is always preceded by primary settling. The plant itself consists of an aeration tank, a secondary settling tank, a sludge return line and an excess sludge waste line leading to digestor. The BOD removal in this process is 85—92%.

The main **limitations of the conventional system** are that : (i) the *aeration tank volume requirement is high ; and (ii) there is a lack of operational stability at times of excessive variation in the rate of inflow or its BOD strength.* Inorder to overcome such difficulties posed by a conventional system plant, and to meet specific treatment objectives, several modifications of the conventional system have been suggested by modifying the process variables. The important modified processes are :

(i) *Tapered aeration process* ;

(ii) *Step aeration process* ;

(iii) *Contact stabilisation process* ;

(iv) *Complete mix process* ;

(v) *Modified aeration process* ;

(vi) *Extended aeration process* ; and

(vii) *Activated aeration process.*

Inspite of its various limitations, the conventional system for historical reasons, is the most widely used type of the activated sludge process. Plants up to 300 MLD capacity have been built in India. In addition to conventional activated sludge plants, the **complete mixed plants** and the **extended aeration plants** have also been found a wider acceptance in modern days, particularly for obtaining high BOD removals in smaller capacity plants.

We will now describe the above mentioned modifications of the basic activated sludge process.

9.39.1. Tapered Aeration Process. This process involves a very little modification of the conventional process, and ensures higher air supply at the inlet and in the initial length of the tank, as compared to the downstream length. The process is surely based on the fact that as the mixed liquor progresses through the aeration tank, its air requirement goes on reducing. Therefore, in a tapered aeration plant, compressed air is supplied at higher rates near the inlet end of the tank, and is gradually decreased as sewage moves towards the outlet end of the tank. *Such a process therefore helps us in ensuring optimal application of air in the aeration tank.*

Ordinarily, 45% of air is supplied to the first one-third length of the tank, 30% to the second one-third length of the tank, and the rest 25% to the remaining one-third length of the tank.

No. of diffuser plates are thus varied accordingly. *Such a modification to the conventional activated plants using diffused air aeration, has now-a-days become a common feature, and is invariably adopted in all modern designs.* The loading parameters of such a plant do not materially differ from a conventional one, and are given in Table 9.10.

9.39.2. Step Aeration Process. In the step aeration process, the sewage is introduced along the length of the aeration tank in several steps, while the return sludge is introduced at the head, as shown in Fig. 9.45.
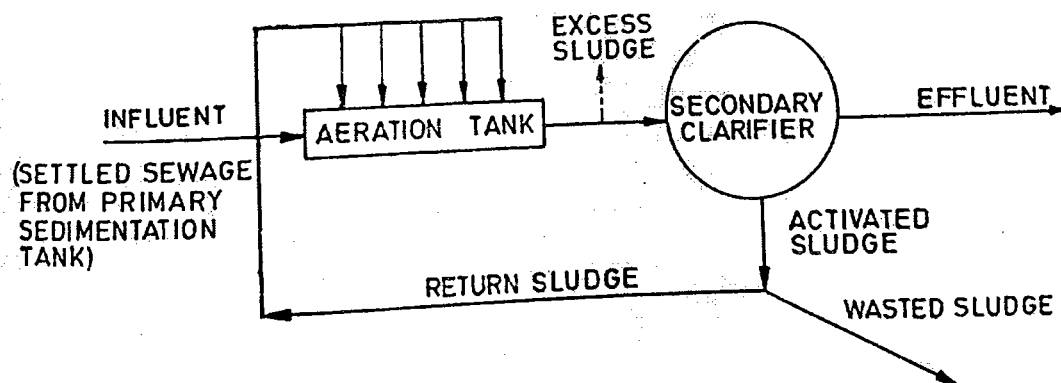

Fig. 9.45. Flow chart of step-aeration process.

Such an arrangement results in a uniform air requirement along the entire length of the tank, and hence the uniform air supply of the conventional plants, can be efficiently used. The process enables an appreciable reduction in the aeration tank volume, without lowering the BOD removal efficiency. Step aeration method has considerable capacity to absorb shock organic loadings. The method has found application for larger plants of capacities up to about 1000 MLD.

The loading parameters of a such a plant are given Table 9.10.

9.39.3. Contact Stabilisation Process or Biosorption Process. This process has been designed for treating colloidal wastewaters.

In this process, the sewage and recycled or returned sludge are mixed and aerated for a comparatively shorter period of 0.5 to 1.5 hour in a special mixing tank, called *contact tank*. This mixing will allow the suspended and dissolved organic matter to be sorbed to the activated sludge floc. The sorbed organics and flocs are removed in the secondary settling tank, where the effluent from the contact tank enters. These settled sorbed organics and flocs are then transferred to a sludge aeration tank (called stabilisation tank or aerodigestor) where the organics are stabilised over a period of about 3 to 6 hours before it is fed back into the contact aeration tank. The stabilised sludge is then mixed with the influent wastewater again, and the process is repeated. The flow diagram for this process is shown in Fig. 9.46, and the loading parameters are given in Table 9.10.
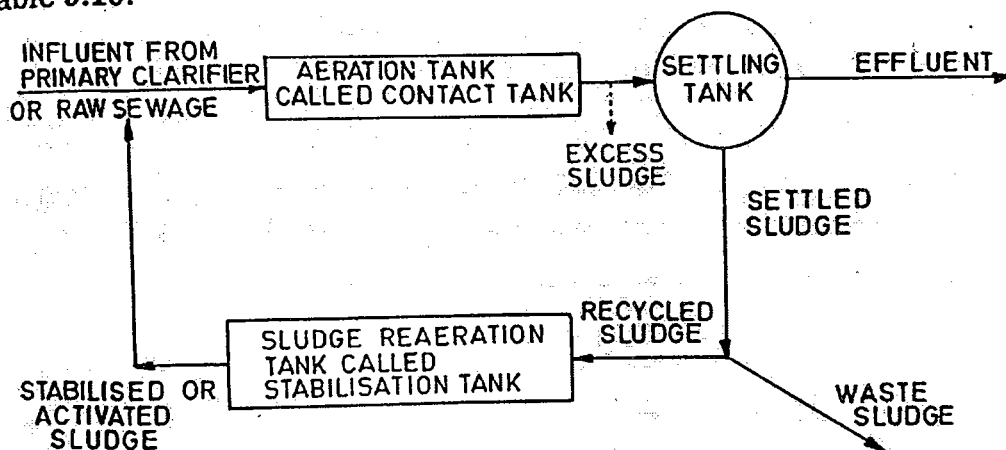

Fig. 9.46. Contact-stabilisation process.

The contact stabilisation process is quite effective in the removal of colloidal and suspended organic matter, but it is not very effective in removing soluble organics. The method is well suited for the treatment of fresh domestic sewage, containing only a low percentage of soluble BOD.

Compared to the conventional system, the contact stabilisation process has greater capacity to handle shock organic loadings, because of the biological buffering capacity of the sludge reaeration tank. The process also presents greater resistance to toxic substances in the sewage as the biological mass is exposed to the main stream of sewage containing the toxic constituents only for a short time.

The air requirements of the process are the same as for the conventional system, the air supply being divided equally between the contact aeration tank and the sludge re-aeration tank. However, the total aeration tank volume required for both the aeration tanks is only about half of the volume reqd. in the single conventional sludge aeration tank. The process therefore presents an effective method of uprating the existing conventional plants, where sewage

characteristics are satisfactory. Moreover, the total aeration time is considerably reduced, and the plant capacity is thereby increased. The process has found application in medium sized plants with capacities up to 40 MLD.

9.39.4. Complete Mix Process.

Complete mix activated sludge plants were developed particularly for smaller cities, where the hourly variations in sewage were quite high, and as such conventional plants were experiencing serious problems of biological instability.

In such a plant, the plug flow regime of a conventional plant is replaced by a completely mixed flow regime. Such a flow regime can be achieved by thorough mixing of sewage and return sludge. Sewage and return sludge are therefore distributed uniformly along one side of the aeration tank, and the aerated sewage is withdrawn uniformly along the opposite side. Mechanical aerators are installed in the centre of a circular or square aeration tank, which may be used in such a plant. Such mechanical aerators must have adequate mixing capacity to ensure thorough mixing of sewage and return sludge. The flow chart for such a plant is shown in Fig. 9.47, and the loading parameters of a such a plant are given in Table 9.10.
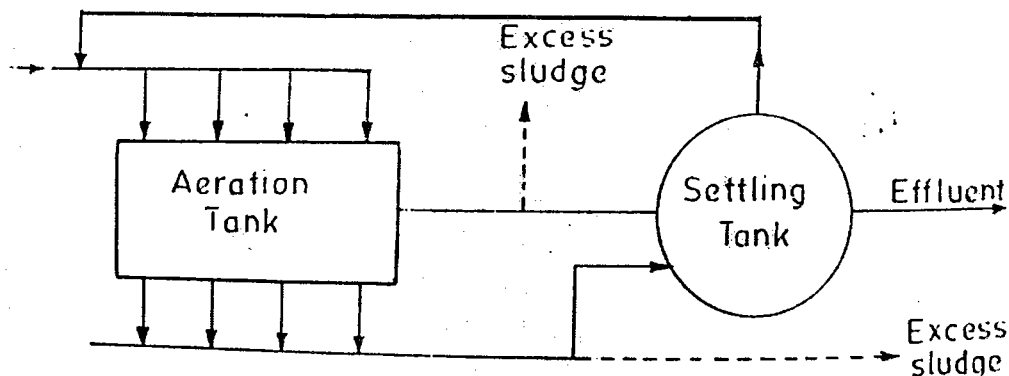

Fig. 9.47. Flow chart of a complete mix plant.

The complete mix plant possesses capacity to hold much higher MLSS concentration level in the aeration tank ; say 3000 to 4000 mg/l as against 1500 to 3000 mg/l of a conventional plant. *This helps in adopting smaller volume for the aeration tank. The plant has an increased operational stability at shock organic loadings, and also increased capacity to treat toxic biodegradable wastewaters like phenols. Such a plant is less liable to upsets by slugs of flows of toxic wastewaters.*

Such plants have been widely used for smaller plant capacities of less than 25 MLD or so, particularly for the towns where municipal and industrial wastewaters flow together.

9.39.5. Modified Aeration Process.

When an intermediate quality of effluent containing higher BOD is permissible, such as at

a place where effluent is to be used for farming, this modified aeration plant may be adopted, as it leads to substantial savings in construction and aeration costs.

Such a process does not need any primary sedimentation tank, as is invariably required in a conventional plant. The process ensures short aeration period, high volumetric loading, high *F/M* ratio, low percentage of sludge return, low concentration of MLSS, as reflected in Table 9.10. The BOD removal is also low, say only 60—75% or so. The process has been employed mainly in large plants with capacities above 200 MLD.

9.39.6. Extended Aeration Process. The flow scheme of an extended aeration process and its mixing regime are similar to that of the complete mix process. Primary sedimentation is frequently avoided in this process, but grit chamber or comminutor is often provided for screenings.

As its name suggests, the aeration period is quite large and extends to about 12—24 hours, as compared to 4 to 6 hours in a conventional plant. The loading parameters for such a process are already given in table 9.10. The process permits low organic loading, high MLSS concentration, and low F/M ratio. The BOD removal efficiency is also quite high, to say about 95—98% as compared to 85—92% of a conventional plant.

The air or oxygen requirement, is of course quite high, which increases the running cost of the plant considerably. The plant, however, offers another advantage, as no separate sludge digestor is required here, because the solids undergo considerable endogenous respiration and get well stabilised over the long detention periods, adopted in the aeration tank. The sludge produced is, thus, capable to be directly taken to the sludge drying beds. Also, the excess sludge production is minimum. *The operation is also simpler due to the elimination of primary settling and separate sludge digestion.* The typical plant based on this principle is known as an *oxidation ditch,* and is described separately under article 9.46.

Such a process is quite suitable for small communities having sewage flows of less than 4 MLD or so.

9.39.7. Activated Aeration Process. In the activated aeration process, a pair of conventional activated sludge plants operating in parallel are used. Excess activated sludge from the secondary settling tank of one unit will supply to the aeration tanks of both units. and the sludge from the second unit is pumped to final disposal. This arrangement, although no modification of the conventional process in the strict sense, reduces the construction costs, and probably operating costs, but does not modify the operating results.

Example 9.30. *Design a conventional activated sludge plant to treat domestic sewage with diffused air aeration system, given the following data :*

Population	= 35,000
Average sewage flow	= 180 lpcd
BOD of sewage	= 220 mg/l
BOD removed in primary treatment	= 30%
Overall BOD reduction desired	= 85%.

Solution. Daily sewage flow

$$= Q = 180 \times 35000 \text{ l/day} = 6300 \text{ m}^3/\text{day}.$$

BOD of sewage coming to aeration

$$= Y_0 = 70\% \times 220 \text{ mg/l} = \textbf{154 mg/l}$$

$(\because$ 30% BOD is removed in primary settling)

BOD left in effluent $= Y_E = 15\% \times 220 \text{ mg/l} = 33 \text{ mg/l}$

$(\because$ Overall 85% BOD removal is desired)

$\therefore$ BOD removed in activated plant

$$= 154 - 33 = \textbf{121 mg/l}$$

$\therefore$ Efficiency required in Activated plant

$$= \frac{121}{154} = 0.79$$

From table 9.10, for efficiency of 85–92%, we use F/M ratio as 0.4 to 0.3, and MLSS between 1500 to 3000. Since efficiency required is on lower side, we can use moderate figures for F/M ratio and MLSS.

So let us adopt F/M = 0.35
Similarly adopt MLSS $(X_t) = 2000$ mg/l
Using equation (9.44), we have

$$\frac{F}{M} = \frac{Q \cdot Y_0}{V \cdot X_t}$$

where $F/M = 0.35$ (assumed)

$$Q = 6300 \text{ m}^3/\text{day}$$
$$Y_0 = 154 \text{ mg/l} = 154 \text{ gm/m}^3$$
$$X_t = 2000 \text{ mg/l (assumed)}$$

$\therefore$

$$0.35 = \frac{6300 \times 154}{V \times 2000}$$

$\therefore$

$$V = \text{volume of aeration tank}$$

$$= \frac{6300 \times 154}{2000 \times 0.35} = \textbf{1386 m}^3.$$

(i) Check for Aeration period or H.R.T. (t)

Using Eq. (9.41), we have

$$t = \frac{V}{Q} \times 24 \text{ h} = \frac{1386}{6300} \times 24 \text{ h}$$

$$= 5.28 \text{ h (within the limits of 4 to 6 h)} \quad \text{O.K.}$$

(ii) *Check for S.R.T.* (θ_c)

From equation (9.56), we have

$$V \cdot X_t = \frac{\alpha_y \cdot Q(Y_0 - Y_E)\theta_c}{1 + K_e \cdot \theta_c}$$

where V = 1386 m^3

X_t = 2000 mg/L

α_y = yield coefficient = 1.0 w.r.t. MLSS

Q = 6300 m^3/d

K_e = Endogeneous respiration constant = $0.06d^{-1}$

Y_0 = BOD of influent in aeration tank = 154 mg/L

Y_E = BOD of effluent = 33 mg/L

Substituting the values, we get

$$1386 \times 2000 = \frac{0.5 \times 6300\,(154 - 33)\theta_c}{1 + 0.06 \times \theta_c}$$

or $\qquad 1 + 0.06\theta_c = \left(\dfrac{1.0 \times 6300 \times 121}{1386 \times 2000}\right)\theta_c = 0.275\,\theta_c$

$\qquad\qquad 1 + 0.06\theta_c = 0.275\,\theta_c$

$\qquad\qquad\quad 1 = (0.275 - 0.06)\,\theta_c$

or $\qquad\qquad 1 = 0.215\,\theta_c$

$$\theta_c = \frac{1}{0.215} = 4.65 \text{ days} \approx \textbf{5 days} \text{ ; OK,}$$

as it lies between 5 to 8 days.

(iii) *Check for volumetric loading*

Using equation (9.42), we have

Volumetric loading $= \dfrac{Q \cdot Y_0}{V}$ gm of BOD/m^3 of tank vol.

$$= \frac{6300 \times 154}{1386} \text{ gm/m}^3 = 700 \text{ gm/m}^3 = 0.7 \text{ kg/m}^3$$

(within the permissible range of 0.3—0.7 kg/m^3) ; O.K.

(iv) *Check for Return sludge ratio (for SVI ranging between 50—150 ml/gm ; say 100 ml/gm).*

Using equation (9.55), we have

$$= \frac{Q_R}{Q} = \frac{X_t\,(i.e.\ \text{MLSS})}{\dfrac{10^6}{\text{SVI}} - X_t}$$

where SVI = 100 ml/gm

X_t = 2000 mg/l

$$= \frac{2000}{\left(\dfrac{10^6}{100} - 2000\right)}$$

= 0.25 (*i.e.* within the prescribed range of 25 to 50%). ; O.K.

We will, for conservative purposes, however provide 33% return sludge, giving SVI = 125, O.K. The **sludge pumps** for bringing recirculated sludge from the secondary sedimentation tank will thus have a capacity = 33% × Q = 33% × 6300 m³/d = **2100 m³/d. Ans.**

Tank Dimensions. Adopt aeration tank of depth 3 m and width 4.5 m. The total length of the aeration channel required

$$= \frac{\text{Total volume required}}{B \times D} = \frac{1386}{4.5 \times 3} \text{ m}$$

= 102.7 m ; say 105 m.

Provide a continuous channel, with 3 aeration chambers, each of 35 m length. Total width of the unit, including 2 baffles each of 0.25 m thickness = 3 × 4.5 m + 2 × 0.25 = 14 m. Total depth provided including free-board of 0.6 m will be 3 + 0.6 = 3.6 m.

Overall dimensions of the Aeration tank will be 35 m × 14 m × 3.6 m. **Ans.**

Rate of Air Supply Required. Assuming the air requirement of the aeration tank to be 100 m³ of air per kg of BOD removed*, we have

Air required *i.e.* blower capacity

$$= 100 \times \frac{121 \times 6300}{1000} \text{ m}^3/\text{day}$$

$$= 121 \times 630 \times \frac{1}{24 \times 60} \text{ m}^3/\text{min}$$

$$= \textbf{53 m}^3/\textbf{min. Ans.}$$

Let standard diffuser plates of 0.3 m × 0.3 m × 25 mm size, releasing 1.2 m³ of air/min/m² with 0.3 mm pores may be used. Then, the total No. of plates required

$$= \frac{53}{1.2 \times 0.3 \times 0.3} = 491 \text{ ; say } \textbf{500.}$$

Let us now provide more plates in the initial length of the tank ; say provide 45% of plates in the 1st $\frac{1}{3}$ length ; 30% of plates in the 2nd $\frac{1}{3}$ length ; and 25% in the last $\frac{1}{3}$ length.

∴ Plates to be provided in the 1st chamber = 45% × 500 = 225
Plates to be provided in the 2nd chamber = 30% × 500 = 150
Plates to be provided in the 3rd chamber = 25% × 500 = 125

The **diffuser plates** are now adjusted as to ensure a minimum clear distance of 0.9 m along the channel length, to avoid interference from the rising streams of bubbles. The centre to centre

*See col. (12)—Table 9.10, it is between 40—100 m³/kg.

spacing of plates along channel length, will therefore be 1.2 m, and hence in a channel of 35 m length, we would be able to accomodate only about $\left[\dfrac{35}{1.2} \ (i.e. \text{ No. of spacings} - 1)\right] = 29 - 1 = 28$ plates. In order to provide a total of 225 plates in this length of 35 m, we will have to use $\dfrac{225}{28} = 8$ plates in each row, placed side by side along the width of the chamber. Hence, use 8 plates placed along the width of each chamber, constituting as one row, and provide 28 such rows @ 1.2 m c/c distance.

In the second chamber of 35 m length, we have to provide only 150 plates, and hence c/c spacing of rows (8 plates in each row)

$$= \dfrac{35}{\dfrac{150}{8}} = \dfrac{35 \times 8}{150} = 1.86 \text{ m}; \quad \text{say @ 1.8 m c/c.}$$

In the 3rd chamber, No. of plates required is only 125; and hence spacing of rows in this chamber $= \dfrac{35}{\dfrac{125}{8}} = \dfrac{35 \times 8}{125} = 2.24$ m; say 2.2 m.

Hence adopt 8 plates in each row; and rows are placed @ 1.2 m c/c in the 1st chamber, @ 1.8 m c/c in 2nd chamber; and @ 2.2 m c/c in the 3rd chamber. **Ans.**

Design of Secondary sedimentation Tank. Adopting a surface loading rate of 20 m³/day/m² at average flow of 6300 m³/day, we have

(i) Surface area required

$$= \dfrac{6300}{20} \text{ m}^2 = 315 \text{ m}^2$$

Adopting a solids loading of 125 kg/day/m² for MLSS of 2000 mg/l, we have

(ii) the surface area required

$$= \dfrac{6300 \times 2000}{1000} \times \dfrac{1}{125} \text{ m}^2 = 100.8 \text{ m}^2$$

The higher surface area of 315 m² is adopted. Adopting a circular tank,

$$\text{dia of tank} = \sqrt{\dfrac{315 \times 4}{\pi}} = 20 \text{ m.}$$

Weir loading for a circular weir placed along the periphery of the tank having length 20 π will be

$$= \dfrac{6300}{20\pi} \text{ m}^3/\text{day/m.} = 100.3 < 150; \text{ O.K.}$$

Note. If weir loading exceeds the permissible value, we may provide a trough instead of a single weir at the periphery.

Hence, provide 20 m dia secondary settling tank. **Ans.**

the qua... sludge will be sludge drying beds, the equation (9.48 a) as : calculated by using the equation (9.48 a) as :

$$\theta_c = \frac{V \cdot X_t}{Q_w \cdot X_R}$$

$$4.65d = \frac{6300 \frac{m^3}{d} \times 2000 \frac{gm}{m^3}}{Q_w \cdot X_R}$$

or

$$Q_w \cdot X_R = \frac{6300 \times 2000}{4.5} \; gm/d$$

or

$$= 2800 \; kg/d$$

For 10 kg/m³ S.S. concentrating in secondary sludge,

$$\text{excess secondary sludge volume} = \frac{2800 \; kg/d}{10 \; kg/m^3} = 280 \; m^3/d.$$

Note. This secondary sludge volume of 280 m³/d shall be taken to sludge drying beds, along with the primary sludge. The volume of primary sludge can be calculated if the concentration of suspended solids in sewage is known along with knowing the degree of removal of suspended solids in primary settling. Since S.S. of sewage is not given in this question, the quantity of primary sludge cannot be worked out ; and hence the design of sludge drying beds cannot to be done with the given data.

9.42. Advantages and Disadvantages of an Activated Sludge Plant

The chief advantage of the activated sludge process is that it offers secondary treatment with minimum area requirements, and an effluent of high quality is obtained. The conventional process was somewhat difficult to operate and needed a lot of supervision. However, the modifications described earlier, have made the process less difficult to operate than formerly, with the result that most secondary treatment plants being installed today are of these types.

Compared to the trickling filter plant, the capital cost of an activated sludge plant is less, but the operating cost is more mainly because of high power consumption for operating air compressors and the sludge circulation pumps. Generally, the power require- ment in an activated sludge plant varies between 55 to 110 H.P. per million litres of sewage.

Loss of head through the plant is also quite less, and there is no fly or odour nuisance here, as is there in a trickling filter plant. However, if there is a sudden increase in the volume of sewage or if there is a sudden change in the character of sewage, adverse effects are produced on the working of the process, producing inferior

effluent. Moreover, the quantity of sludge obtained is larger, and needs suitable thickening and disposal.

9.43. Activated Sludge Process Vs Trickling Filter Process and the Choice of One

As discussed earlier, the conventional biological or secondary treatment of sewage is usually carried out, either by using *trickling filters* or by using an *activated sludge process*. The basic difference between an activated sludge process and the action involved in a trickling filter is that ; whereas in a trickling filter, the bacterial film coating the grains of the filter media is *stationary* and likely to become clogged after sometime ; in the activated sludge process, on the other hand, the finer suspended organic particles of sewage (settled as activated sludge) are themselves coated with the bacterial film, which is kept *moving* by the constant agitation. In the activated sludge process, therefore, the sludge flocs are coated with bacteria, and they act like free moving organisms, which are being continuously swept through the sewage, and which in their search for food and work, oxidise the organic matter present in sewage *in a much more efficient way* than that carried out in a *filter* by the bacteria coated around the particles of filter media. *As such, it can be stated that an activated sludge process is more efficient than a trickling filter.*

The quality of the effluent obtained in a conventional activated sludge plant is also better than that of a trickling filter plant. But since a conventional activated sludge plant requires a lot of skilled attendance and supervision during its operation, the modified activated sludge processes are generally used these days (as discussed earlier).

Moreover, the modern plants are operated more rapidly at higher rates than those adopted in conventional plants, with shorter detention periods in the aeration tanks (2 to 3 hours), smaller amounts of air ($3.13 \text{ m}^3/\text{m}^3$ of sewage treated), and lesser amount of returned sludge (10 to 25% of sewage flow). Such a **high rate activated sludge plant*** can produce sufficiently good quality effluent (removing 70 to 80% of suspended solids, and 80 to 85% of BOD from sewage), with the results lying in between those obtained by a conventional activated sludge plant and a high rate trickling filter plant ; although the available finances and the technical development of a country is another major guiding factor.

In addition to giving a slightly better effluent, other **advantages offered by an activated sludge plant** are :

(i) lesser land area is required ;

*The sludge settled in secondary tanks in a high rate activated process is much dense than the conventional activated sludge, and therefore, settles much faster. The surface loading in secondary clarifiers can, therefore be increased to 50,000 $l/\text{m}^2/\text{day}$ or so. However, the sludge becomes septic too early, and must be immediately removed.

(ii) the head loss in the plant is quite low ;

(iii) there is no fly or odour nuisance ;

(iv) capital cost is less ; and

(v) greater flexibility of operation, permitting control on the quality of effluent, is possible.

The disadvantages of the activated plant, on the other hand, are also laudable, as :

(i) high cost of operation, with greater power consumption ;

(ii) a lot of machinery to be handled ;

(iii) the sudden change in the quantity and character of sewage may produce adverse effects on the working of the process, thus producing inferior effluent ;

(iv) bulking of sludge is a common trouble, which has to be controlled, especially when industrial wastewaters with high carbohydrate content or antiseptic properties are present. In any case, if such bulking is not there, comparatively larger sludge volume is to be handled ; and

(v) the quantity of returned sludge has to be adjusted every time, as and when there is a change in the quantity of sewage flow, thus making the operation a little cumbersome.

Since these advantages and disadvantages weigh against each other, it is therefore, suggested that before recommending any of these two methods, the following factors must be thoroughly studied.

(i) availability of land for installing the treatment ;

(ii) availability of suitable method of sludge disposal ;

(iii) cost and availability of power ;

(iv) availability and cost of machinery and its spare parts required for installing the plant ;

(v) degree of flexibility required in the operation of the process ;

(vi) the quality and quantity of sewage, and the chances of variation in its quality and quantity ; and wear and tear of civil works and machinery employed in the process.

Moreover, detailed estimates for both types of treatments should also be prepared for a particular project. The pros and cons of both methods for that project should also be reviewed and thoroughly considered. A final decision is then taken, keeping in consideration the economy as well as the comparative merits and demerits of each method, and their effects on the desired aims. Normally, *it is found that for towns or small cities with medium sized plants, trickling filters are better ; whereas in big cities with large sized plants, the activated sludge plant is better.*

SECONDARY TREATMENT THROUGH ROTATING BIOLOGICAL CONTACTORS
(Aerobic Attached Culture)

9.44. Rotating Biological Contactors (RBCs)

The Rotating Biological Contractor's process of secondary waste-water treatment has been recently developed and does not fit precisely in to either the trickling filter or the activated sludge categories, but does employ principle common to both of them.

A **rotating biological contactor** (RBC) is a cylinderical media made of closely mounted thin flat circular plastic sheets or discs of 3 to 3.5 m in diameter, 10 mm thick, and placed at 30 to 40 mm spacings mounted on a common shaft. Thinner materials can be used by sandwitching a corrugated sheet between two flat discs and welding them together as a unit, as shown in Fig. 9.48.
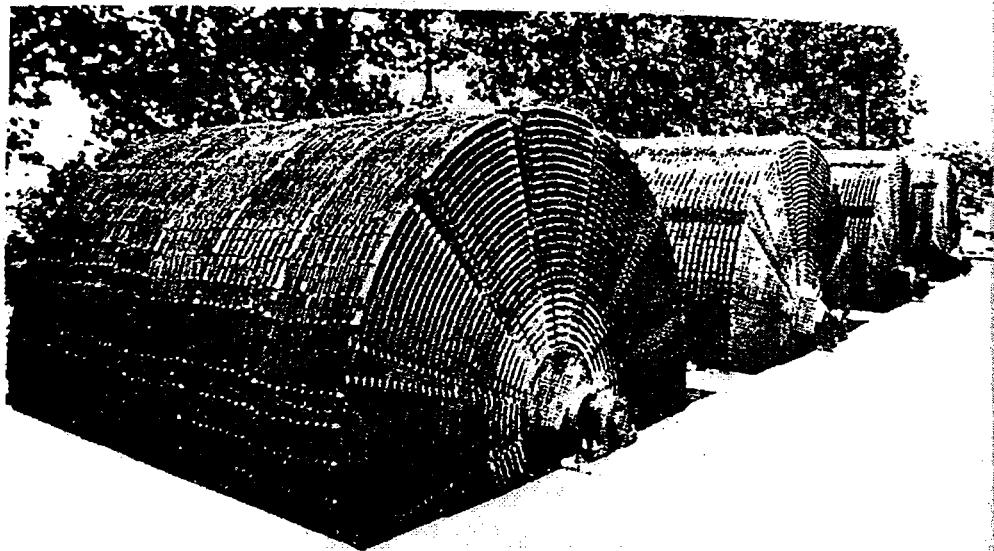

Fig. 9.48. Rotating Biological Contactors placed in series.

The R.B.C.'s are usually made in up to 8 m length, and may be placed in series or parallel in a specially constructed tank(s), through which the wastewater is allowed to pass. The RBC's are kept immersed in wastewater by about 40% of their diameter. The RBC's are rotated around their central horizontal shaft, at a speed of 1—2 rpm by means of power supplied to the shaft. Approximately 95% of the surface area is thus alternately immersed in the was-tewater and then exposed to the atmosphere above the liquid.

When the process is operated, the microorganisms of the waste-water begin to adhere to the rotating surfaces and grow there, untill the entire surface area of the discs gets covered with 1 to 3 mm layer of biological slime. As the discs rotate, they carry a *film of was-tewater* into the air, where it trickles down the surface of the discs,

absorbing oxygen. As the discs complete their rotation, this film mixes with the wastewater in the tank, adding to the oxygen of the tank and mixing the treated and partially treated wastewater. As the attached microorganisms pass through the tank, they absorb other organics for breakdown. The excess growth of microorganisms is sheared from the discs, as they move through the wastewater tank. The dislodged organisms are kept in suspension by the moving discs. This suspended growth finally moves down with the sewage flowing through the tank to a downstream settling tank for removal. The effluent obtained is of equal or even better quality than what is obtained from other secondary treatments. The quality of the effluent can further be improved by placing several contractors in series along the tank. The method can thus provide a high degree of treatment, including biological conversion of ammonia to nitrates.

As is evident, a given set of discs (*i.e.* an RBC) serves the following purposes :

(i) They provide media for build up of *attached microbial growth*.

(ii) They bring the growth of microbes in contact with the wastewater.

(iii) The aerate the wastewater and the suspended microbial growth in the wastewater tank.

In this process, *the attached growths are similar in concept to a trickling filter, except that here the microorganisms are passed through the wastewater, rather than the wastewater passing over the microbes, as happens in a trickling filter. This method realises some of the advantages of both the trickling filter and the activated sludge process.*

The sludge produced in the process contains about 95—98% moisture, and may amount to about 0.4 kg per kg of BOD_5 applied. The theoretical model of the process is similar to that for trickling filter, *but actual design is still empirical* and based on the data from the successful working plants and as developed by the process manufacturers.

The *hydraulic loading* rates may vary between 0.04—0.06 m/day, and *organic loading rates* between 0.05—0.06 kg BOD_5/m^2 per day, based upon the disc surface area. Sloughing of biological solids is more or less continuous and the effluent contains a relatively constant concentration. The solids settle well and clarifier surface overflow rates of about 33 m^3/m^2 per day are reported to be satisfactory.

AEROBIC STABILISATION UNITS
(Aerobic Suspended Culture)

9.45. Oxidation Ponds and Stabilisation Ponds

Stabilisation ponds are open flow-through earthen basins, specifically designed and constructed to treat sewage and biodegradable

The details of the pond are shown in Fig. 15.12, 15.13 and 15.14
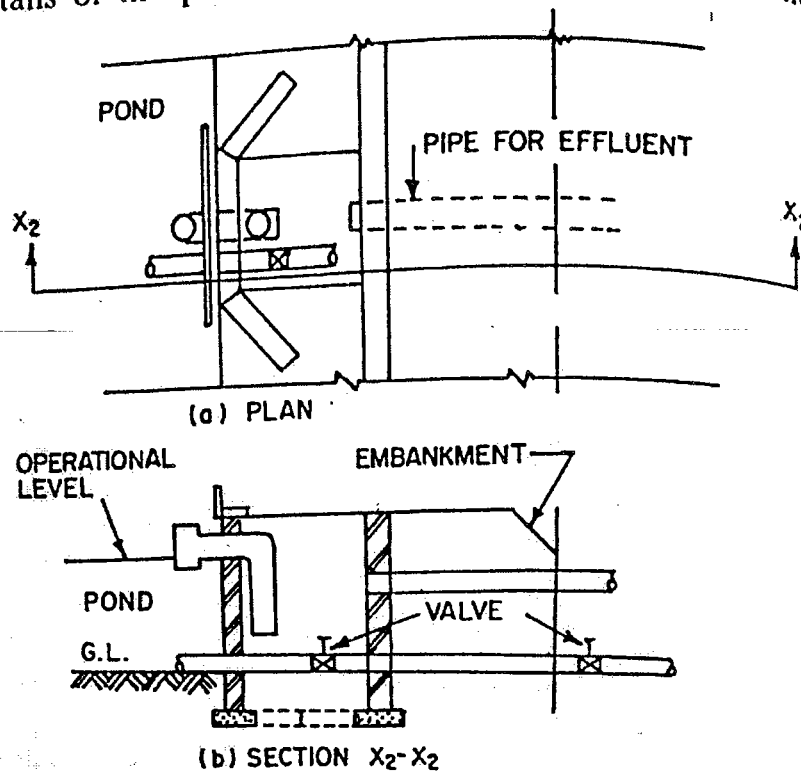

(a) PLAN

(b) SECTION X₂-X₂

FIG. 15.14. OUTLET ARRANGEMENT

15.6. AERATED LAGOONS

Stabilization ponds overloaded due to industrial wastes or reduced temperature under ice cover often produce odours and insufficient BOD removals. Aeration and mixing provide distribution of dissolved oxygen for decomposition of organic matter when oxygenation by algae and wind mixing are not sufficient. Such ponds are known as *aerated lagoons* which represent a system of treatment that is intermediate between oxidation ponds and activated sludge systems.

In contrast to oxidation ponds, where oxygen required for stabilizing the organic matter is furnished by algae, the oxygenation is provided in aerated lagons by mechanical surface aeraters installed on floats or rafts or on fixed platforms. In the aerated lagoons, the depth varies from 2.5 to 4 m, and the land requirements are much less because of the greater dephs and smaller detention times needed for the stabilisation of organic matter. Depending upto the extent of mixing, the lagoons may be classified as either (i) *complete mix* or (ii) *partially mixed*. These lagoon types are also often identified as *aerobic areated lagoons*, and *facultative aerated lagoons* respectively. In the complete mix type of aerobic lagoons, greater amount of aeration is provided to keep all the solids in suspension due to which the entire pond is aerobic. The partially mixed type or facultative aerated lagoons are operated at a low rate of aeration not adequate

to keep all the solids in suspension but enough to keep top layers aerobic. In a facultative aerated lagoons, the sewage solids tend to settle down and anaerobic bottom is established.

The complete mix aerated lagoons essentially consist of two units. In the first, the mechanical surface aerators are so designed that solids do not settle to the bottom of the tank, while the second unit is used as settling tank for the removal of suspended solids. As such, the process is similar to activated-sludge-process without recirculation. In such aerobic-flow-through type lagoons, SS concentration may be anywhere between 60 and 300 mg/l with detention times of 2 to 10 days. Oxygenation requirements are of the order of 0.7 to 1.3 kg per kg of BOD$_5$ removed and the power requirements vary from 0.75 to 2.25 kW per 1000 m^3. The BOD removals are only of the order of 75 to 85%, since the sludge solids are not allowed to accumulate and are carried along the effluent led to the sedimentation tank.

In the partially mixed system, a large portion of incoming solids and the biological solids produced within the lagoon settle to the bottom of the tank where anaerobic decomposition takes place. Therefore effluent from this type of tanks is more stable. In such facultative type lagoons, SS concentration varies from 30 to 150 mg/l with detention times ranging from 3 to 5 days. Oxygenation requirements are about 0.8 kg per kg of BOD$_5$ removed and the power requirements vary from 0.8 to 1.0 kW per 1000 m^3. The BOD removals are of the order of 75 to 90%, but there is need for removal of accumulated sludge after some years.

9.50. Imhoff Tanks

An Imhoff tank is an improvement over septic tank, in which the incoming sewage is not allowed to get mixed up with the sludge produced, and the outgoing effluent is not allowed to carry with it large amount of organic load, as in the case of a septic tank. An Imhoff tank is, infact, a two storeyed tank, and was designed by a German scientist Mr. Karl Imhoff. They are sometimes also known as *Two-storey Digestion Tanks*.

9.50.1. Constructional Details. The typical details of an Imhoff tank are shown in Fig. 9.59. It is a double chamber rectangular tank.

Fig. 9.59. Imhoff tank.

The upper chamber is called the *sedimentation chamber* or *flowing through chamber*, through which the sewage flows at a very low velocity; and the lower chamber is the *digestion chamber*, in which the sludge gets digested due to anaerobic decomposition.

The solids of the slowly moving sewage, settling down to the bottom of the sedimentation chamber, through the sloping bottom sides of the sedimentation chamber (slope being 1.25 vertical to 1 horizontal, as shown) will slide down into the digestion chamber through an *entrance slot* at the lowest point of a sedimentation chamber. The slot (with minimum width as 15 cm) is trapped or overlapped in such a way that the gases generated in the digestion chamber cannot enter the sedimentation chamber, and thus avoiding direct contact of sewage with the foul gases, and its consequent pollution.

A *Gas vent* also called *Scum chamber* is also provided above the digestion chamber and along side the sedimentation chamber to take care of the gases escaping to the surface. The chief gas is methane, having a considerable fuel value, and may, therefore, be separately collected for use.

In order to prevent the particles of sludge or scum from entering into the sedimentation chamber from the digestion chamber, the scum and sludge must be maintained at least 45 cm above and below the slots, respectively. This free or clear zone is called the *neutral zone*.

The digestion chamber (*i.e.* the lower chamber) is divided into a number of (three to four generally) interconnected compartments as shown. The bottom of each digestion compartment is made up in the form of an *inverted cone* or *hopper* with sides sloping 1 : 1, so as to concentrate the sludge at the bottom of the hopper. The digested sludge from the bottom of the hoppers is removed periodically (after 1 to 1½ months, depending upon the temperature of sludge) through the cast-iron de-sludging pipes provided in each compartment. The sludge is removed with flow under hydrostatic pressure of 1.2 to 1.8 m. Moreover, all the sludge is not removed, and only the bottom layers which are completely digested, are withdrawn, leaving some sludge to keep the tank seeded with anaerobic bacteria. The removed sludge may be dried and disposed of in a sanitary manner. Further, *in order to ensure uniform distribution of solids in the different hoppers, the flow of sewage in the sedimentation compartment above, is reversed intermittently.*

9.50.2. Design Considerations. In designing Imhoff tanks, the following important design points may be kept in mind :

(A) *Sedimentation Chamber.* It is rectangular in shape with the following specifications :

(i) Detention period = 2 to 4 hours (usually 2 hours)

(*ii*) Flowing through velocity

     = should not be more than 0.3 m/minute.

(*iii*) Surface loading = should not exceed 30,000 litres/m² of plan area/day.

     It may, however, be increased to about 45,000 l/m²/day for effluent coming from activated sludge plant, or where recirculation is adopted.

(*iv*) Length of the tank should preferably not exceed 30 m or so, as to provide good sludge distribution. Length to width ratio may vary between 3 to 5.

(*v*) Depth of this chamber should as far as possible be kept shallow, so as to permit sliding of the solids up to the slot before reaching the end of the sedimentation. In practice, a total depth of 9 to 11 m has been found sufficient for Imhoff tanks ; with the depth of sedimentation chamber as about 3 to 3.5 m or so. The free-board provided may be about 45 cm.

(*B*) *Digestion Chamber.* This chamber is generally designed for a minimum capacity of 57 litres per capita*. But in warmer climates, where shorter periods between sludge withdrawals are possible, it may be reduced to about 35 to 40 litres per capita.

(*C*) *Gas Vent or Scum Chamber.* The surface area of the scum chamber should be about 25 to 30 per cent of the area of the horizontal projection of the top of the digestion chamber. Sufficient area for escape of gases is necessary, so as to prevent troubles due to foaming. Moreover, the width of a vent should be 60 cm or more.

9.50.3. Advantages and Disadvantages of Imhoff Tanks. Imhoff tanks combine the advantages of both the septic as well as sedimentation tanks, and as such, they find use in case of small treatment plants requiring only primary treatment. They are quite economical, and do not require skilled attention during operations. The results obtained are quite good, with 60 to 65% removal of solids, and 30 to 40% removal of BOD. Moreover, there is no problem of sludge disposal, as in the case of sedimentation tanks. They suffer, however, from the following *drawbacks* :

(*i*) Depth of tank is more, which may make the constructions costlier, especially in hard rocks or quick sands. At such places, these tanks may, thus become uneconomical.

(*ii*) Imhoff tanks may give out offensive odours, when improperly operated.

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*This is larger than that provided in separate digestion tanks because of the lack of control on temperature of digestion (since there is no heating arrangement in Imhoff tanks), and inadequate mixing of the raw and the digested sludge (since there are no mechanical devices installed in Imhoff tanks for this purpose as is done in sludge digestion tanks).

(iii) They are unsuitable and do not function properly where sewage is highly acidic in character.

(iv) These tanks have a tendency to foam or boil. This may cause the scum to go up to the top of the tank, and it may also force the sludge particles to enter the sedimentation chamber through the slot. The foaming may, thus, adversely affect the quality of the effluent.

(v) There is no adequate control over their operation. This makes them unsuitable for use in large treatment plants, where separate sludge digestion tanks are preferred in addition to sedimentation tanks. Imhoff tanks are, therefore, useful only for small cities and institutions, where it is not possible or economical to install separate sludge digestion tanks. Mostly, however, they have become obsolete these days.

Example 9.37. *Design an Imhoff tank to treat the sewage from a small town with 30,000 population. The rate of sewage may be assumed as 150 litres per head per day. Make suitable assumptions, wherever needed.*

Solution.

Design of Sedimentation Chamber

The sewage discharge per day
$$= 30,000 \times 150 = 4.5 \text{ M. litres/day}$$
$$= 4500 \text{ cu-m/day}$$

Assuming a detention period of sewage in the sedimentation chamber as 2 hours, we have

The volume of sewage entering in two hours, *i.e.* the capacity of the sedimentation chamber
$$= 150 \times 30,000 \times \frac{2}{24} \text{ litres}$$
$$= 3,75,000 \text{ litres} = 375 \text{ cu-m.}$$

Assume an effective depth of 2.2 m (effective depth includes part of the bottom sloping walls of the chamber) and a width of 4.3 m (say).

The length of the sedimentation chamber
$$= \frac{375}{2.2 \times 4.3} = 39.64 \text{ m ; say 40 m.}$$

This length is too large for a single tank. So let us adopt two tank units, each of length 20 m and width 4.3 m ; then
$$\frac{L}{B} = \frac{20}{4.3} = 4.65$$

which is between 3 to 5 and, therefore, satisfactory.

Now, discharge passing through each unit

$$= \frac{1}{2} \text{ of the total discharge}$$

$$= \frac{4.5}{2} \text{ M. litres/day} = 2.25 \text{ M. litres/day}$$

Check for velocity

Length of tank   = Velocity × Detention time

∴   20 m = Velocity in m/min × (2 × 60 min.)

∴ Velocity in m/min

$$= \frac{20}{2 \times 60} = 0.17 \text{ m/min} < 0.3 \text{ m/min} ;$$

and, therefore, Safe.

Check for Surface Loading

Surface loading $= \dfrac{Q}{BL}$

$$= \frac{2.25 \times 10^6}{4.3 \times 20} = 26,162 \ l/m^2/day$$

which is less than 30,000 $l/m^2/day$; and, therefore, satisfactory. Hence, the dimensions chosen can be accepted.

Now let us decide the depth of the rectangular and sloping portions of the sedimentation chamber with its effective depth as 2.2 m.

With 4.3 m width and bottom sides sloping at 1 H : 1.25 V, the height of the sloping bottom = x = 1.25 × 2.15 = 2.69 m.

Now, with effective depth of 2.2 m, the height of the vertical portion below the liquid surface (y) is given by

$$y = 2.2 - \tfrac{1}{2}(2.69)^*$$
$$= 2.2 - 1.345$$
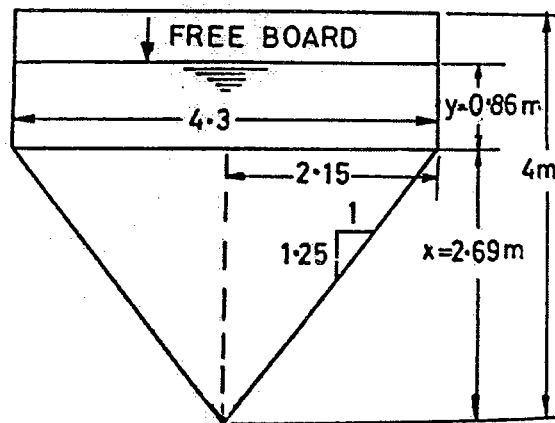$$= 0.855 ; \text{ say } 0.86 \text{ m.}$$

Fig. 9.60

Adding 0.45 m for the free board, the total depth of the sedimentation chamber up to bottom at the entrance of the slot

$$= 0.45 + 0.86 + 2.69 = 4.00 \text{ m.}$$

Design of Gas Vent and Neutral Zone

Provide a neutral zone of 0.45 m below this depth of 4 m. The tank, in general, is of 20 m length, but below this 4.00 m depth, it

*Effective depth of triangular portion will be half, to make it equivalent to a rectangular section.

shall be divided into a number of compartments, say 4, each of length

$$\frac{20}{4} = 5 \text{ m}.$$

The area of gas vent has now to be provided on both sides of the sedimentation chamber. This width should be about 25 to 30% of the total width of the tank. Using an overall width of 6.5 m, the total width of the gas vent (i/c both sides of sedimentation chamber), assuming 15 cm thickness of chamber walls

$$= 6.5 - 4.3 - 2 \times 0.15 = 1.9 \text{ m} ;$$

This is about $\frac{1.9}{6.5} \times 200 = 29.23\%$ of the total width, and therefore,

OK (between 25 to 30%). Hence, 0.95 m width of gas vent will be provided on either side of the sedimentation chamber.

Design of Digestion Chamber

Assuming the capacity of the digestion chamber @ 40 litres/capita, we have

The capacity of the digestion chamber
$$= 30,000 \times 40$$
$$= 12 \times 10^5 \text{ litres} = 1200 \text{ cu-m.}$$

Now, considering four compartments or units per tank (8 units in both tanks with 6.5 m width), we have

The capacity of each unit or compartment
$$= \frac{1200}{8} = 150 \text{ cu-m.}$$

Now, assume the depth of each hopper as 2.1 m, side slopes 1 : 1, and bottom section as
$$(6.5 - 2 \times 2.1 = 2.3 \text{ m}) \times (5 - 2 \times 2.1 = 0.8)$$

Capacity of each hopper

$$= \frac{h}{3} \left[A_1 + A_2 + \sqrt{A_1 A_2} \right]$$

where h = 2.1 m
A_1 = 6.5 × 5 m
= 32.5 m²
A_2 = 2.3 m × 0.8 m
= 1.84 m²

[See Fig. 9.61]

∴ Capacity of each hopper

$$= \frac{2.1}{3} \left[32.5 + 1.84 + \sqrt{32.5 \times 1.84} \right]$$

$$= 0.7[32.5 + 1.84 + 7.73]$$

$$= 0.7[42.07] = 29.45 \text{ m}^3.$$

Balance capacity to be provided by rectangular portion of section 6.5 m × 5 m

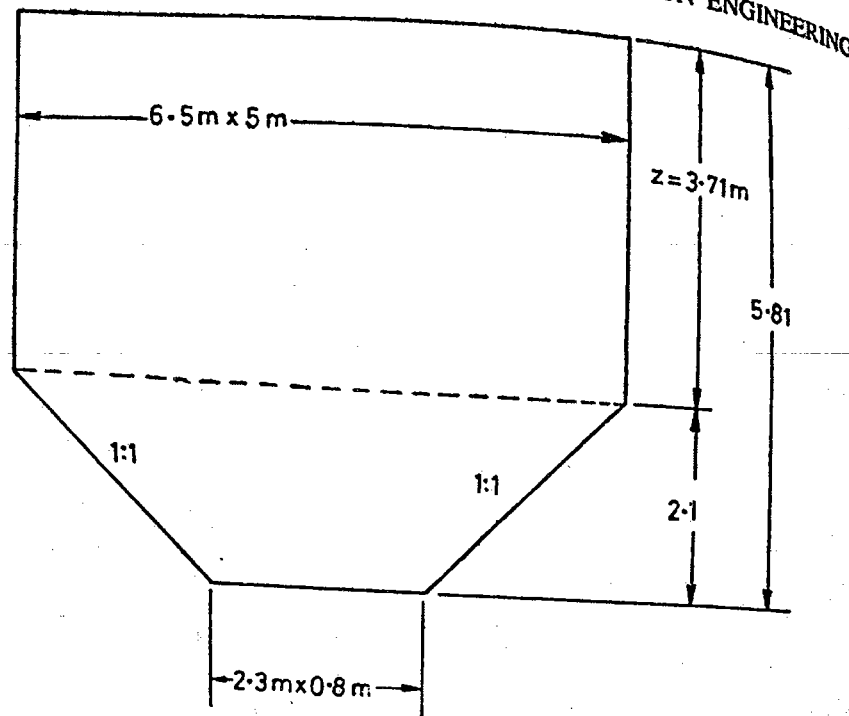$$= 150 - 29.45 = 120.55 \text{ m}^3.$$

Fig. 9.61

∴ Height of this portion

$$= z = \frac{120.55}{6.5 \times 5} = 3.71 \text{ m.}$$

∴ Total height of digestion chamber including neutral zone

$$= 0.45 + 3.71 + 2.1 = 6.26 \text{ m}$$

Total height of tank from top to bottom

= Height of sedimentation chamber
+ Height of sludge chamber
= 4 + 6.26 = 10.26 m.

This height is well within the practical limits (of 9 to 11 m) and hence the design is O.K. The plan, L-section and cross-section of the tank with these dimensions have been shown in Fig. 9.59. **Ans.**

Example 9.38. *Design an Imhoff-tank to treat the sewage from a small town with 30,000 population, assuming that the suspended solids in the influent sewage are 200 ppm. Water content of sludge is 97 per cent. Design the tank for three months sludge storage. Rate of sewage is 135 litres per head per day.*　　　(Engg. Services, 1968)

Solution. *Design of Sedimentation Chamber*

The sewage discharge per day = 30,000 × 135 litres
= 4.05 M-litres/day = 4050 cu. m/day

Assuming a detention period for sewage in the sedimentation chamber as 2 hours, we have

The volume of sewage entering in two hours, *i.e.* the capacity of the sedimentation chamber

9.52.4. Fluidized and Expanded Bed Reactors.

A *fluidized bed reactor* as well as an *expanded bed reactor* [Fig. 9.65 (d)] are both characterised by the presence of *mobile packing material*, such as sand, clay, coal, etc. The organics and the microbes in the reactor get attached to these particles, with which the reactor tank is filled only to a part height. The wastewater is entered from the bottom of the tank with an upflow velocity to move (fluidize) the media particles, which act as *mobile biomass carriers*, causing digestion of the organics present in the wastewater.

In a fluidized bed reactor, the overflow velocity is kept higher, causing the entire bed material to move with the up moving sewage, thereby distributing the sludge, virtually almost over the entire reactor volume. Whereas, in an expanded bed reactor, the upflow velocity is kept lower, thereby confining the sludge mostly in the lower part of the reactor. The expanded bed reactors, therefore, do not aim at complete fluidization of the bed, permitting the use of lower upflow pressure on the wastewater, thereby cosuming lesser electrical power. These reactors can be used for treating industrial as well as municipal wastewaters.

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*The organic matter in wastewater can be expressed in terms of BOD or COD. In anaerobic treatment systems, however, the COD value is finding greater usage, which lends itself directly to mass balance calculations. Reduction in COD for municipal wastewaters would normally correspond to approximately equivalent amount of reduction in ultimate BOD, since non-biodegradable solids are negligible in such wastewater.

SRT, which is a more rational design parameter, is however, difficult to be calculated for anaerobic reactors. For AC and UASB, it ranges between 15 to 30 days; while for other systems, it is estimated to be about 100 days or more giving them greater operational stability.

(d) Volume after drying

Mass of total solids = 520 kg (as before)

Moisture content after drying = 10%

$\therefore$ Total mass of sludge = 520/0.9 = 577.8 kg

Hence mass of water in sludge = 577.8 − 520 = 57.8 kg

$\therefore$ Volume of water = 57.8 l

Hence total volume = 57.8 + 266.7 + 96 = **420.5** l

(e) After incineration

After incineration, only non-volatile (i.e. fixed) solids remain. Total mass of non-volatile solids is 240 kg and its volume is 96 l. Hence volume after incineration is **96** l

Summary: In terms of percentage of original volume, the volumes computed above are (a) 100%, (b) 32% (c) 7.9% (d) 2.1% and (e) 0.5%, respectively.

16.4. SLUDGE THICKENING OR CONCENTRATION

Thickening is a procedure used to increase the solids content of sludge by removing a portion of the liquid fraction. This volume reduction, obtained by sludge thickening or sludge concentration, is done for the following purposes: (i) to permit increased loadings to sludge digesters (ii) to increase feed solids concentration to vacuum filters, (iii) to economise on transport costs as in ocean barging in case of raw sludges, (iv) to minimise the land requirements as well as handling costs when digested sludge has to be transported to disposal site, and (v) to a save on the auxiliary fuel that may otherwise be needed when incineration of sludge is practised. As an example, if waste activated sludge, which is typically pumped from secondary settling tanks with a content of 0.8 percent solids can be thickened to a content of 4 percent solids, then a five fold decrease in sludge volume is achieved, as is evident from Eq. 16.4, where $P_1/P_2 = 0.8/4 = 1/5$.

Sludge thickening is commonly achieved by the following three methods :

(i) Gravity thickening

(ii) Air flotation

and (iii) Centrifugation

1. **Gravity thickening.** Gravity thickening is the most common practice for concentration of sludges. Gravity thickening is the simplest and the least expensive. This is adopted for primary sludge or combined primary and activated sludge but is not successful in dealing with activated sludge independently. Gravity thickening of combined sludge is not effective when the activated sludge exceeds 40% of the total sludge weight, and other methods of thickening of activated sludge have to be considered.

Fig. 16.2 shows a typical waste sludge thickener. The tank resembles a circular clarifier except that the depth/diameter ratio is greater and the hoppered bottom has a steeper slope. Dilute sludge is fed to a center feed well. The feed sludge is allowed to settle and compact, and the thickened sludge is withdrawn from the bottom of the tank. A bridge fastened to the tank walls supports a truss-type scraper arm mounted on a pipe shaft equipped with a power lift device thereby opening up channels for water to escape and promoting densification. Also, the slow motion of the scraper dislodges gas bubbles, prevents bridging of solids and moves the sludge towards a central well for withdrawal. The continous supernatant flow that results is returned to the primary settling tank. Feed is provided continuously while the under flow may be extracted intermittently for further processing.
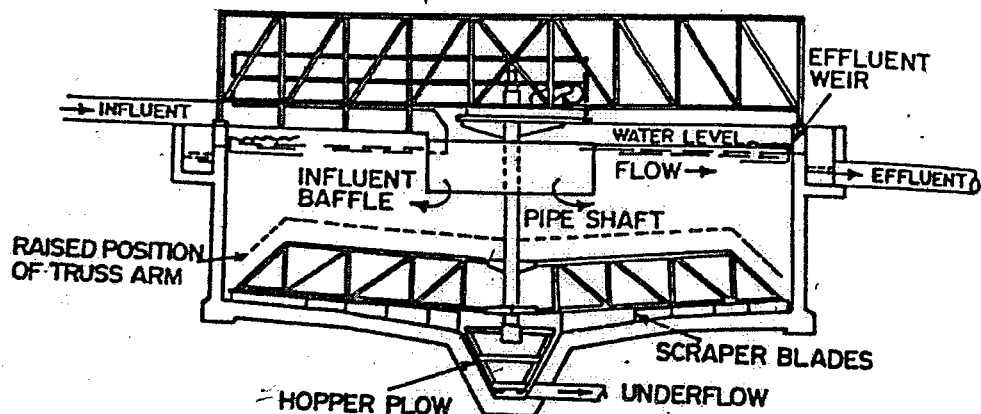

FIG. 16.2. GRAVITY SLUDGE THICKENER

Gravity thickeners are either 'continuous flow' or 'fill and draw type'. Continuous flow tanks are designed for a hydraulic loading likely to given rise to odour problems. The normal sludges contain too much with the plant effluent. The recommended solids loading for primary, and activated-sludge (60 : 40 weight ratio) are 100, 60 and 40 kg/day/m^2 respectively. Underflow solids concentration that can be expected with these loadings are about 8 to 10% with primary, 6 to 8% with primary and trickling filters and 2 to 6 % with primary and activated sludge. Better efficiency can be obtained by providing slow revolving stirrers, particularly with gassy sludges.

The continuous flow type circular thickeners have a side water depth of 3 m. Three settling zones occur in a thickener : (i) clear supernatant on top, (ii) feed zone characterised by hindered settling and (iii) Compression zone near the bottom where consolidation occurs. Settling data may be collected from batch type laboratory tests conducted on 200 mm dia. cylinder with initial height of atleast 900 mm. Evaluating the performance of a thickener often involves mass-balance calculations.

In operation, a *sludge blanket* is maintained on the bottom of the thickener to aid in concentrating the sludge. Concentration of the underflow solids is governed by the depth of sludge blanket upto 1m, beyond which there is very little influence of the blanket. Underflow solids concentration is increased with increased sludge detention time, 24 hours being required to achieve maximum compaction. Sludge blanket depths may be varied with fluctuation in solids production to achieve good compaction. During peak conditions, lesser detention times will have to be adopted to keep the sludge blanket depth sufficiently below the overflow weirs to prevent excessive solids carryover.

2. **Floatation thickening.** *Air floatation units* employ floatation of sludge by air under pressure or vacuum. The former process more commonly used, is known as *dissolved air floatation* or *pressure type floatation*. Floatation thickeners offer significant advantages in thickening light sludges such as waste activated sludges which have a density very close to that of water and are thus readily buoyed to the surface.

In the dissolved-air floatation unit (or pressure type floatation unit), a portion of subnatant is pressured from 3 to 5 kg per cm² and then saturated with air in the *pressure tank* or *air dissolution tank*. The effluent from the pressure tank is mixed with influent sludge immediately before it is released into the floatation tank. Excess dissolved air then rises up in the form of minute bubbles (with dia. upto 80 μ m) at atmospheric pressure attaching themselves to particles which form the sludge blanket. Thickened blanket (of
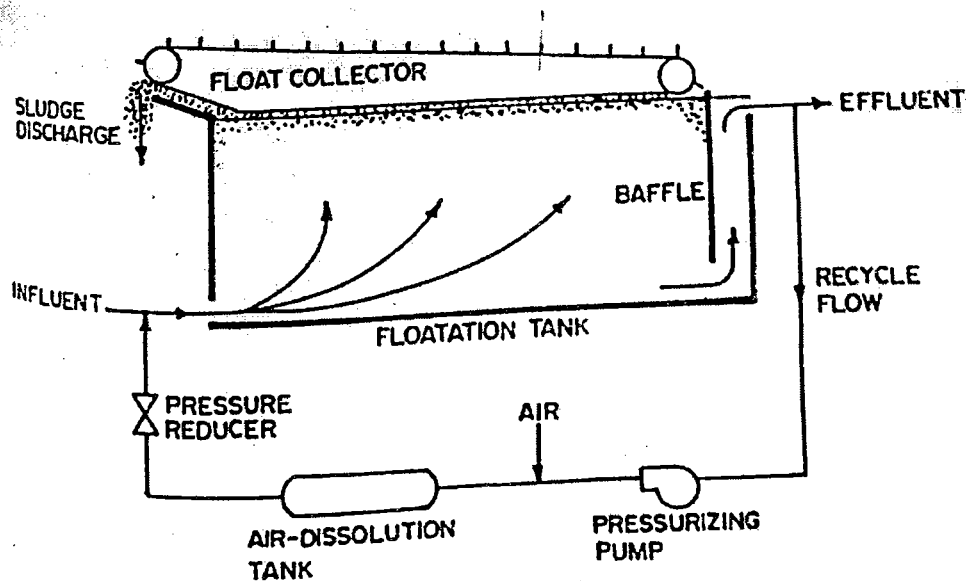

FIG. 16.3. SCHEMATIC DIAGRAM OF A DISSOLVED-AIR FLOATATION SYSTEM

thickness varying from 0.2 m to 0.6 m) is skimmed off from the surface, while the unrecycled subnatant is returned to the plant. Effluent is recycled at a rate of 30–150% of the influent flow through the air-dissolution tank to the feed inlet. Recycle ratio is interrelated to with (i) feed solids concentration, (ii) detention time, and (iii) air/solids ratio. Detention time in the floatation unit is not critical, provided that particles rise rapidly enough and the horizontal velocity does not scour the bottom of the sludge blanket. An air/solids ratio of 0.01 to 0.03 is sufficient to achieve acceptable thickening of waste-activated sludge. Surface overflow rates in floatation thickeners vary from 10 to 45 m/day at retention time of 30 min. to 1 hour. The efficiency of air floatation units is increased by the addition of chemicals like alum and polyelectrolytes. The addition of polyectrolytes does not increase the solids concentration but improves the solids capture from 90 to 98%.

3. **Centrifugal thickening.** Centrifuges are used both to thicken and to dewater sludges. Their application in thickening is normally limited to waste activated sludge. Thickening by centrifugation in-volves the settling of sluge particles under the influence of centrifugal forces. Thickening by centifugation is resorted to only when the space limitation or sludge characteristics will not permit the adoption of the other two methods mentioned above. This method involves high maintenance and power costs. The three basic types of centrifuges currently available for sludge thickening are (i) nozzle disc, (ii) solids bowl and (iii) basket centrifuges. Disc centrifuges are prone to clogging while the latter types gives poorer quality effluent.

Example 16.5. Design of gravity thickener

Design a gravity thickener for thickening the combined primary and activated sludge from a treatment plant for 200,000 population.

Solution

Let us assume per capita settleable solids in primary sludge as 54 gm/day and per capita settleable SS in activated sludge as 31 gm/day, making a total SS as 54 + 31 = 85 gm/day (Table 16.1)

$$\therefore \text{Weight of combined sludge} = (85 \times 10^{-3}) \times 200,000$$
$$= 17000 \text{ kg/day}$$

Recommended average surface loading = 40 kg/m²/day

Hydraulic loading required = 25 m³/m²/day

Let us assume specific gravity of wet mixed sludge = 1.008 and solids in combined sludge as 3%.

The volume of wet sludge/day is given by Eq. 16.3.

$$V_{sl} = \frac{W_s}{\rho_w \cdot S_{sl} \, p_s}$$

Solution. Using Eqn. (9.38), we have

$$V = V_1 \left[\frac{100 - p_1}{100 - p} \right]$$

$$V = V_1 \left[\frac{100 - 95}{100 - 90} \right]$$

or

$$= V_1 \times \frac{5}{10}$$

$$= \frac{V_1}{2}$$

Thus, the volume at 90% moisture will be half of that at 95% moisture. Hence, the percentage decrease in moisture will be 50%. **Ans.**

9.26. Sludge Digestion Process

As pointed out earlier, the sludge withdrawn from the sedimentation basins contains a lot of putrescible organic matter, and if disposed of without any treatment, the organic matter may decompose, producing foul gases and a lot of nuisance, pollution, and health hazards. In order to avoid such pollutions, the sludge is, first of all, *stabilised by decomposing the organic matter under controlled anaerobic conditions*, and then disposed of suitably after drying on drying beds, etc. The process of stabilisation is called the *sludge digestion* ; and the tank where the process is carried out is called the *sludge digestion tank*. In a sludge digestion process, the sludge gets broken into the following three forms :

(i) **Digested sludge.** It is a stable humus like solid matter, tary black in colour, and with reduced moisture content, and, is therefore, having reduced volume (about $\frac{1}{3}$ times the undigested sludge volume). Moreover, the quality of digested sludge is much better than that of the undigested sludge, and it is free of pathogenic bacteria which are killed in the digestion process. It may still, however, contain cysts and eggs of bacteria, protozoa and worms.

(ii) **Supernatant liquor.** It includes the liquified and finely divided solid matter, and is having high BOD (about 3000 ppm).

(iii) **Gases of decomposition.** Gases like methane (65 to 70%), carbon dioxide (30%), and traces of other inert gases like nitrogen, hydrogen sulphide, etc. are evolved. They may be collected (particularly the methane which has a high calorific value) and used as fuel.

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*50 to 60% of the organic solids are converted by bacteria into carbon dioxide and methane gases. The organic matter which remains, is chemically stable and practically odourless, and contains 90 to 95% moisture.

The sludge gas, having 70% methane, has a fuel value of about 5800 kilo calorie/cu. m (*i.e.* 650 Btu per cu. ft. app.). The amount of gas produced, on an average, is about 0.9 cu. m. per kg of volatile solids reduced in digestion. The gas produced thus varies with the sewage produced, and works out to about 14 to 18 litres per capita per day (usually 17 *l/c/*d).

The digested sludge is *dewatered, dried up, and* used as fertiliser ; while the gases produced are also used for fuel or for driving gas engines. The supernatant liquor contains about 1500 to 3000 ppm of suspended solids ; and is, therefore, re-treated at the treatment plant along with the raw sewage.

9.27. Stages in the Sludge Digestion Process

Three distinct stages have been found to occur in the biological action involved in the natural process of sludge digestion. These stages are :

(*i*) Acid fermentation ;

(*ii*) Acid regression ; and

(*iii*) Alkaline fermentation.

These stages are briefly summarised here :

(*i*) **Acid Fermentation Stage or Acid Production Stage.** In this first stage of sludge digestion, the fresh sewage-sludge begins to be acted upon by anaerobic and facultative bacteria, called acid formers. These organisms solubilize the organic solids through *hydrolysis*. The soluble products are then fermented to volatile acids and organic alcohols of low molecular weight like propionic acid, acetic acid, etc. Gases like methane, carbon dioxide and hydrogen sulphide are also evolved. *Intensive acid production makes the sludge highly acidic, and lowers the pH value to less than 6. Highly putrefactive odours are evolved during this stage, which continues for about 15 days or so* (at about 21°C). BOD of the sludge increases to some extent, during this stage.

(*ii*) **Acid-Regression Stage.** In this *intermediate stage,* the volatile organic acids and nitrogenous compounds of the first stage, are attacked by the bacteria, so as to form acid carbonates and ammonia compounds. Small amounts of hydrogen sulphide and carbon-dioxide gases are also given off. The decomposed sludge has a very offensive odour, and its pH value rises a little, and to be about 6.8. The decomposed sludge, also, entraps the gases of decomposition, becomes foamy, and rises to the surface to form scum. *This stage continues for a period of about 3 months or so* (at about 21°C). BOD of the sludge remains high even during this stage.

...(i) **Alkaline Fermentation Stage.** In this *final stage* of sludge ...ion, more resistant materials like proteins and organic acids ...acked and broken up by anaerobic bacteria, called **methane** ...ters, into simple substances like ammonia, organic acids and ...During this stage, *the liquid separates out from the solids, and* ...*digested sludge is formed.* This sludge is granular and stable, ...does not give offensive odours. (It has a musty earthy odour). ...digested sludge is collected at the bottom of the digestion tank, ...is also called **ripened sludge.** Digested sludge is alkaline in ...ure. The pH value during this stage rises to a little above 7 (about ...or so) in the alkaline range. *Large volumes of methane gas* ...ing a considerable fuel value) along with small amount of carbon* ...xide and nitrogen, are evolved during this stage. This stage* ...ends for a period of about one month or so (at about 21°C). The* ...D of the sludge also rapidly falls down during this stage.*

It is, thus, seen that several months (about $4\frac{1}{2}$ months or so) are ...quired for the complete process of digestion to take place under ...dural uncontrolled conditions at about 21°C. This period of diges- ...on is, however, very much dependent upon the temperature of ...gestion, and other factors. If these factors are controlled, quicker ...d effective digestion can be brought about, as discussed below.

28. Factors Affecting Sludge Digestion and Their Control

...important factors which affect the process of sludge digestion, ...d are, therefore, controlled in a digestion tank, are :

1. *Temperature ;*
2. *pH value ;*
3. *Seeding with digested sludge ; and*
4. *Mixing and stirring of the raw sludge with digested sludge.*

Besides these important factors, certain other minor conditions ...e quality of water supply ; presence of copper, fluorides, and radio ...tive substances, etc., may also affect the rate of digestion, but not ...any appreciable extent. These important factors which are largely ...ponsible for controlling the rate and effectiveness of sludge-diges- ...on are discussed below :

(1) **Temperature.** The process of digestion is greatly influenced ...temperature ; *rate of digestion being more at higher temperatures* ...d *vice-versa.* The effect of temperature on digestion period is ...own in Fig. 9.27. In this figure, two distinct temperature zones ...e indicated ; *i.e.*

(i) *Zone of Thermophilic Digestion.* In this zone of high tempera- ...re, digestion is brought about by heat loving *thermophilic*
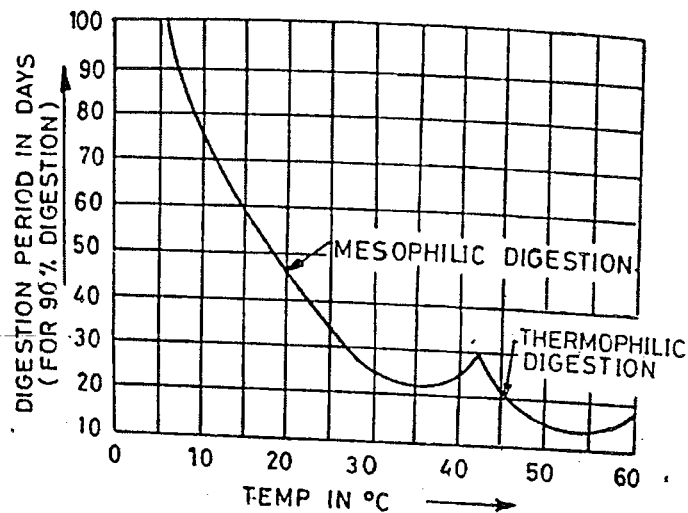
Fig. 9.27. Showing effect of temperature on sludge digestion period.

organisms. The temperature in this zone ranges between 40 to 60°C. The optimum temperature in this zone is about 54°C, and at this temperature, the digestion period can be brought down to about 10—15 days only. However, thermophilic range temperatures are generally not employed for digesting sewage sludge, owning to odours and other operational difficulties.

(ii) *Zone of Mesophilic Digestion.* In this zone of moderate temperature, digestion is brought about by common *mesophilic* organisms. The temperature in this zone ranges between 25 to 40°C. The optimum *mesophilic temperature is about 29°C ; and at this temperature, the digestion period can be brought down to about 30 days.*

Hence, it can be concluded that the sludge can be quickly digested, if the temperature in the digestion tank is kept high. The best results are obtained at about 29°C (i.e. the optimum mesophilic temperature) when about 90% of digestion takes place in about 30 days. But it may, however, be pointed out that it is difficult to control temperature in practice, as it mainly depends upon the prevailing local climatic conditions. Although external heating devices may sometimes be employed to control temperature in the digestion tanks, especially in cold countries.

(2) pH Value. It was pointed out earlier that during the digestion process, a lot of volatile organic acids are formed, as an intermediate step, in the breakdown of organic material. These volatile acids are then converted into methane gas by a specialised group of strictly anaerobic and slow growing bacteria, called methane formers. If the methane formers are not operating properly, an accumulation of volatile acids may occur, causing the pH to drop to a value as low as 5.0, *which will suppress further bacterial action.* Hence, during

... care must be taken to keep the acidity well under control, ... the pH during the digester start-up does not go below 6.5 or ... thus to see that alkaline conditions (with optimum pH about ... may prevail ultimately, in the final stage of digestion.

... acidity increases, (i) with the overdosing of raw sludge ; (ii) ... the over withdrawal of digested sludge ; and (iii) with the ... admission of industrial wastes. The remedy in such cases is ... hydrated lime in doses of 2.3 to 4.5 kg. per 1000 persons to ... raw sludge. The weight of raw sludge to be added daily, for the ... tenance of optimum value of pH, should also be limited to 3 to ... cent of the weight of the digested sludge removed.

Seeding with the Digested Sludge. When a sludge digestion ... is first put in operation, it is highly beneficial to *seed* it with ... digested sludge from another tank. Without seeding, it may take ... months to get a tank operating properly. Proper seeding will ... attain quick balance conditions of reaction.

Mixing and Stirring of the Raw Sludge with the Digested ... Incoming fresh raw sludge should be thoroughly mixed with ... digested sludge, by some effective method of agitation, so as to ... a homogenous mass of raw as well as digested (or partly ...) sludge. In this way, the bacterial enzymes present in the ... sludge will get every opportunity to get mixed with the raw ... , and to attack it for subsequent decomposition.

... mixing of raw and digested sludge achieved by stirring the ... in the sludge digestion tank by slow moving mechanical ... ; or the gases of decomposition may be used to set up ... by circulating from bottom to top of the tank and *vice versa*, ... means of a pumping device.

... Excessive stirring may produce harmful effects, as it may kill the ... teria. The proper stirring however, results in even distribution ... incoming sludge, breaks and reduces the scum, and helps in ... easing the production of gases.

... In cold countries, where it is necessary to heat the digestion ... nks, so as to maintain optimum mesophilic temperature (about ... C), the stirring may help in transmitting heat from the heating ... ils to the tank contents ; and thus to attain uniform temperature ... roughout the tank.

9. Sludge Digestion Tank or Digestors
(Aerobic Suspended Culture)

9.1. Constructional Details. A typical sludge digestion tank ... shown in Fig. 9.28, It consists of a circular R.C.C. tank with ... ppered bottom, and having a fixed or a floating type of roof* over
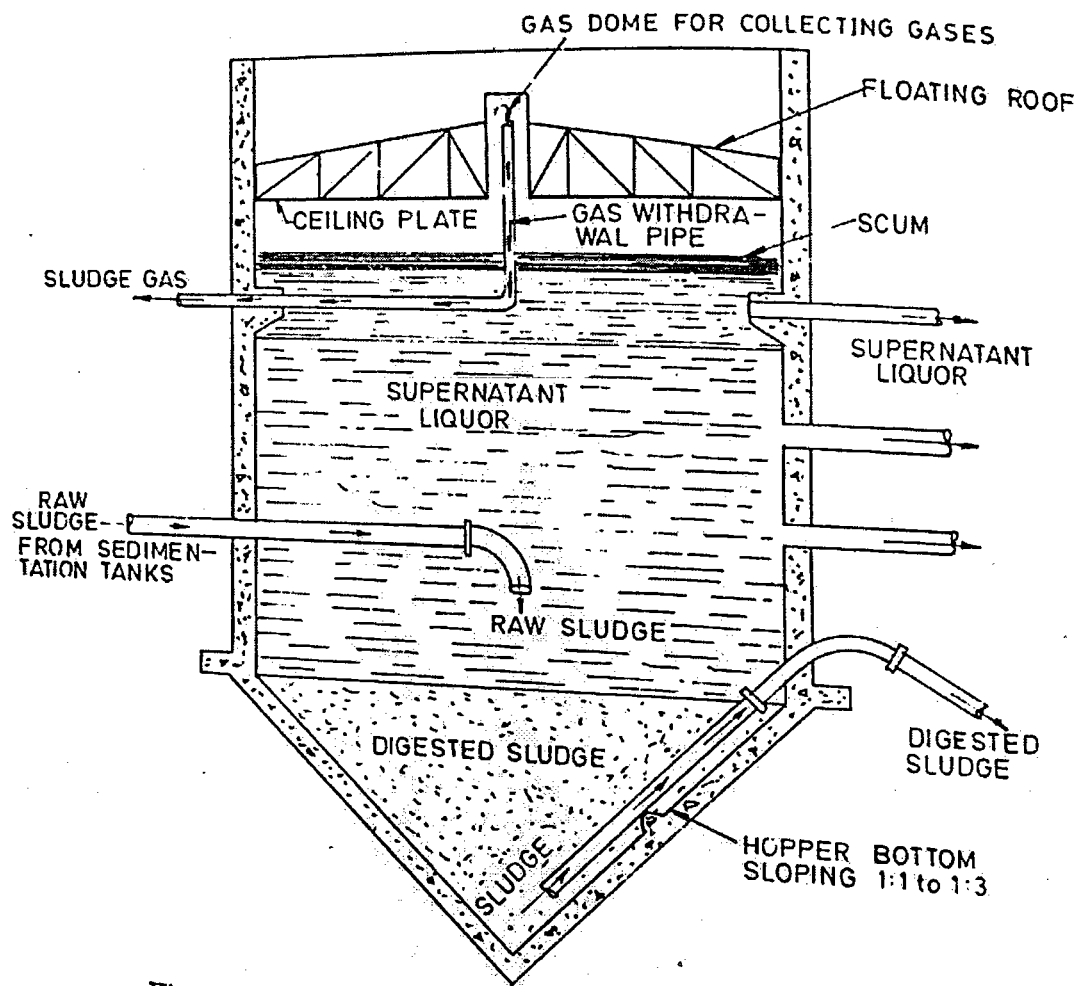
Fig. 9.28. Cross-section of a typical sludge digestion tank.

its top. The raw sludge is pumped into the tank, and when the tank is first put into operation, it is seeded with the digested sludge from another tank, as pointed out earlier. A screw pump with an arrangement for circulating the sludge from bottom to top of the tank or vice versa (by reversing the direction of rotation of the screw) is commonly used, for stirring the sludge. Sometimes, power driven mechanical devices may be used for stirring the sludge, although these are not very popular, at present.

In cold countries, the tank may have to be provided with heating

*Floating type of roof rises or falls with the pressure of gas, and it keeps out air successively. It may also be enlarged to work as a gas holder. Such type of roof is useful for small installations.

Fixed type of roof is used when separate gas holders are provided. The arrangement in this case should be so made that when the working level of the tank falls due to the withdrawal of supernatant liquor or the digested sludge, the gas from gas holders returns to the tank. Such an arrangement will ensure positive pressure of gas inside the digestion tank. Such type of roofs are used for larger plants.

...through which hot water is circulated in order that the ...temperature inside the tank is maintained at optimum digestion temperature level.

The gases of decomposition (chiefly methane and carbon dioxide) are collected in a gas dome* (in smaller tanks) or collected separate-ly in gas holders (in larger tanks) for subsequent use. The digested sludge which settles down to the hoppered bottom of the tank is removed under hydrostatic pressure, periodically, once a week or so. The supernatant liquor lying between the sludge and the scum is removed at suitable elevations, through a number of withdrawal pipes, as shown. The supernatant liquor, being higher in BOD and suspended solids contents, is sent back for treatment along with the raw sewage in the treatment plant. The scum formed at the top surface of the supernatant liquor is broken by the recirculating flow or through the mechanical rakers called *scum breakers*.

9.2. Design Considerations. The digestion tanks are cylindri-cal shaped tanks (*i.e.* circular in plan) with *dia* ranging between 3 to 12 m. The bottom hoppered floor of the tank is given a slope of about 1 : 1 to 1 : 3 (*i.e.* 1 H : 3 V). However, when the sludge is moved to the outlet by means of some mechanical equipment, the bottom slopes may be made relatively flat.

The *depth of the digestion tank* is usually kept at about 6 m or so. Deeper tanks are costlier, though more effective. Except in very large plants, it is usual not to provide more than 2 units.

The *capacity of the digestion tank* is a function of sludge produc-tion, digestion period, degree of digestion required, loss of moisture, and conversion of organic matter. If the progress of sludge digestion is assumed to be linear, then the capacity of the digestion tank is given as :

$$= \left(\frac{V_1 + V_2}{2} \right) t \qquad \qquad ...(9.39)$$

where V = Vol. of the digestion, m^3

$\qquad V_1$ = Raw sludge added per day, m^3/d

$\qquad V_2$ = Equivalent digested sludge produced per day on comple-tion of digestion, m^3/d

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*A gas dome is made of suitable metal, and is cylindrical in shape. It is fixed to the roof of the digestion tank, along with various accessories such as, gas meter, pressure relief valve, etc., Gas is taken off from gas dome, and is then stored in gas holders.

which exert increased demand of chemicals in conditioning are removed in the process. There are three methods of elutriation : (i) single stage, (ii) multi-stage and (iii) counter current washing. The water requirement is dependent upon the method used. For a given alkalinity reduction, single stage elutriation requires 2.5 times as much water as the two stage and 5 times as much water as counter-current washing. Hence single stage washing is used only in small plants. Counter current washing although higher in initial cost, in adopted in all large plants. Water requirement also depends on alkalinity of dilution water, alkalinity of sludge and desired alkalinity of elutriated sludge. Sludge and water are mixed in a chamber with mechanical mixing arrangement, the detention period being about 20 seconds. The sludge is then settled in settling tanks and excess water decanted. A maximum surface loading on settling tank of about 40 m^3/m^2/day and a detention period of about 4 hours are adopted.

Counter current elutriation is generally carried out in twin tanks sim ar to sedimentation tanks, in which sludge and wash water enter at opposite ends. Piping and channels are so arranged that wash water entering the second stage tank comes first in contact with sludge already washed in the first stage tank. The volume of wash water required is roughly 2 to 3 times the volume of sludge elutriated.

16.8. SLUDGE DEWATERING

Dewatering is a physical unit operation used to reduce the moisture content of the sludge, and thus to increase the solids concentration. Dewatering is accomplished either by *air drying* in *sludge drying beds* or by mechanical means such as vacuum filtration, centrifugation, pressure filtration etc.

Purposes of dewatering

1. Cost of trucking sludge to ultimate disposal site is reduced, because of reduced sludge volume consequent to dewatering.
2. Ease in handling dewatered sludge.
3. Increase in calorific value of sludge by removal of moisture, prior to incineration.
4. Rendering the sludge totally odourless and nonputrescible.
5. Sludge dewatering is commonly required prior to land filling to reduce lechate production at landfill site.

Sludge drying beds. This method of dewatering and drying the sludge is specially suitable for those locations where temperature are higher, similar to the one prevailing in our county. The method consists of applying the sludge on specially prepared open beds of land. A typical section of sludge drying beds is shown in Fig. 16.13. A sludge drying bed usually consists of a bottom layer of gravel

of uniform size over which is laid a bed of clean sand. Open jointed tile underdrains are laid in the gravel layer to provide positive drainage as the liquid passes through the sand and gravel.

Underdrains are made of vitrified clay pipes or tiles of atleast 10 cm dia. laid with open joint. Underdrains are placed not more than 6 m apart. Graded gravel is placed around the underdrains in layers upto 30 cm with a minimum of 15 cm above the top of the under drains. Atleast 8 cm of the top layer should consist of gravel of 3 to 6 mm size. Clean sand of effective size of 0.5 to 0.75 mm and uniformity coefficient not greater than 4.0 is placed over the gravel. The depth of sand may vary from 15 to 30 cm.

The drying beds are commonly 6 to 8 m wide and 30 to 45 m long. A length of 30 m away from the inlet should not be exceeded with a single point of wet sludge discharge, when the bed slope is about 0.5%. Multiple discharge points should be used with large sludge beds to reduce the length of wet sludge travel. In order to have flexibility in operation, beds should be atleast two in number.

The area needed for dewatering digested sludge is dependent on total volume of sludge, climate, temperature and location. Areas required for drying beds range from 0.1 to 0.15 m^2/capita with dry solids loading of 80 to 120 kg/m^2 of bed per year for digested primary sludge and from 0.175 to 0.25 m^2/capita with dry solids loading of 60 to 120 kg/m^2/year for digested mixed sludge.

Sludge should be deposited evenly to a depth of not greater than 20 cm. When digested sludge is deposited on a well drained bed of sand described above, the dissolved gases tend to buoy up and float the solids leaving a clear liquid at the bottom which drains through the sand rapidly. The major portion of the liquid drains off in a few hours after which drying commences by evaporation. The sludge cake shrinks producing cracks which accelerates evaporation from the sludge surface. With good drying conditions, the sludge
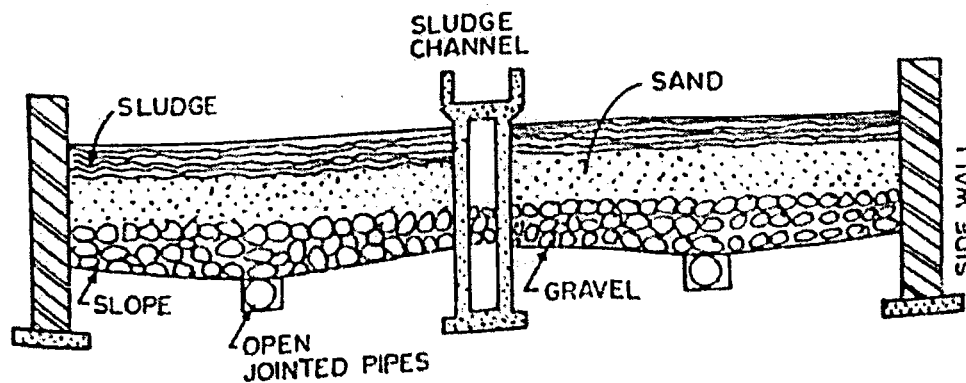

FIG. 16.13. SLUDGE DRYING BEDS.

will dewater satisfactorily and become fit for removal in about 2 to 3 weeks producing a volume reduction of 20 to 40%. Dried sludge cake can be removed by shovel or forks when the moisture content is less than 70%. When the moisture content reaches 40%, the cake becomes lighter and suitable for grinding. Wheel barrows or pick up trucks are used for hauling of sludge cakes.

Mechanical methods. *Vacuum filtration* is the most common mechanical method of dewatering, *filter presses* and *centrifugation* being the other methods. Chemical conditioning is normally required prior to the mechanical methods of dewatering. Mechanical methods may be used to dewater raw or digested sludges preparatory to heat treatment by vacuum filtration because the coarse solids are rendered fine during digestion. Hence filtration of raw primary or a mixture of primary and secondary sludges permits slightly better yields, lower chemical requirements and lower cake moisture contents than filtration of digested sludges. When the ratio of secondary and primary sludge increase, it becomes more and more difficult to dewater the filter. The feed solids concentration has a great influence, the optimum being 8 to 10%. Beyond 10%, the sludge becomes too difficult to pump and lower solids concentration would demand unduly large filter surface. In this method, conditioned sludge is spread out in a thin layer on the filtering medium, the water portion being separated due to the vacuum and the moisture content is reduced quickly.

Vacuum filters : Vacuum filter consists of a cylindrical drum over which is laid a filtering medium of wool, cloth or felt, synthetic fiber or plastic or stainless steel mesh or coil springs. The drum is suspended horizontally so that one quarter of its diameter is submerged in a tank containing sludge. Valves and piping are arranged to apply a vacuum on the inner side of the filter medium as the drum rotates slowly in the sludge. The vacuum holds the sludge against the drum as it continues to be applied as the drum rotates out of sludge tank. This pulls water away from the sludge, leaving a moist cake mat at the outer surface. The sludge cake on the filter medium is scraped from the drum just before it enters the sludge tank again. The filtration rate is expressed in kg of dry solids per square meter of medium per hour. It varies from 10 kg/m^2/hr for activated sludge alone to 50 kg/m^2/hr for primary sludges.

16.9. FINAL DISPOSAL OF SLUDGE

Sludge (either wet, dry or incinerated) can be finally disposed off by the following methods :

1. Spreading on farm land, 2. Dumping, 3. Land filling, 4. Sludge lagooning, 5. Disposing is water or sea.

1. Spreading on farm land. Dewatered sludge may be disposed of by spreading over farm land and ploughing under after it has

dried. Wet dewatered sludge can be incorporated into soil directly by injection. Usually a number of shallow trenches, 50 to 90 cm wide and 0.3 to 0.4 m deep are provided about 1 to 1.5 m apart, and wet sludge is discharged into it. After a sludge cake is formed due to evaporation of water, it is covered with dry earth. After about a month, the whole land is ploughed and used for cultivation.

In general, digested sludges are of moderate but definite value as a source of slowly available nitrogen and some phosphate. They are comparable to farmyard manure except for deficiency in potash. They also contain many essential elements to plant life and minor nutrients, in the form of trace metals. The sluge humus also increases the water holding capacity of soil and reduces soil erosion making an excellent *soil conditioner* specially in arid regions by making available needed humus content which results is greater fertility.

2. **Dumping.** Dumping in an abandoned mine quarry can be resorted to only for sludges and solids that have been stabilized so that no decomposition or nuisance conditions will result. This method can be safely adopted for digested sludge, clean grit and incinerator residue.

3. **Disposal by land filling.** If a suitable site is convenient, a sanitary landfill can be used for disposal of sludge, grease, grit and other solids, whether stabilised or not. However, dewatering is recommended before such disposal, so that the cost of hauling the sludge is reduced. In a true sanitary landfill, the wastes are deposited in a designated area, compacted in place with a tractor or roller and covered with 30 cm layer of clean soil. The *sanitary land fill method* is most suitable if it is also used for disposal of the other solids wastes of the community. However, drainage from the site that would cause pollution of ground water supplies or surface streams must be guarded against.

4. **Sludge Logooning.** A lagoon is a shallow earth basin into which untreated or digested sludge is deposited. Untreated-sludge lagoons stabilize the organic solids by anaerobic and aerobic decomposition, which may give rise to objectionable odours. Hence the lagoons should be located away from the town. Fig 16.14 shows a typical section of a lagoon. The depth of the lagoon may vary from 0.5 to 1.5 m. The depth of the lagoon and its area should be about twice that is required for sand drying under comparable conditions. Agricultural tile drains of about 10 cm dia. are placed at 3 m centres at the bottom of the lagoon, and a 15 cm thick layer of ashes or clinker is placed over it to facilitate drain of water from wet sludge. The detention time may vary from 1 to 2 months. After the sludge has been stabilized and the moisture is drained/evaported,
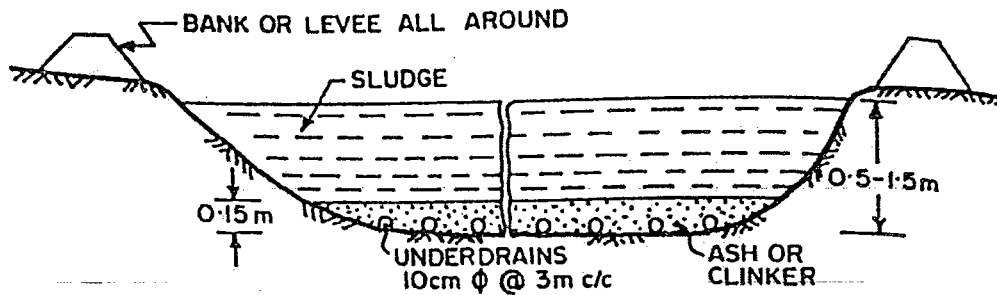
FIG. 16.14. SLUDGE LAGOON

the contents of the lagoon are dug out to about half of its volume and used as manure.

Use of sludge lagoons for storage, digestion , dewatering and final disposal of dried sludge may be adopted in isolated locations where soil is fairly porous and when there is no chance of ground water contamination. Drainage water should not be allowed to enter the lagoon, and hence earth banks are constructed along its perimeter. Lagoons have been used for regular drying of sludge on a fill and draw basis or allowed to fill dry and then levelled out and used as lawns. Lagoons have also been employed as emergency storage when digesters have to be emptied for repairs. As they are less expensive to build and operate, they have been resorted to, particularly for digested sludge in areas where large open land suitably located is available. Use of lagoons is not generally desirable, as they present an ugly sight and cause odour and mosquito breeding.

5. **Disposal in water or sea.** This is not common method of disposal because it is contingent on the availability of a large body of water adequate to permit dilutions. At some sea coast sites, the sludge either raw or digested may be barged to sea far enough to make available the required dilution and dispersion. The method requires careful consideration of all factors for proper design and siting of outfall to prevent any coastal pollution or interference with navigation.

Example 16.17. **Design of sludge drying beds**

Design a sludge drying bed for digested sludge from an activated sludge plant serving 200,000 people.

Solution

From Table 16.1, total solids remaining in digested sludge (combined primary and activated) = 57 gm/capita/day

∴   Daily solids = $200000 \times 57 \times 10^{-3} = 11400$ kg/day

Let us adopt a dry solids loading of 100 kg/m²/year

∴   Area of bed needed = $\dfrac{11400 \times 365}{100} = 41610$ m²

$$= \frac{\text{Incoming BOD} - \text{Outgoing BOD}}{\text{Incoming BOD}}$$

$$= \frac{250 - 20}{250} \times 100\% = \frac{230}{250} \times 100\% = 92\%. \quad \textbf{Ans.}$$

(d) Sludge age in days (θ_c) is given by Eq. (9.48) as

$$\theta_c = \frac{X_t \cdot V}{Q_w \cdot X_R + (Q - Q_w) \cdot X_E}$$

$$= \frac{27250 \text{ kg}}{(220 \text{ m}^3/\text{d} \times 9700 \text{ mg/l}) + (35000 \text{ m}^3/\text{d} - 220 \text{ m}^3/\text{d}) \, 30 \text{ mg/l}}$$

$$= \frac{27250 \text{ kg}}{\frac{220 \times 9700}{1000} \text{ kg/d} + (35000 - 220) \frac{30}{1000} \text{ kg/d}}$$

$$= \frac{27250}{2134 + 1043.4} = \frac{27250}{3177.4} = \textbf{8.58 days.} \quad \textbf{Ans.}$$

9.36. Sludge Volume Index (S.V.I.)

The term *sludge volume index* or *sludge index* is used to indicate the physical state of the sludge produced in a biological aeration system. It represents the degree of concentration of the sludge in the system, and hence decides the rate of recycle of sludge (Q_R) required to maintain the desired MLSS and *F/M* ratio in the aeration tank to achieve the desired degree of purification.

S.V.I. is defined as the volume occupied in ml by one gm of solids in the mixed liquor after settling for 30 minutes, and is determined experimentally.

The standard test, which is performed in the laboratory to compute SVI of an aeration system involves collection of one litre sample of mixed liquor from the aeration tank from near its discharge end in a graduated cylinder. This 1 litre sample of mixed liquor is allowed to settle for 30 minutes and the settled sludge volume (V_{ob}) in ml is recorded as to represent sludge volume. This volume V_{ob} in ml per litre of mixed liquor will represent the quantity of sludge in the liquor in ml/l.

The above sample of mixed liquor, after remixing the settled solids, is further tested in the laboratory for MLSS by the standard procedure adopted for measuring the suspended solids in sewage. Let this concentration of suspended solids in the mixed liquor in mg/l be X_{ob}. Then SVI is given by the equation

$$SVI = \frac{V_{ob}\ (ml/l)}{X_{ob}\ (mg/l)} = \frac{V_{ob}}{X_{ob}}\ ml/mg$$

or

$$\boxed{SVI = \frac{V_{ob}}{X_{ob}} \times 1000\ ml/g}$$

...(9.52)

The usual adopted range of SVI is between 50—150 ml/gm and such a value indicates good settling sludge.

Note. When the given SVI value in ml/gm is divided by $10^3 \times 10^3$ (i.e. 10^6), we will get SVI value in l/mg. SVI value in l/mg will therefore be $\frac{SVI}{10^6}$. SVI value in mg/l will, thus, be given by $\frac{10^6}{SVI}$.

9.37. Sludge Recycle and Rate of Return Sludge

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. The relationship between sludge recirculation ratio $\left(\frac{Q_R}{Q}\right)$ with X_t (MLSS in tank) and X_R (MLSS in returned or wasted sludge) is given as :

$$\frac{Q_R}{Q} = \frac{X_t}{X_R - X_t}$$

...(9.53)

where Q_R = Sludge recirculation rate in m³/d

X_t = MLSS in the aeration tank in mg/L

X_R = MLSS in the returned or wasted sludge in mg/L

The settleability of sludge, as stated in the previous article, is

determined by sludge volume index (SVI), which is determined in the laboratory.

If it is assumed that the sedimentation of suspended solids in the laboratory is similar to that in the sedimentation tank, then

$$X_R = \frac{10^6}{S.V.I.} \quad (i.e. \text{ SVI value in mg/l}) \quad ...(9.54)$$

Eq. (9.53) then becomes

$$\frac{Q_R}{Q} = \frac{-X_t}{\left(\frac{10^6}{S.V.I.} - X_t\right)} \quad ...(9.55)$$

Values of *Return sludge ratios* adopted in different types of Activated sludge systems are shown in table 9.10. Its value for conventional sludge plant varies between 0.25 to 0.50.

The return sludge has always to be pumped, and the pump capacity should be designed for a minimum return sludge ratio of 0.50 to 0.75 for large plants and 1.0 to 1.5 for smaller plants, irrespective of the theoretical requirement. The required capacity should be provided in multiple units to permit variation of return sludge ratio as found necessary during the operation of the plant.

9.38. Wasting of Excess Sludge (Qw)

We know that the sludge generated in the aeration tank has to be partly discharged and wasted out of the plant to maintain a steady level of MLSS in the system. *The excess sludge quantity will increase with the increasing F/M ratio, and decrease with temperature.* In the case of domestic sewage, Q_w will be about 0.50—0.75 kg per kg BOD removed for the conventional sludge plants (having F/M ratio varying between 0.4 to 0.3).

Excess sludge may be wasted either from the sludge return line, or directly from the aeration tank as mixed liquor. The latter procedure is usually preferred, since the concentration of suspended solids will then be fairly steady in the waste discharge making the control easy.

In conventional plants, the wasted sludge is taken directly to a sludge thickner and digestor, or to the primary settling tank for its disposal along with the primary sludge. In extended aeration plants, however, the excess sludge is directly taken to the sludge drying beds.

9.39. Modifications of the Basic Activated Sludge Process

In the basic activated sludge process, also called *conventional aeration process,* the recirculated activated sludge is added to the inlet end of the aeration tank as a single dose. The regime flow employed in the aeration tank is *plug flow* and not *mixed flow.* Plug flow implies that the sewage moves down progressively along the aeration tank, essentially unmixed with the rest of the tank con-