UNIT III

Wastewater Quantity Estimation

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

Quantity= Per capita sewage contributed per day x Population

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

Fluctuations in Dry Weather Flow

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

Population Equivalent

Population equivalent is a parameter used in the conversion of contribution of wastes from industrial establishments for accepting into sanitary sewer systems. The strength of industrial sewage is, thus, written as Std. BOD5 = (Std. BOD5 of domestic sewage per person per day) x (population equivalent)

Wastewater Characterization

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

Temperature pH
Colour and Odour Carbonaceous substrates Nitrogen
Phosphorous Chlorides
Total and volatile suspended solids (TSS and VSS) Toxic metals and compounds

Design of Sewers

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

- The sewage contain particles in suspension, the heavier of which may settle down at the bottom of the sewers, as and when the flow velocity reduces, resulting in the clogging of sewers. To avoid silting of sewers, it is necessary that the sewer pipes be laid at such a gradient, as to generate self cleansing velocities at different possible discharges.

- The sewer pipes carry sewage as gravity conduits, and are therefore laid at a continuous gradient in the downward direction upto the outfall point, from where it will be lifted up, treated and disposed of.

Hazen-William's formula; U=0.85 C rH0.63S0.54

Manning's formula: U=1/n rH2/3S1/2

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

Darcy-Weisbach formula: hL= (fLU2)/(2gd)

Minimum Velocity

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

Maximum Velocity

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

Effects of Flow Variation on Velocity in a Sewer

Due to variation in discharge, the depth of flow varies, and hence the hydraulic mean depth (r) varies. Due to the change in the hydraulic mean depth, the flow velocity (which depends directly on r2/3) gets affected from time to time. It is necessary to check the sewer for maintaining a minimum velocity of about 0.45 m/s at the time of minimum flow (assumed to be 1/3rd of average flow). The designer should also ensure that a velocity of 0.9 m/s is developed tleast at the time of maximum flow and preferably during the average flow periods also. Moreover, care should be taken to see that at the time of maximum flow, the velocity generated does not exceed the scouring value.

Sewer Appurtenances

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

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

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

Special Man-holes:

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

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

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

Lamp-holes: Lamp holes are the openings constructed on the straight sewer lines between two man-holes which are far apart and permit the insertion of a lamp into the sewer to find out obstructions if any inside the sewers from the next man-hole.
Street inlets: Street inlets are the openings through which storm water is admitted and conveyed to the storm sewer or combined sewer. The inlets are located by the sides of pavement with maximum spacing of 30 m.

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

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

Pumping of Sewage

Pumping of sewage is required when it is not possible to have a gravitational flow for the entire sewerage project.
Sufficient pumping capacity has to be provided to meet the peak flow, atleast 50% as stand by.
Types of pumps :

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

The raw sewage must be treated before it is discharged into the river stream. The extent of treatment required to be given depends not only upon the characteristics and quality of the sewage but also upon the source of disposal, its quality and capacity to tolerate the impurities present in the sewage effluents without itself getting potentially polluted. The layout of conventional wastewater treatment plant is as follows:
Indian Standards for discharge of sewage in surface waters are given in the table below.

Characteristic of the Effluent
BOD5

TSS
Tolerance limit for Discharge of Sewage in Suface Water Sources 20 mg/L 30 mg/L
The unit operations and processes commonly employed in domestic wastewater treatment, their functions and units used to achieve these functions are given inthe following table:
Unit Operations/Processes, Their Functions and Units Used for Domestic Wastewater Treatment
Unit

Operations/Processes
Screening
Grit Removal
Functions Treatment Devices
Removalof largefloating, Bar racks and screens of various
suspended and settleable solids description
Removal of inorganic suspended Grit chamber solids
Primary Sedimentation Removal of organic/inorganic
Primary sedimentation tank
settleable solids

Aerobic Biological Conversion of colloidal, dissolved
Activated sludge processunits and
Suspended Growth and residual suspended organic its
modifications, Waste stabilisation
Process matter into settleable biofloc and ponds,
Aerated lagoons
stable inorganics

Aerobic Biological Attached same as above Trickling filter,
Rotating biological
Growth Process contactor

Anaerobic biological growth Conversion of organic matter into
Anaerobic filter, Fluid bed
processes CH4 & CO2 and relatively stable
submerged media anaerobic reactor,
organic residue Upflow anaerobic sludge
blanket
reactor, Anaerobic rotating
biological
contactor

Anaerobic Stabilization of same as above Anaerobic digestor
Organic Sludges

Screening

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

Types of Screens

Coarse Screens: Coarse screens also called racks, are usually bar screens, composed of vertical or inclined bars spaced at equal intervals across a channel through which sewage flows. Bar screens with relatively large openings of 75 to 150 mm are provided ahead of pumps, while those ahead of sedimentation tanks have smaller openings of 50 mm.
Bar screens are usually hand cleaned and sometimes provided with mechanical devices. These cleaning devices are rakes which periodically sweep the entire screen removing the solids for further processing or disposal. Hand cleaned racks are set usually at an angle of 45° to the horizontal to increase the effective cleaning surface and also facilitate the raking operations. Mechanical cleaned racks are generally erected almost vertically. Such bar screens have openings 25% in excess of the cross section of the sewage channel.
Medium Screens: Medium screens have clear openings of 20 to 50 mm.Bar are usually 10 mm thick on the upstream side and taper slightly to the downstream side.
The bars used for screens are rectangular in cross section usually about 10 x 50 mm, placed with larger dimension parallel to the flow.

Fine Screens: Fine screens are mechanically cleaned devices using perforated plates, woven wire cloth or very closely spaced bars with clear openings of less than 20 mm. Fine screens are not normally suitable for sewage because of clogging possibilities.
The most commonly used bar type screen is shown in figure:
Velocity
The velocity of flow ahead of and through the screen varies and affects its operation. The lower the velocity through the screen, the greater is the amount of screenings that would be removed from sewage. However, the lower the velocity, the greater would be the amount of solids deposited in the channel. Hence, the design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0.6 to 1.2 mps through the open area for the peak flows have been used satisfactorily. Further, the velocity at low flows in the approach channel should not be less than 0.3 mps to avoid deposition of solids.

Head loss
Head loss varies with the quantity and nature of screenings allowed to accumulate between cleanings. The head loss created by a clean screen may be calculated by considering the flow and the effective areas of screen openings, the latter being the sum of the vertical projections of the openings. The head loss through clean flat bar screens is calculated from the following formula:
$h = 0.0729 (V2 - v2)$

where, h = head loss in m
V = velocity through the screen in mps
v = velocity before the screen in mps
Another formula often used to determine the head loss through a bar rack is Kirschmer's equation:
$h = K(W/b)4/3\ hv \sin \theta$
where h = head loss, m
K = bar shape factor (2.42 for sharp edge rectangular bar, 1.83 for rectangular bar with semicircle upstream, 1.79 for circular bar and 1.67 for rectangular
bar with both u/s and d/s face as semicircular).
W = maximum width of bar u/s of flow, m
b = minimum clear spacing between bars, m
hv = velocity head of flow approaching rack, m = v2/2g
θ = angle of inclination of rack with horizontal
The head loss through fine screen is given by h = (1/2g) (Q/CA)
where, h = head loss, m Q = discharge, m3/s
C = coefficient of discharge (typical value 0.6) A = effective submerged open area, m2
The quantity of screenings depends on the nature of the wastewater and the screen openings.

Equalization Tanks

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

Types of Equalization Tanks

Equalization tanks are generally of three types:

1. Flow through type

2. Intermittent flow type

3. Variable inflow/constant discharge type

The simple flow through type equalization tank is mainly useful in assisting self neutralization or evening out of fluctuating concentrations, not for

balancing of flows since a flow through type tank once filled, gives output equal to input.

Flow balancing and self-neutralization are both achieved by using two tanks, intermittently one after another. One tank is allowed to fill up after which it

is checked for pH (or any other parameter) and then allowed to empty out. The second tank goes through a similar routine. Intermittent flow type tanks are

economic for small flows from industries.
When flows are large an equalization tank of such a size may have to be provided that inflow can be variable while outflow is at a constant rate, generally by a pump.The capacity required is determined from a plot of the cumulative inflow and a plot of the constant rate outflow and measuring the gaps between the two plots. A factor of safety may be applied if desired.
Generally, detention time vary from 2 to 8 hours but may be even 12 hours or more in some cases. When larger detention times are required, the equalization unit is sometimes provided in the form of facultative aerated lagoon.

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

Aerated Grit Chamber

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

Principle of Working of Grit Chamber

Grit chambers are nothing but like sedimentation tanks, designed to separate the intended heavier inorganic materials (specific gravity about 2.65) and to pass forward the lighter organic materials. Hence, the flow velocity should neither be too low as to cause the settling of lighter organic matter, nor should it be too high as not to cause the settlement of the silt and grit present in the sewage. This velocity is called "differential sedimentation and differential scouring velocity". The scouring velocity determines the optimum flow through velocity. This may be explained by the fact that the critical velocity of flow 'vc' beyond which particles of a certain size and density once settled, may be again introduced into the stream of flow. It should always be less than the scouring velocity of grit particles. The critical velocity of scour is given by Schield's formula:
$V = 3 \text{ to } 4.5 \, (g(Ss - 1)d)1/2$

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

Types of Velocity Control Devices

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

Design of Grit Chambers

Settling Velocity

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

- Stoke's law: v= g(Ss-1)d2

- 18μ

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

Re=ρvdμ

For grit particles of specific gravity 2.65 and liquid temperature at 10°C, μ =1.01 x 10-6m2/s. This corresponds to particles of size less than 0.1 mm.

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

v2=4g(ρp-ρ)d 3CDρ

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

CD=18.5=18.5

Re0.6 (ρvd/ μ)0.6

Primary Sedimentation

Primary sedimentation in a municipal wastewater treatment plant is generally plain edimentation without the use of chemicals. In treating certain industrial astes chemically aided sedimentation

may be involved. In either case, it constitutes flocculent settling, and the particles do not remain discrete as in the case of grit, but tend to agglomerate or coagulate during settling. Thus, their diameter keeps increasing and settlement proceeds at an over increasingvelocity. Consequently, they trace a curved profile.

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

Long tube settling tests can be performed in order to estimate specific value of surface loading and detention time for desired efficiency of clarification for a given industrial wastewater using recommended methods of testing. Scale-up factors used in this case range from 1.25 to 1.75 for the overflow rate, and from 1.5 to 2.0 for detention time when converting laboratory results to the prototype design.
For primary settling tanks treating municipal or domestic sewage, laboratory tests are generally not necessary, and recommended design values given in table may be used. Using an appropriate value of surface loading from table, the required tank area is computed. Knowing the average depth, the detention time is then computed. Excessively high detention time (longer than 2.5 h) must be avoided especially in warm climates where anaerobicity can be quickly induced.
Design parameters for settling tank

Types of settling	Overflow rate m3m2/day		Solids loading kg/m2/day		Depth	Detention time
	Average	Peak	Average	Peak		
Primary settling only	25-30	50-60	-	-	2.5-3.5	2.0-2.5
Primary settling followed by secondary treatment	35-50	60-120	-	-	2.5-3.5	

Primary settling with activated 25-35 50-60 - - 3.5- - sludge return 4.5

Secondary settling for trickling 15-25 40-50 70-120 190 2.5- 1.5-2.0 filters 3.5

Secondary settling for activated 3.5- sludge (excluding extended 15-35 40-50 70-140 210 - 4.5 aeration)

Secondary settling for extended 8-15 25-35 25-120 170 3.5- - aeration 4.5

Classification of Micro organisms

1. Nutritional Requirements: On the basis of chemical form of carbon required, microorganisms are classified as

a. Autotrophic: organisms that use CO2 or HCO3- 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
The curve shown may be divided into six well defined phases:
1. Lag Phase:adaptation to new environment, long generation time and null growth rate.
2. Accelaration phase: decreasing generation time and increasing growth rate.
3. Exponential phase: minimal and constant generation time, maximal and constant specific growth rate and maximum rate of substrate conversion.

4. Declining growth phase: increasing generation time and decreasing specific growth rate due to gradual decrease in substrate concentration and increased accumulation of toxic metabolites.
5. Stationary phase: exaustion of nutrients, high concentration of toxic metabolites, and cells in a state of suspended animation.
6. Endogenous phase: endogenous metabolism, high death rate and cell lysis.

Biomass Growth Rate

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

Total rate of microbial growth, $\frac{dx}{dt} = \frac{\mu m X S}{Ks + S}$

where,
μm = maximum specific growth rate
X = micro organism concentration
S = substrate concentration
Ks = substrate concentration at one half the maximum growth rate
Similarly, rate of substrate utilization,
$\frac{dS}{dt} = \frac{k X S}{Ks + S}$ 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 $= \frac{\mu m X S}{Ks + S} - kdX$
where, kd = 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: $= Y.k$ m 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, kd, k and Ks 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:

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 andsoluble 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.

$$q = \frac{Q\,(SO - Se)}{V\,X}$$

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

$$\theta c = \frac{V\,X}{QwXr + (Q - QwXe)}$$

where SO and Se are influent and effluent organic matter concentration respectively, measured as BOD5 (g/m3), X, Xe and Xr are MLSS concentration in aeration tank, effluent and return sludge respectively, and Qw= waste activated sludge rate.

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

$$QwXr = YQ\,(SO - Se) - kd\,XV$$

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

From the above equation it is seen that $1/\theta c = Yq - kd$

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

$$F/M = Q(SO - Se) / XV = QSO / XV$$

The θc 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.

$$VX = \frac{YQ\theta_c(SO - S)}{1 + k_d \theta_c}$$

Alternately, the tank capacity may be designed from F/M = QSO / XV

Hence, the first step in designing is to choose a suitable value of 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/m3. For typical domestic sewage, the MLSS value of 2000-3000 mg/l if conventional plug flow type aeration system is provided, or 3000-5000 mg/l for completely mixed types. Considerations which govern the upper limit are: initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitations of oxygen transfer equipment to supply oxygen at required rate in small reactor volume, increased solids loading on secondary clarifier which may necessitate a larger surface area, design criteria for the tank and minimum HRT forthe 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, theaeration 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 icro-organisms in the system. The total oxygen requirement of the process may be formulated as follows:

O_2 required (g/d) = Q(SO - S) - 1.42 QwXr

f

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

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

Aeration Facilities

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

dissolved oxygen in the wastewater.

Secondary Settling

Secondary settling tanks, which receive the biologically treated flow undergo zone or compression settling. Zone settling occurs beyond a certain concentration when the particles are close enough together that interparticulate forces may hold the particles fixed relative to one another so that the whole mass tends to settle as a single layer or "blanket" of sludge. The rate at which a sludge blanket settles can be determined by timing its position in a settling column test whose results can be plotted as shown in figure.
Compression settling may occur at the bottom of a tank if particles are in such a concentration as to be in physical contact with one another. The weight of particles is partly supported by the lower layers of particles, leading to progressively greater compression with depth and thickening of sludge. From the settling column test, the limiting solids flux required to reach any desired underflow concentration can be estimated, from which the rquired tank area can be computed. The solids load on the clarifier is estimated in terms of (Q+R)X, while the overflow rate or surface loading is estimated in terms of flow Q only (not Q+R) since the quantity R is withdrawn from the bottom and does not contribute to the overflow from the tank. The secondary settling tank is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. Beyond an MLSS concentration of 2000 mg/l the clarifier design is often controlled by the solids loading rate rather than the overflow rate.
Recommended design values for treating domestic sewage in final clarifiers and mechanical thickeners (which also fall in this category of compression settling) are given in lecture 22.

Sludge Recycle

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

Qr = X

Q Xr-X

where Qr = Sludge recirculation rate, m3/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 Xr = 106/SVI. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids. The Xr value may not be taken more than 10,000 g/m3 unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value.

Excess Sludge Wasting

The sludge in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing

F/M and decrease with increasing temperature. Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed

liquor. The latter is preferred as the sludge concentration is fairly steady in that case. The excess sludge generated under steady state operation may be

estimated by
θc = VX

QwXr

or QwXr = YQ (SO - S) - kd XV

Theory of Aeration

Aeration is a gas-liquid mass transfer process in which the driving force in the liquid phase is the concentration gradient (Cs - C) for slightly soluble gases.
Mass transfer per unit time =KL.a (Cs - C) where, KL = Liquid film coefficient
=Diffusion coefficient of liquid (D)
Thickness of film (Y)
a = Interficial area per unit volume
Cs =saturation concentration at the gas-liquid interface and C = some lower value in the body of the liquid.
The value of a increases as finer and finer droplets are formed, thus increasing the gas transfer. However, in practice, it is not possible to measure this
area and hence the overall coefficient (KL.a) per unit time, is determined by experimentation.
Adjustment for Field Conditions

The oxygen transfer capacity under field conditions can be calculated from the standard oxygen transfer capacity by the formula: $N = [Ns(Cs- CL)x\ 1.024T-20\alpha\ ]/9.2$

where,

N = oxygen transferred under field conditions, kg O_2/h.

Ns= oxygen transfer capacity under standard conditions, kg O_2/h. Cs= DO saturation value for sewage at operating temperature. CL= operating DO level in aeration tank usually 1 to 2 mg/L.

T = Temperature, degree C.

α = Correction factor for oxygen transfer for sewage, usually 0.8 to 0.85.

Aeration Facilities

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

Trickling Filters

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

Process Description

- The wastewater in trickling filter is distributed over the top area of a vessel containing non-submerged packing material.
- Air circulation in the void space, by either natural draft or blowers, provides oxygen for the microorganisms growing as an attached biofilm.
- During operation, the organic material present in the wastewater is metabolised by the biomass attached to the medium. The biological slime grows in thickness as the organic matter abstracted from the flowing wastewater is synthesized into new cellular material.
- The thickness of the aerobic layer is limited by the depth of penetration of oxygen into the microbial layer.
- The micro-organisms near the medium face enter the endogenous phase as the substrate is metabolised before it can reach the micro-organisms near the medium face as a result of increased thickness of the slime layer and loose their ability to cling to the media surface. The liquid then washes the slime off the medium and a new slime layer starts to grow. This phenomenon of losing the slime layer is called sloughing.
- The sloughed off film and treated wastewater are collected by an underdrainage which also allows circulation of air through filter. The collected liquid is passed to a settling tank used for solid- liquid separation.

Types of Filters

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

S.No.	Design Feature	Low Rate Filter	High Rate Filter
1.	Hydraulic loading, m3/m2.d	1 - 4	10 - 40
2.	Organic loading,kg BOD / m3.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 ettling tank.

Process Design

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

of equations:

1. NRC equations (National Research Council of USA)

2. Rankins equation

3. Eckenfilder equation

4. Galler and Gotaas equation

NRC and Rankin's equations are commonly used. NRC equations give satisfactory values when there is no re-circulation, the seasonal variations in temperature
are not large and fluctuations with high organic loading. Rankin's equation is used for high rate filters.
NRC equations: These equations are applicable to both low rate and high rate filters. The efficiency of single stage or first stage of two stage filters, E2
is given by
E2= 100
1+0.44(F1.BOD/V1.Rf1)1/2
For the second stage filter, the efficiency E3 is given by
E3=100
[(1+0.44)/(1- E2)](F2.BOD/V2.Rf2)1/2
where E2= % efficiency in BOD removal of single stage or first stage of two-stage filter, E3=% efficiency of second stage filter, F1.BOD= BOD loading of
settled raw sewage in single stage of the two-stage filter in kg/d, F2.BOD= F1.BOD(1- E2)= BOD loading on second-stage filter in kg/d, V1= volume of first
stage filter, m3; V2= volume of second stage filter, m3; Rf1= Recirculation factor for first stage, R1= Recirculation ratio for first stage filter, Rf2=
Recirculation factor for second stage, R2= 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 BOD5/day/ m3 filter volume.
2. Hydraulic load (including recirculation) should not exceed 30 m3/m2 filter surface-day.
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 = (R/Q) + 1 (R/Q) + 1.5
Other Aerobic Treatment Units

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

Anaerobic Treatment

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

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

Fundamental Microbiology

The anaerobic treatment of organic wastes resulting in the production of carbon dioxide and methane, involves two distinct stages. In the first stage, complex waste components, including fats, proteins, and polysaccharides are first hydrolyzed by a heterogeneous group of facultative and anaerobic bacteria.

These bacteria then subject the products of hydrolysis to fermentations, -oxidations, and other metabolic processes leading to the formation of simple organic compounds, mainly short-chain (volatile) acids and alcohols. The first stage is commonly referred to as "acid fermentation". However in the second stage the end products of the first stage are converted to gases (mainly methane and carbon dioxide) by several different species of strictly anaerobic bacteria. This stage is generally referred to as "methane fermentation".

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

The bacteria responsible for acid fermentation are relatively tolerant to changes in pH and temperature and have a much higher rate of growth than the bacteria responsible for methane fermentation. As a result, methane fermentation is generally assumed to be the rate limiting step in anaerobic wastewater treatment.