11 : Aerobic Secondary Treatment Of Wastewater (Contd.)

19.2.2 Design of Trickling Filters

Organic loading rate and recirculation ratio are main consideration in design of trickling filter. The early performance equations for trickling filter were empirical as proposed by National Research Council (1946), Rankine (1955) and based on biochemical kinetics Velz (1948).

Rankine's formula

For single stage filters: the BOD of influent to the filter (including recirculation) shall not exceedthreetimestheBODrequiredforsettledeffluent.HencereferringtotheFigure19.13 and using following notations, wehave

$$S_2 + R_1(S_4) = 3\,(1+R_1)S_4 \qquad (38)$$

Or $$S_4 = S_2/(3+2R_1) \qquad (39)$$

where, S_2 = BOD of settled influent, S_4 = BOD of TF effluent after SST, R_1 = Recirculation ratio, and if E = efficiency, then

$$E = (1 + R_1)/(1.5+R_1) \qquad (40)$$

Value of recirculation is given by

$$R = (Q_1 – Q)/Q \qquad (41)$$

Where Q_1 is total flow including recirculation and Q is sewage flow.

For second stage filter: The BOD of the wastewater applied to the second stage filter including recirculation shall not exceed two times the effluent BOD. Therefore,

$$S_4 + R_2(S_6) = 2\,(1+R_2)S_6 \qquad (42)$$

Or $$S_6 = S_4 / (R_2+2) \qquad \text{and efficiency} = (1 + R_2)/(2+R_2) \qquad (43)$$

Where S_4 = BOD of influent to second stage filter, S_6 = BOD of TF effluent after SST, R_2 = Recirculation ratio.

Eckenfelder (1970) has developed performance equation based on the specific rate of substrate removal for a pseudo-first-order reaction.

$$-\frac{1}{x}\frac{ds}{dt} = K.S \qquad (44)$$

Where $\frac{1}{x}\frac{ds}{dt}$ = Specific rate of substrate utilization, $\frac{\text{mass of substrate}}{\text{Microbial mass x Time}}$

$\frac{ds}{dt}$ = rate ofsubstrateutilization, $\frac{\text{mass}}{\text{(Volume)x Time}}$

K =rateconstant, $\frac{\text{volume}}{\text{(mass of microbes) x Time}}$

S = Substrate concentration, $\frac{\text{mass}}{\text{Volume}}$

Rearranging the above equation (44) for integration,

$$\int_{S_o}^{S_t}\frac{dc}{S} = -KX\int_{O}^{t} dt \qquad (45)$$

X = Average cell mass concentration, $\frac{\text{mass}}{\text{Volume}}$

$S_{o=}$ Substrate concentration applied for filter bed

S_t = Substrate concentration after contact time, t

Integrating the equation

$$\frac{S_t}{S_o} = e^{-K.X.t} \qquad (46)$$

X is proportional to surface area of the media (A_s) i.e.,

$$X \approx A_s^m \qquad (47)$$

Where, A_s is the specific area of the packing media

The mean contact time 't' for a filter is given by Howland (1950) (Reynolds & Richard, 1996).

$$t = \frac{C.D}{Q_L^n} \qquad (48)$$

Where t = mean contact time

D = Depth of filter bed

Q_L = Surface loading

C and n =constant

Substituting equation 46 & 47 in equation 48

$$\frac{S_t}{S_o} = e^{\frac{(-KA_s^m D)}{Q_L^n}} \qquad (49)$$

(EliminationofconstantCasitistakenintoaccountinK) m =

experimentalconstant

Thevalueof‘n’dependsonflowcharacteristicsthroughpackingandusuallyabout0.5to 0.67. For specific wastewater and filter media equation 49 may be simplified bycombining KA_s^m to give

$$\frac{S_t}{S_o} = e^{\frac{-KD}{Q_L^n}} \qquad (50)$$

K= 0.01 to 0.1 for various wastewater and media

For Surfpac, Dow chemical (89 m^2/m^3), K = 0.088 and n = 0.5 (Reynolds & Richard, 1996)

When D- feet, Q_L = gal/(min-ft^2). Actual K values can be determined from the pilot performance.Withtemperaturevariationthevalueofkcanbeconvertedasbelow:

$$K_T = K_{20} \text{ x} 1.035^{(T-20)} \qquad (51)$$

K_T = rate constant at temperature T

K_{20} = rate constant at 20˚C

T= temperature, ˚C

One of the most common kinetic equations for filter performance while treating municipal wastewater was developed by Eckenfelder, 1961 as

$$\frac{S_t}{S_o} = \frac{1}{1+C\left(\frac{D^{0.67}}{Q_L^{0.5}}\right)} \qquad (52)$$

S_t = BOD_5 of effluent, mg/L

S_o = BOD_5 of influent, mg/L

C=constant=2.5forFPSunitand5.358forSIunits D =

filter depth, ft (orm)

Q_L = unit loading rate MG/acre-day (m^3/m^2-d)

The above equation is obtained from second order kinetics equation,

$$\frac{1}{x}\frac{ds}{dt}=KS^2 \qquad (53)$$

Integrating

$$\frac{S_t}{S_o}=\frac{1}{(1+S_0.K.X.t)} \qquad (54)$$

Substituting, $t=C\left(\frac{D^{0.67}}{Q_L^{0.5}}\right)$ and combining constant S_o, K, X & C for the same wastewater treatment under steady state performance results, the equation becomes:

$$\frac{S_t}{S_o}=\frac{1}{1+\mathrm{Conct}\left(\frac{D^{0.67}}{Q_L^{0.5}}\right)} \qquad (55)$$

Example: 2

Design low rate trickling filter for secondary treatment of sewage generated from 10000 persons with rate of water supply 170 LPCD. The BOD_5 after primary treatment is 110 mg/L and BOD_5 of final effluent should be 20 mg/L. Consider C = 5.358.

Solution

$$\frac{S_t}{S_o}=\frac{1}{1+C\left(\frac{D^{0.67}}{Q_L^{0.5}}\right)}$$

Provide depth D = 1.5 m

Average sewage flow = 10000 x 170 x 0.80 x 10^{-3} = 1360 m^3/d

Now, $\frac{20}{110}=\frac{1}{1+5.358\left(\frac{1.5^{0.67}}{Q_L^{0.5}}\right)}$

$$0.182\left(1+\frac{7.03}{Q_L^{0.5}}\right)=1$$

$$\therefore Q_L^{0.5}=1.562$$

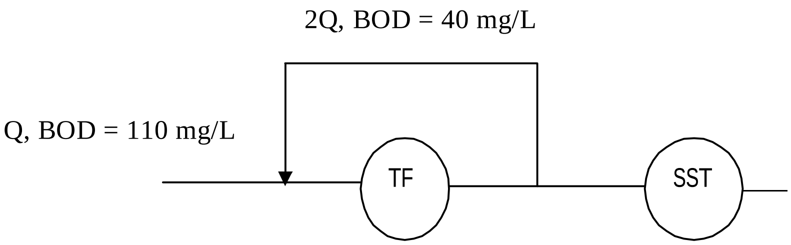
$Q_L = 2.441 m^3/m^2$-d where, Q_L = flow/area

∴ Plan area = $\frac{1360}{2.441}$ = 557.35 m^2

Hence, Diameter of trickling filter = 26.64 m

Example: 3

Design high rate trickling filter for the data given above except effluent BOD_5 = 40 mg/l since polishing treatment is provided after high rate trickling filter. Consider recirculation ratio of 2 and filter depth of 1.8 m.



BOD in the effluent,

$Q \times 110 + 2Q \times 40 = (1 + 2)\,Q.S_0$

$\therefore S_0 = 63.33$ mg/L

$$\frac{40}{63.3} = \frac{1}{1 + 5.358\,\left(\frac{1.8^{0.67}}{Q_L^{0.5}}\right)}$$

$Q_L = 186\ m^3/m^2\text{-d}$

Wastewater flow is $1360 m^3/d$

∴ Recycle flow = 2720 m^3/d

and, total flow = 4080 m^3/d

∴ Area of Trickling Filter = $\frac{4080}{186}$ = 21.94 m^2

∴ Diameter = 5.285 m

Example: 4

DesignasinglestageBiotower(superratetricklingfilter)forthefollowingdata: Average wastewater flow = $500 m^3/d$,

Influent BOD = 160 mg/L

BODremovalinprimarytreatment=30%,

Effluent BOD required = 20mg/L

Filter depth = 5.0 m; Recycle ratio R/Q = 2

Pilot plant studies using synthetic packing have shown a removal rate constant K = 2.26 at 20^{0}C,andn=0.5(Reynolds&Richard,1996).Thewinterwastewatertemperature=15^{0}C. Provide minimum two filters inparallel.

Solution:

$$\frac{St}{SO} = e^{\left(-\frac{KD}{Q_L^{0.5}}\right)}$$

Now, $K_{15} = K_{20} 1.035^{(T-20)}$

$= 2.26 \times 1.035^{(-5)}$

$= 1.903$ per day

BOD$_5$inincomingwastewatertoBiotower=160(1-0.30)=112mg/L Recycle

flow = 2Q and BOD = 20mg/L

Therefore, 112 (Q) + 2Q x 20 = 3Q. S_0

Hence, $S_0 = 50.67$ mg/L

Substituting values in theequation

$$\frac{2O}{5O.67} = e^{\left(-\frac{1.903 \times 5.0}{Q_L^{0.5}}\right)}$$

Solvingweget, $Q_L = 104.8$ m^3 /m^2.d

Wastewater flow = 500m^3/d

Hydraulicloadonfilter=500+2x500

= 1500 m^3/d

Area required = 1500/104.8 =14.313 m^2

Hence, diameter required for each Biotower when two are provided in parallel = 3.02 m

12 : Aerobic Secondary Treatment Of Wastewater (Contd.)

19.3 Ponds System for Treatment of Wastewater

It is a shallow body of water contained in an earthen basin, open to sun and air. Longer time of retention from few days to weeks is provided in the pond. The purification of wastewater occurs due to symbiotic relationship of bacteria and algae. The ponds are classified according to the nature of the biological activity which takes place within the pond as aerobic, facultative and anaerobic. These are cheaper to construct and operate in warm climate as compared to conventional treatment system and hence they are considered as low cost wastewater treatment systems. However, they require higher land area as compared to conventional treatment system.

19.3.1 Classification of Ponds

Aerobic Ponds: In aerobic pond the microbial population similar to ASP exists along with algae. The aerobic population release CO_2, which is taken up by the algae for their growth. Algae in turn release O_2, which helps in maintaining the aerobic condition in the pond. Very shallow depth of aerobic pond (0.15 to 0.45 m) is used for the treatment of wastewater for removal of nitrogen by algae growth. For general wastewater treatment depth of 0.5 to 1.2 m may be used. The solar radiation should penetrate to the entire depth of the pond to support photosynthesis to keep entire pond content aerobic. When shallow ponds (0.5 m deep) are used for tertiary treatment of wastewater, they are very lightly loaded and such ponds are called as *maturation pond*. These maturation ponds may release oxygen in atmosphere during daytime.

Facultative stabilization Ponds: Most of the ponds exist in facultative nature. Three zones exist in this type of ponds (Figure 19.14). The top zone is an aerobic zone in which the algal photosynthesis and aerobic biodegradation takes place. In the bottom zone, the organic matter present in wastewater and cells generated in aerobic zone settle down and undergo anaerobic decomposition. The intermediate zone is partly aerobic and partly anaerobic. The decomposition of organic waste in this zone is carried out by facultative bacteria. The nuisance associated with the anaerobic reaction is eliminated due to the presence of top aerobic zone. Maintenance of an aerobic condition at top layer is important for proper functioning of facultative stabilization pond, and it depends on solar radiation, wastewater characteristics, BOD loading and temperature. Performance of these ponds is comparable with conventional wastewater treatment.

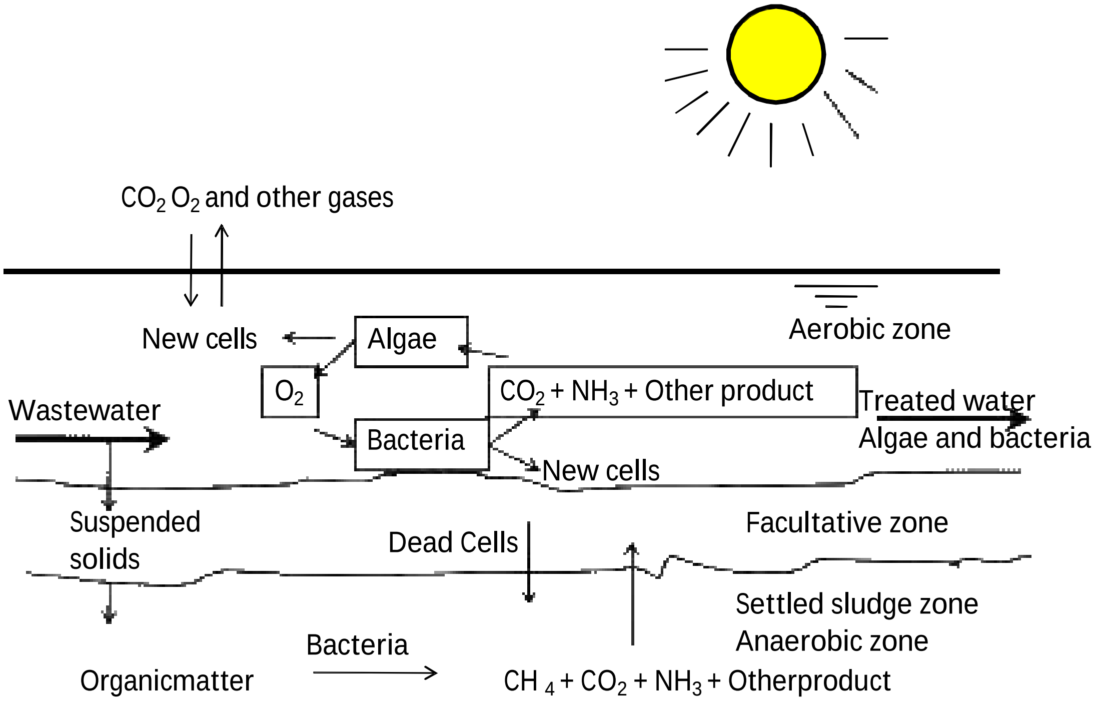


Figure 19.14 Facultative stabilization pond

Anaerobic pond: In anaerobic pond, the entire depth is under anaerobic condition except an extremelyshallowtoplayer.Normallythesepondsareusedinseriesfollowedbyfacultative or aerobic pond for complete treatment. The depth of these ponds is in the range of 2.5 to6 m. They are generally used for the treatment of high strength industrial wastewaters and sometimes for municipal wastewater and sludges. Depending upon the strength of the wastewater, longer retention time up to 50 days is maintained in the anaerobic ponds. Anaerobic lagoons are covered these days by polyethylene sheet for biogas recovery and eliminating smell problem and green house gas emission in atmosphere.

Fish pond**:** It can be part of maturation pond or altogether separate pond, in which fish are reared. Sometimes, fishes are also reared in the end compartment of primary pond.

Aquatic plant ponds: These are secondary ponds in which aquatic plants e.g. hyacinths, duckweeds,etc.areallowedtogroweitherfortheirabilitytoremoveheavymetalsandother substances from wastewaters, or to give further treatment to wastewaters and produce new plantbiomass.Thisrecoveredbiomasscanbeusedforbiodiesel,bioethanol,combustiblegas recoveryasfuelormanyotherchemicalscanberecoveredusingtheseplantsasfeedstock.

High-rate algal ponds: The high rate algal pond (HRAP) is potentially an effective disinfectionmechanismwithintherequirementsofsustainability.Inadditiontodisinfection, nutrientremovalmechanismsarealsoactiveintheHRAP,specificallythoseinvolvedinthe removalofphosphate.Thesepondsarenotdesignedforoptimumpurificationefficiencybut formaximumalgalproduction.Thealgaeareharvestedforavarietyofuses,principallyhigh qualityalgalprotein.Thepondsareshallowlagoons20–50cmdeep,witharetentionperiod of 1–3 days. The whole pond is kept aerobic by maintaining a high algal concentration and using some form of mechanical mixing. Mixing is normally carried out for short periods at night to prevent the formation of a sludge layer. Mixing may be required for short periods duringthedaytopreventariseinpHinthesurfacewaterduetophotosynthesis.Thepondis commissioned in the same way as a facultative pond except that continuous loading should notbepermitteduntilanalgalbloomhasdeveloped.Loadingdependsonsolarradiation,and theaverageloadingthroughouttheyearcouldbe100to200kgBODha^{-1}d^{-1}.Strongorganic sewageinhibitsthephotosyntheticactionduetohighammoniaconcentrations,whichresults inthepondbecominganaerobic.Highratealgalpondsaredesignedtopromotethesymbiosis between the microalgae and aerobic bacteria, each utilizing the major metabolic products of the other. Microalgae grow profusely releasing oxygen from water by photosynthesis. This oxygenisimmediatelyavailabletobacteriatooxidizemostofthesolubleandbiodegradable organic matter remaining from the facultative pond. HRAPs are shallower than facultative pondsandoperateatshorterhydraulicretentionstimes(HRTs).Attherapidgrowthofalgae, thepHcanraisetoabove9sinceatpeakalgalactivity.Carbonateandbicarbonateionsreact toprovidemorecarbondioxideforthealgae,leavinganexcessofhydroxylions.ApHabove 9for24hoursensuresa100%killingofE.coliandpresumablymostpathogenicbacteria.

Primaryandsecondaryponds:Pondsreceivinguntreatedwastewatersarereferredasrawor *primary waste stabilization ponds*. Those receiving primary treated or biologically treated wastewaters for further treatment are called as *secondary waste stabilization ponds*. *Maturation pond* is the secondary pond receiving already treated wastewater either fromthe ponds or other biological wastewater treatment process, like UASB reactor or ASP. The detention time of 5 to 7 days is provided in these ponds, with the main purpose of achieving natural bacterial die-off to desired levels. In warm climate they often constitute an economical alternative for chlorination. They are lightly loaded in terms of organic loading andtheoxygengeneratedbyphotosynthesismaybemorethantheoxygendemand.

19.3.2 Typical Flow Chart of Pond Based TreatmentPlant

The typical treatment flow sheets for different types of ponds in use are illustrated in the Figure 19.15. The ponds can be used in series or in parallel. Chlorination of the treated effluent is optional. The primary treatment after screen can be combined in the ponds along with secondary treatment. In all the flowcharts of the ponds in the Figure 19.15, screens are provided ahead of the first pond.

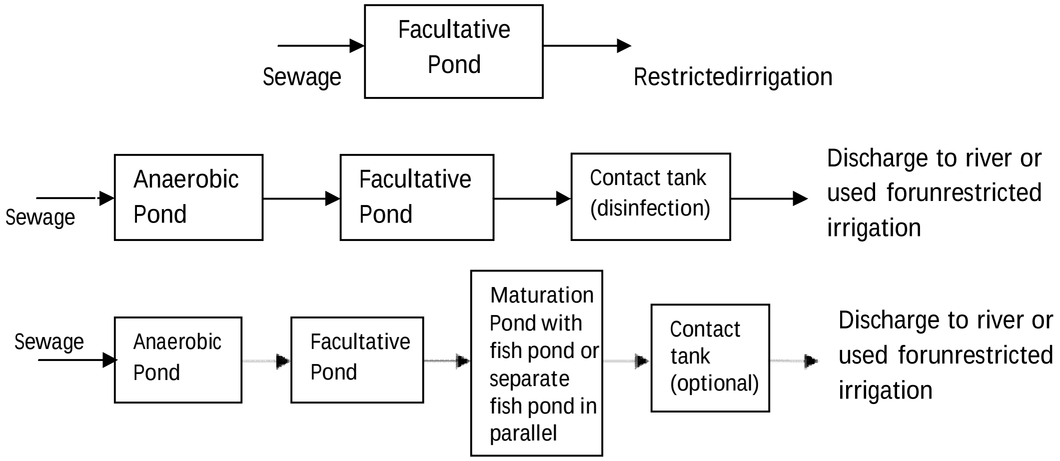


Figure 19.15 Flowcharts of the waste stabilization ponds

19.3.3 Factors Affecting PondEcosystem

The principal abiotic components of ponds ecosystem are oxygen, carbon dioxide, water, light and nutrients; while the biotic components are algae, bacteria, protozoa, and variety of other organisms. Various factors affect the pond design, such as (Arceivala and Asolekar, 2007):

- Wastewater characteristics andfluctuation,
- Environmental factors such as solar radiation, sky clearance, temperature, and their variation,
- Algal growth pattern and their diurnal and seasonalvariation,
- Bacterial growth pattern and decayrates,
- Hydraulic transportpattern,
- Evaporation andseepage,
- Solidssettlement,liquefaction,gasification,upwarddiffusion,sludgeaccumulation,
- Gas transfer atinterface.

19.3.4 Design Guidelines for Oxidation Pond

1. *Depth of Pond:* It should be within 1 m to 1.5 m. The ponds are designed with such a shallow depth to provide proper penetration of light, thus allowing growth of aquatic plants and production oxygen. When these ponds are used for sewage treatment the primary objective is organic matter removal and a depth of 1 m to 1.2 m is used. Shallow ponds experience higher temperature variation than deeper ponds. So, an optimum pond depth is necessary.

2. *Surface area of Pond:* Sufficient surface area must be provided so that oxygen yields from the pond is greater than the ultimate BOD load applied. NEERI gives photosynthetic oxygen yield for different latitude in India:

Latitude (^{0}N)	Yield of photosynthetic O_2 (kg/ha.day)
16	275
20	250
24	225
28	200
32	175

Individual pond area should not be greater than 0.5 ha. If any system requires more area then it is desirable to have more than one pond. 25% more area is provided than that calculated to account for embankments.

3. *Substrate removal rate:* Substrate removal rate K_p varies from 0.13 to 0.20 per day at 25^0C and 0.10 to 0.15 at 20^0C. For other temperature it can be calculated as:

$$K_p\,(T^oC) = K_p(20^oC)(1.035)^{(T-20)} \qquad (56)$$

The size of the pond will be half when plug flow pattern is maintained rather than completely mixed conditions. This can be achieved by providing ponds in series.

4. *Detention time (T):* It should be adequate enough for the bacteria to stabilize the applied BOD load to a desirable degree.

5. *Sulphide production:* Sulphide production in oxidation ponds can be calculated from the following empirical relationship (Arceivala and Asolekar, 2007):

$$S^{2-}\,(mg/l) = (0.0001058 * BOD_5 - 0.001655 * T + 0.0553) * SO_4^{2-}$$

Where, BOD_5 is in kg/ha.days,

T = detention time in days,

SO_4^{2-} in mg/l.

Sulphide ion concentration should not be greater than 4 mg/L. At concentrations higher than this algal growth is inhibited.

6. *Coliform removal:* To use the pond effluent for irrigation Coliform concentration should be less than 1000/100 ml. Coliform removal follows the first order rate equation (Arceivala and Asolekar,2007):

$$dN/dt=K_b.N, \qquad (57)$$

where, N = Number of organisms at any given time, t

K_b = Death rate per unit time (1 to 1.2 per day at 20^0C)

7. *Sludgeaccumulation:* Sludgeaccumulationoccursinthepondattherate0.05to0.08 m^3/capita/year. Sludge accumulation causes decrease in efficiency of the ponds, so they require cleaning every 7 to 10year.
8. *Pretreatment:* Medium screens and grit removal devices should be provided before theponds.
9. Inlet pipe with the bell mouth at its end discharging near the centre of the pond is provided.
10. The overflow arrangement is box structure with multiple valve draw-off lines to permit operation with seasonal variations indepth.
11. Ifthesoilisperviousitshouldbesealedtoprohibitseepage.

Example: 5

Design an Oxidation Pond with efficiency 85 % for a wastewater stream of 2 MLD with a BOD of 200 mg/L and the effluent coming out of the pond should have a BOD less than 30 mg/L. Temperature of the influent wastewater is 30^0C and the oxidation pond is located at a place having latitude 22^0N.

Solution

- At 22^0N, oxygen production by photosynthesis = 235kg/hectare.day,
- And K_p =0.23/day.
- The oxidation pond is designed for plug flowconditions.
- For plug flow conditions, dispersion number, D/UL =0.2
- $K_p t$ (for efficiency = 85%, D/UL = 0.2) = 2.5 (Arceivala and Asolekar,2007)
- Therefore, detention time, $t = K_p t/K_p = 2.5/0.23 = 10.87$days.
- Now, wastewater flow = 2 MLD = 2000m^3/day

- Therefore,pondvolume=detentiontime*flow=2000*10.87=21739.14m^3
- MaximumBODloadthatcanbeappliedonthepond=235/0.85=276.47kg/day
- Influent ultimate BOD = (1/0.68) * 200* 2 = 588.2kg/ha.day
- Therefore,minimumpondarearequired=588.2/276.47=2.13ha
- Gross land area required = 1.25 * 2.13 = 2.66ha
- Minimumponddepth=(PondVolume)/(Pondarea)=[21739.14/(2.13*10000)=
- = 1.02m
- Providelength=225m,breadth=118.2m,freeboard=1m,
- Therefore depth of the pond = 2.02m
- To maintain plug flow conditions the pond is divided into 3 cells along length with each cell length = 75m.

12 : Aerobic Secondary Treatment Of Wastewater

19.3.5 Design of Facultative Stabilization Pond

In design the oxygen resources of the pond are equated to the applied organic loading. The principal source of oxygen is photosynthesis and that is dependent on solar energy. The solar energy again is related to geographical, meteorological and astronomical phenomenon, and varies principally with time in year and the altitude of the place. The yield of photosynthetic oxygen for different latitude is given earlier. Yield of photosynthetic oxygen may be calculated directly if the amount of solar energy in Cal/m^2.day and the efficiency of conversion of light energy to fix energy in the form of algal cells are known.

In design of facultative ponds part of the organic matter is considered to undergo anaerobic decomposition and the photosynthetic oxygen yield is equated to the remaining organic matter to support aerobic oxidation. The organic loading in kg of BOD per hectare per day applied on pond can be estimated using (Rao and Dutta, 2007):

$$Lo = 10(d/t)BODu \qquad (58)$$

where, Lo = Organic loading in kg/ha.day

d = depth of pond in m

t = detention time in days

BODu = ultimate soluble BOD, mg/L

(Loading = ((BOD*Q)/A), now A=V/d, therefore loading = BOD*Q/(V/d), hence loading = BOD.d/t)

The organic loading may be modified for **elevations** above mean sea level by dividing by factor(1+0.003EL). Where, EL is elevation of pond site above MSL in hundred meters. For every 10% decrease in the sky clearance factor below 75%, the pond area may be increased by 3% (CPHEEO,1993).

Example: 6

Design facultative stabilization pond to treat a domestic sewage of 2 MLD, located at a place where the latitude is 20 ^oN and 500 m above mean sea level. The five day 20 ^oC BOD of the sewage is 200 mg/L. Suitable other data may be assumed for Indian conditions.

Solution

$BOD_5 = BODu(1 – e^{-k.t})$

Therfore, $BODu = BOD_5 / (1 – e^{-k.t})$

Assuming k = 0.23 per day

Ultimate BOD = $200 / (1 – e^{-5x0.23}) = 293$ mg/L

At 20 ^oN, the yield of photosynthetic oxygen = 250 kg/ha.day.

Sincetheplaceis500maboveMSL,theoxygenyield=250/(1+0.003x5)=246.3kg/ha.d Now

organic loading can be calculatedas

Lo = 10 (d/t) BODu = 10 (d/t) x 293

Assuming 50% of this load is non settleable, and it undergoes aerobic decomposition in the top layer.

The oxygen requirement = 10 (d/t) x 293 x 0.5

Equatingthistophotosyntheticoxygenyieldof246.3kg/ha.dayandsolving d/t

=0.168

Provide d = 1.5 m, hence t = 1.5/0.168 = 8.922 days

Now,Volume=flowxdetentiontime=depthxsurfacearea

Therefore, Area required = $2 \times 10^3 \times 8.922 / 1.5 = 11900$ m^2 (~1.19 Ha.)

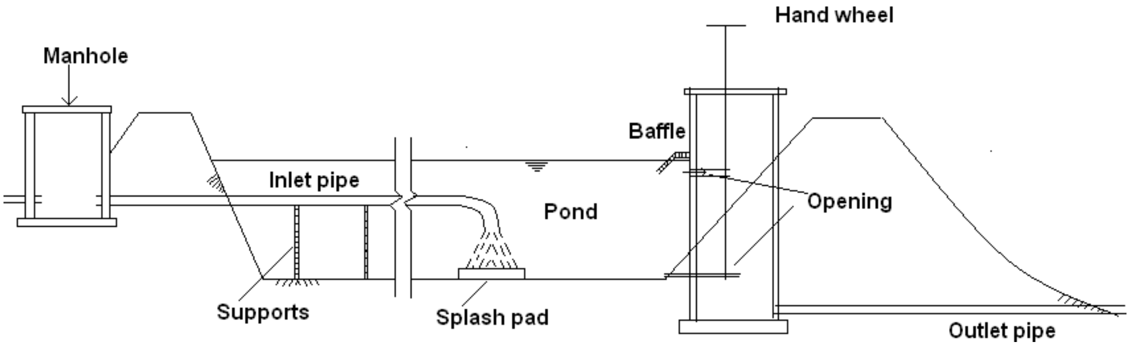


Figure 19.16 Sectional elevation of facultative waste stabilization pond

19.4 AeratedLagoon

Aerated lagoons are one of the aerobic suspended growth processes. An aerated lagoon is a basin in which wastewater is treated either on a flow through basis or with solids recycle. Oxygen is usually supplied by means of surface aerators on floats or on fixed platforms or diffused air aeration units instead of photosynthetic oxygen yield as in case of oxidation pond. The action of the aerators and that of the rising air bubbles from the diffuser are used tokeepthecontentsofthebasininsuspension.Theyareconstructedwithdepthvaryingfrom 2 to 5m.

The contents of an aerobic lagoon are mixed completely. Depending on detention time, the effluent contains about 1/3 to 1/2 the value of the incoming BOD in the form of cell tissue. Before the effluent can be discharged, the solids must be removed by settling. If the solids are returned to the lagoon, there is no difference between this and modified ASP.

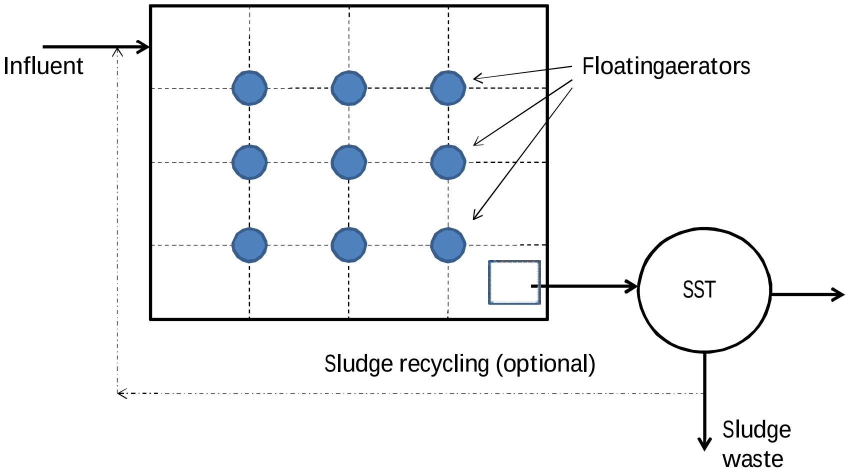


Figure 19.17 Aerobic lagoon

The mean cell retention time should be selected to assume, 1) that the suspended microorganisms will easily flocculate by sedimentation and 2) that the adequate safety factor is provided when compared to mean cell residence time of washout. The oxygen requirement is as per the activated sludge process. In general, the amount of oxygen required has been found to vary from 0.7 to 1.4 times the amount of BOD_5 removed.

Aerated lagoons have the advantages such as ease of operation and maintenance, equalization of wastewater, and a high capacity of heat dissipation when required. The disadvantages of aerated lagoons are (Barnhart, 1972) large area requirement, difficulty in process modification, high effluent suspended solids concentration, and sensitivity of process efficiency to variation in ambient air temperature.

Aerobic lagoons: In aerobic lagoons, power levels are great enough to maintain all the solids in the lagoons in suspension and also to provide dissolved oxygen throughout the liquid volume. Aerobic lagoons are operated with high F/M ratio and short MCRT. These systems achieve little organic solids stabilization but convert the soluble organic material into cellular organic material. Based on the solid handling manner, the aerobic lagoons can be classified into (i) aerobic flow through with partial mixing, and (ii) Aerobic lagoon with solid recycle and nominal complete mixing (Arceivala, 1998).

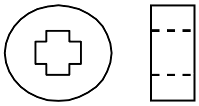
Aerobic flow through with partial mixing: This type of aerobic lagoons operate with sufficient energy input to meet the oxygen requirement, but the energy input is insufficient to keep all the biomass in suspension. The HRT and SRT are the same in this type of lagoon. The effluent from this lagoon is settled in an external sedimentation facility to remove the solid prior to discharge.

Aerobic lagoons with solid recycle: This type of lagoons are same as the extended aeration activated sludge process with the exception that the aeration is carried out in an earthen basin instead of a reinforced concrete reactor basin and have longer HRT than the extended aeration process. The oxygen requirement in this type of lagoon is higher than the aerobic flow through lagoons to keep all the biomass in suspension. The analysis of this type of lagoons is same as the activated sludge process.

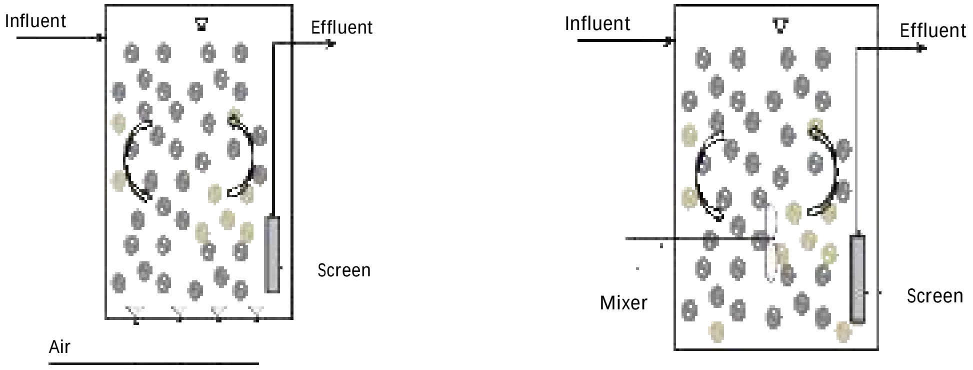
19.5 Moving Bed Biofilm Reactor (MBBR)

Moving bed biofilm reactor was developed by Norwegian company, KaldnesMiljoteknologies. This is mainly attached growth process where media is not stationary and it moves freely in the reactor to improve substrate removal kinetics. Small cylindrical shaped polyethylene carrier elements (sp. density 0.96 g/cm^3) are added in aerated or non-aerated basins to support biofilm growth. Cylinders of 10 mm ø and 7 mm thick with a cross inside are popularlyused.

Figure 19.18 Typical polyethylene media used in MBBR



The biofilm carriers are retained in the reactor by the use of a perforated plate (5 x 25 mm slots) at the tank outlet. Thus, this media having larger size cannot escape the reactor along with the effluent. Air agitation or mixers are used to continuously circulate the packing and to keep it moving so as to establish optimum contact with substrate present in wastewater and bacteria attached to the media. Packing may fill 25 to 50% of tank volume, with specific surface area of about 200 to 500 m^2 /m^3 of bulk packing volume. This arrangement offers advantage that no return sludge is required and since the media is moving, there is no chance of blocking the media which may require back washing. A final clarifier is used to settle sloughed solids. Another advantage is use of more efficient fine bubble aeration equipment is not required, which would require periodic drainage of aeration tank and removal of packing for cleaning ofdiffusers.



(a) (b)

Figure 19.19 Typical reactors used with suspended packing materials a) Aerobic b) Anaerobic/Anoxic with internal mixer.

MBBR are finding increasing application for post treatment of anaerobically treated industrial effluents and also as a secondary treatment system for treatment ofsewage. These reactors can be used for removal of organic matter and also for nitrification and denitrification. Single stage MBBR may meet the effluent standards for treatment of primarily treated sewage. Whereas, to achieve nitrification and denitrification along with organicmatterremovalmultistageMBBRisusedwithintermediatesettlerorsettlerprovided at the end of all the reactors. As per the need the first few MBBRs will be anoxic to achieve carbonaceous organic matter removal and denitrification and these will be followed by aerobicMBBRmainlyfornitrificationandremainingorganicmatterremoval.

Typical design parameters used for MBBR are stated below:

Detentiontime,h	3 –5
Biofilmarea, m^2/m^3	200 –250
BODloading,kg/m^3.d	1.0 –2.0
Secondary clarifier hydraulic loading rate, m^3/m^2.d	12 –20

Questions

1. Why aerobic treatment systems produce more sludge than anaerobic treatment systems?
2. Define SVI. Calculate SVI of the sludge for the laboratory test results furnished below:

Sludgesettledvolume(SSV)after30min.settling=280mL
MLVSSinaerationtank=3500mg/L,andSS/VSS=0.8

3. Whyrecyclingofthesludgeis necessaryinactivatedsludge process?
4. Describe different types of activated sludge processused.
5. Differentiate between completely mixed activated sludge process and extended aeration activated sludgeprocess.
6. Using microbial growth kinetics, derive expression for determination of reactor volumeforcompletemixed activatedsludgeprocesswithsludgerecycling.
7. Calculate oxygen required per day for treatment of 500 m^3/d wastewater containing 300 mg of BOD /L and

	TKN of 30 mg/L in activated sludge process. The effluent should have BOD of 20 mg/L and TKN of 3 mg/L. Consider Y = 0.5 mg VSS/mg BOD,meancellresidencetime=10daysandk_d=0.06perday.
8.	Estimate recirculation ratio for ASP when the MLVSS concentration in the aeration tank is 4000 mg/L and the return sludge concentration is 9000 mg of SS/L. Consider VSS/SS =0.8.
9.	An activated sludge process is to be used for secondary treatment of 10000 m^3/d of wastewater.TheBODofsettledwastewaterafterprimarytreatmentis150mg/Land it is desirable to have not more than 10 mg/L of soluble BOD in the effluent. Consider Y = 0.5; K_d = 0.05 per day; MLVSS concentration in the aeration tank = 3000 mg/L and underflow concentration from the

clarifier 10,000 mg/L of SS. VSS/SS=0.80.Determinei)thevolumeofaerationtank ,ii)sludgetobewastedper day(massandvolume),iii)t herecycleratio,andiv)Vol umetricloadingandF/M.

10. What is sludge bulking? How it can becontrolled?
11. Describe sequencing batchreactor.
12. Describe the working of moving bed biofilm reactor. What advantages this reactor willoffer?
13. Describedifferenttypesoftricklingfilterusedinwastewatertreatment.

14. Design a biotower for treatment of wastewater generated from the housing scheme havingpopulationof5000personswithrateofwatersupply180LPCD.Removalrate constantforsyntheticpackingmediaK=2.26at20^{0}C.
15. Classify the ponds used for wastewatertreatment.
16. Describe high rate algalpond.
17. Designfacultativestabilizationpondtotreatadomestic sewageof5MLD,locatedat a place where the latitude is 22 ^{o}N and 300 m above mean sea level. The five day 20^{o}C BOD of the sewage is 150 mg/L. Suitable other data may be assumed for Indian conditions.

Answers:

Q. 2: SVI = 100 mL/g of SS

Q. 7: Oxygen required = 205.452 kg O_2/day

Q. 8: Recirculation ratio of for ASP = 0.714.

Q. 9: i) The volume of aeration tank = 1560 m^3; ii) sludge to be wasted per day (mass and volume) = Sludge to be wasted per day = 577.5 Kg SS/day, sludge waste volume = 46.8

m^3/d; iii) the recycle ratio = 0.6; and iv) Volumetric loading = 0.96 Kg BOD/ m^3.day and F/M = 0.32 kg BOD/kg VSS.d.

Q. 14: For biotower depth of 5.0 m, area required = 10.09 m^2, hence diameter = 3.58 m

Q. 17: For depth of 1.5 m, area of pond = 2.36 hectare.

13 : Secondary Sedimentation

The secondary sedimentation facility is provided after the biological reactor to facilitate the sedimentation of the cells produced during biological oxidation of organic matter. If these cells produced are not removed, complete treatment will not be achieved as these cells will represent about 30 to 60% of the organic matter present in untreated wastewater in aerobic treatment. Depending on the type of reactor used fraction of these settled cells is returned back to the reactor and remaining cells are wasted as excess sludge for further treatment.

20.1 Analysis of hindered settling (Type3)

In systems that contain high concentration of suspended solids, both hindered (zone) settling (type 3) and compression settling usually occur in addition to discrete and flocculent settling. The settling phenomenon that occurs when a concentrated suspension, initially of uniform concentration throughout, is placed in cylinder as shown in Figure 20.1.

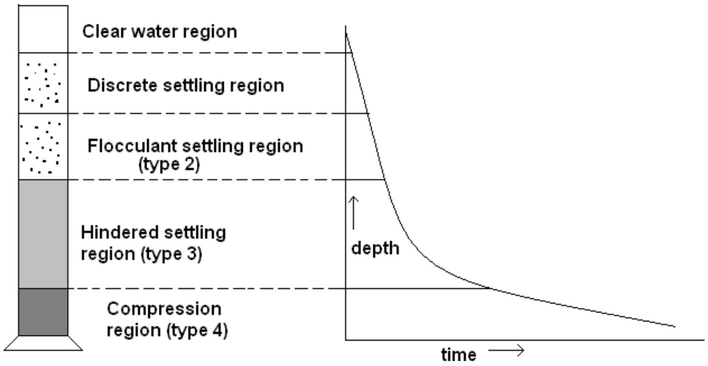


Figure 20.1 Schematic diagram of settling regions for ASP

Due to high concentration of particles, the liquid tends to move up through theinterstices of contacting particles. As a result, the particles settle as a zone or ‘blanket’, maintaining the same relative position with respect to each other. This is known as ‘hindered settling’. As the solids settle as a zone, a relatively clear layer of water is produced above the particles in the settling region. The rate of settling in the hindered settling region is a function of concentration of solids and theircharacteristics.

As settling continues, a compressed layer of particles begins to form at the bottom of the cylinder in the compression settling region. Thus in hindered settling region a gradation in solid concentration exists from interface of settling region to that found in the compression settling region. Due to variability of nature of solids and concentration, settlingtest is necessary to determine the settling characteristics. Two different approaches can be used for conducting the laboratory test.

1. Single (batch) settling test,or
2. Solid flux method (based on series of tests at different suspended solids concentration).

20.2 Area Requirement Based on Single Batch TestResult

Thefinaloverflowrateselectedfordesignofsedimentationtankisbasedonthe following:

- Area needed forclarification,
- Area needed forthickening,
- The rate of sludgewithdrawal.

Since the area needed for the free settling region is less than the area required for thickening, the rate of free settling is rarely the controlling factor. In case of activated sludge process where light, fluffy floc particles may be present, it is conceivable that the free or flocculent settling velocity of these particles could control the design.

For a column of height = $H_{o,}$ and uniform solid concentration = C_o, the position of interface as the time elapses is given in Figure 20.2. The rate at which interface subsides is equal to slope of the curve at that point in time.

The area required for thickening

$$A = \frac{Qt_u}{H_o} \qquad (1)$$

Where, A = Area required for sludge thickening, m^2

Q = Flow rate in the tank,m^3/sec

H_o = Initial height of interface in column, m

t_u= Time to reach desired underflow concentration, sec.

The critical concentration controlling the sludge handling capability of the tank occurs at a height H_2 (Figure 20.2), where concentration is C_2 (C_2 is determined by extending tangent and bisecting angle of intersection).

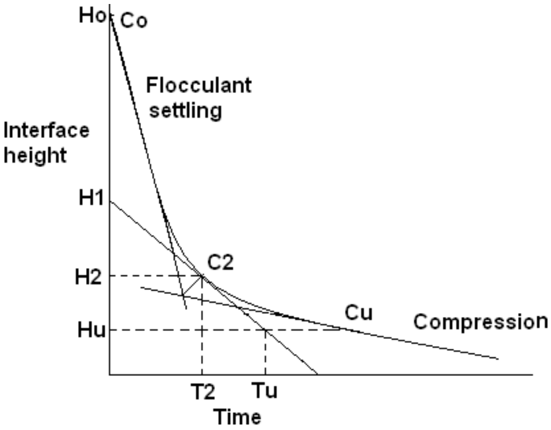


Figure 20.2 Analysis of the single batch test result

The time t_u can be determined as follows (Metcalf and Eddy, 2003):

a. Construct a horizontal line at the depth ‘H_u’ that corresponds to the depth at which the solids are at the desired underflow concentration,‘C_u’.

The value of H_u can be determined as:

$$H_u = \frac{C_o . K_o}{C_u} \qquad (2)$$

b. Construct a tangent to a settling curve at point C_2.

c. Construct a vertical line from intersection of tangent and horizontal line from ‘H_u’. This vertical line will determine ‘t_u’.

With this value of ‘t_u’ the area required for thickening is computed using relation,

$$A = \frac{Q t_u}{H_o}$$

The area required for clarification is then determined. The larger of the two areas is the controlling value. Although ‘C_u’ in settling test will occur at longer time, due to continuous withdrawal from the bottom of tank this time may not reach in settling tank, hence ‘t_u’ is worked out from tangent.

Example: 1

Calculate the size of SST for ASP. In a settling cylinder of 2.0 m height the settling test was performed, and the settling curve as shown in Figure 20.3 was obtained for an activated sludge with initial solids concentration, C_o = 4300 mg/L. Determine the area to yield a thickened sludge concentration C_u of 20 g/L with an inflow of 500 m^3/day. In addition, determine the solids loading in kg/m^2.day and the overflow rate in m^3/m^2.day.

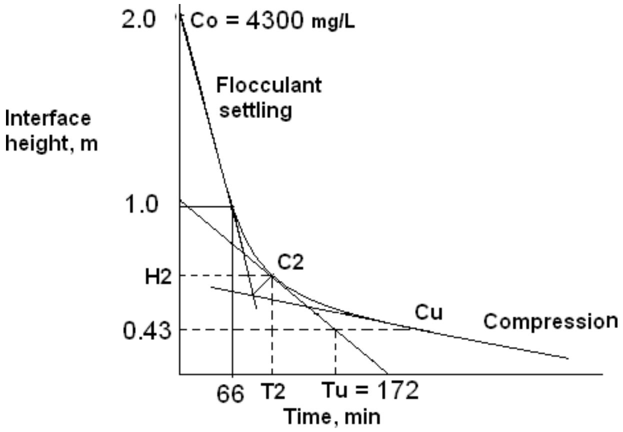


Figure 20.3 Results of the batch settling test

Solution:

1. Thearearequiredforthickening,

$$H_u = \frac{C_o.K_o}{C_u}$$

$$= \frac{4300 \text{ x } 2.0}{20000} = 0.43\text{m}$$

2. Draw a horizontal line at height H_u =0.43m
3. Construct a tangent to the settling curve at C_2, the midpoint of the regionbetween hindered and compression settling. The intersection of the tangent at C_2 and horizontal line is at t_u= 172min.

   Hence, the required area

$$A = \frac{Qt_u}{H_o} = \frac{500}{24 \text{ x } 60} \text{ x } \frac{172}{2.0} = 29.86 \text{ m}^2$$

4. This area should be adequate for clarificationalso.
   a. Determine subsidence velocity '*v*' from the beginning of the settling portion of the curve, (considering velocity of particles present at the interface).

$$v = \frac{2.0\text{–}1.0}{66} \text{ x } 60 = 0.91\text{m/h}$$

   b. Determine overflowrate:

The overflow rate is proportional to the liquid volume above the critical sludge zone (sludge is drained from bottom).

$$Q = 500\ (m^3/day) \times \frac{2.0 – 0.43}{2.0} = 392.5\ m^3/day$$

c. Determine area required forclarification:

The area required is obtained by dividing overflow rate by settling velocity.

$$A = \frac{Q}{v} = \frac{392.5}{0.91} \times \frac{1}{24} = 17.97\ m^2$$

5. The controlling requirement is the thickening area of 29.86 m^2 because it exceeds the area required forclarification.
6. Determine the solidsloading:

$$\text{Solids loading(kg/day)} = \frac{500(\frac{m^3}{day}) \times 4300(g/m^3)}{10^3\ (gm/kg)} = 2150 kg/day$$

$$\text{Solidsloadingrate} = \frac{2150 kg/day}{29.86\ m^2} = 72\ kg/\ m^2\ .day$$

7. Determine hydraulic loadingrate:

$$\text{Hydraulic loading rate} = \frac{392.5}{29.86} = 13.15\ m^3/m^2.day$$

20.3 Design of secondary sedimentationtank

The design guidelines for secondary sedimentation tank for different biological processes as recommended by CPHEEO manual (1993) is presented in the Table 20.1.

Table 20.1 Design parameters for SST

Parameter	Overflow rate, $m^3/m^2.d$		Solid loading rate, $kg/m^2.d$		Depth, m	Detention time, h
	Average	Peak	Average	Peak		
SST for TF	15 - 25	40 - 50	70 - 120	190	2.5 – 3.5	1.5 – 2.0
SST for ASP	15 - 25	40 - 50	70 - 140	210	3.5 – 4.5	1.5 – 2.0
SST for extended aeration ASP	8 - 15	25 - 35	25 - 120	170	3.5 – 4.5	1.5 – 2.0

Weir loading rate for the secondary sedimentation tank is kept less than or equal to 185 m^3/m.d.Otherguidelinesforthedimensionsofthetankaresimilartoasdescribedearlier in primarysedimentation.

Example: 2

Design secondary sedimentation tank for treatment of 10 MLD effluent coming from conventional ASP. The MLSS in aeration tank is 3000 mg/L and peak flow factor is 2.0

Solution

Adopt surface loading rate of 20 m^3/m^2.d at average flow.

Therefore surface area required = 10000/20 = 500m^2

Then surface overflow rate at peak flow = 20000/500 = 40 m^3/m^2.d (within 40 to 50)

Check for solid loading

At average flow solid loading rate = 10000 x 3/500 = 60 kg/m^2.d

At peak flow solid loading rate = 20000 x 3/500 = 120 kg/m^2.d (less than permissible)

Diameter of the tank for 500 m^{2}area = 25.24m

Provide detention time of 2 h, hence volume = 10000*2/24 = 833.33

m^{3}Hence depth of the tank = 833.33/500 = 1.67m

Providedepthof2.0m+0.3mforsludgeaccumulationand0.4mfreeboard.Hencetotal depth = 2.7m

Check for weir loading

Weir loading = 10000/(π * D) = 126.18 m^3/m.d, hence safe.

Questions

1. Describe different types ofsettling.
2. Explain hindered settling. How the area required for sedimentation is worked out on the single batch testresult?
3. Preparenotesondifferenttypesofsettlingoccurringintreatmentofwastewaters.
4. Explain the purpose of providing secondarysedimentation.
5. Design secondary sedimentation tank for the industrial effluent treatment plant employing completely mixed ASP. The MLVSS in aeration tank is 3500 mg/L and average annual wastewater flow rate is 500 m^3/d. However in summer the wastewatergenerationis1.5timestheannualaverage.ConsiderMLVSS/MLSS= 0.8.

Answer:

Q. 5. Provide surface loading rate of 20 m^3/m^2.d at average flow. Diameter 5.64 m and total depth = 2.7 m.

14 : Anaerobic Wastewater Treatment Processes

21.1 Background

Anaerobic biological treatment is well understood and used frequently as anaerobic digesters to treat complex organic solid wastes such as primary and secondary wastewater sludges. However, it has not been used much in the past to treat low strength organic wastewaters from industrial and domestic applications. Aerobic processes were preferred for treatment of these wastewater streams because they are easy to operate and can tolerate process fluctuations. In comparison, anaerobic reactors were assumed to be less stable under fluctuations, more expensive to install and require long start-up time. This belief was due to limited knowledge of the process and reactor design.

Now the technology advances have significantly reduced the historical weakness of anaerobic treatment. With the work of Young and McCarty in the year 1969, application of anaerobic process for the treatment of industrial and municipal wastewaters has gradually increased in last three decades. Today the anaerobic treatment has emerged as a practical and economical alternative to aerobic treatment due to significant advantages over aerobic treatment.

21.2 Anaerobic Degradation of Organic matter

The factors that determine the removal efficiency of biodegradable organic matter are:

1. The nature and composition of the organic matter to be removed
2. Suitability of environmental factors
3. Sludge retention time in the reactor
4. The intensity of mixing, hence contact between bacterial biomass and organic matter.
5. Specific loading of organic matter with respect to bacterial sludge mass, and retention time.

Factors (1) and (2) are basically dependent on wastewater characteristics, whereas (3) to (5) are related to the type and design of the treatment system. The transformation of complex macromolecules of organic matter present in wastewater into biogas requires several groups of microorganisms. The reaction sequence of the anaerobic digestion of complex macromolecules is presented in Figure 21.1 [Gujer and Zehnder, 1983]. Different steps are necessary for the anaerobic digestion of proteins, carbohydrates, and lipids. Four different phases can be distinguished in the overall conversion process of organic matter to biogas as 1) Hydrolysis, 2) Acidogenesis, 3) Acetogenesis, and 4) Methanogenesis.

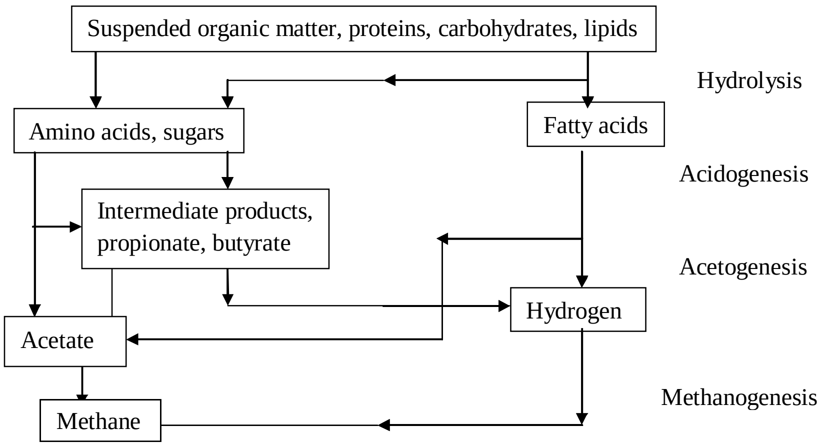


Figure 21.1 Reaction Sequences for the Anaerobic Digestion of Complex Organic Matter

21.3 Overview of Anaerobic DegradationProcess

The anaerobic biological conversion of organic waste to methane is a complex process involvinganumberofmicrobialpopulationslinkedbytheirindividualsubstrateandproduct specificities.Theoverallconversionprocessmaybedescribedtoinvolvedirectandindirect symbiotic association between different groups of bacteria. The product of one bacterium is often the substrate for others and hence, a balance between the bacterial numbers and the substrate concentrations must be maintained. The biological conversion of organic matter occurs in three steps. The first step in the process involves transformation of higher-molecular-mass compounds into compounds suitable for use as a source of energy and cell carbon (hydrolysis). The second step (acidogenesis) involves the bacterial conversion of the compounds resulting from the first step into identifiable lower-molecular-massintermediate compounds. Lower chain volatile fatty acids produced during acidogenesis are utilized by a group of bacteria (acetogens) to produce acetate. The third step (methanogenesis) involves the bacterial conversion of the intermediate compounds into simpler end products, such as methane and carbon dioxide. Several nomenclatures have been proposed for these three steps. Speece and McCarty (1962) called the first and the second steps the constant BOD phaseandthethirdstep,thereducingBODphase,becauseonlythemethaneformationinthe thirdstepbringsaboutthereductionofBODorCODthroughthewholeprocess.

According to trophic requirements the bacteria involved can be conveniently divided into three groups as follows.

Hydrolytic bacteria - acidogens: These bacteria hydrolyze the substrate (macromolecule) into short-chain organic acids and other small molecules, which can be taken up and converted into soluble short-chain organic molecules, e.g., carbohydrates are converted into low-chain fatty acids, alcohols, hydrogen and carbon dioxide under anaerobic condition. Strict anaerobes are composed most part of this group of bacteria. The generation time of these bacteria is 2 to 3 hours. The principle intermediate compounds resulting from conversion of the substrate during acid fermentation are acetate (CH_3COOH), propionate (CH_3CH_2COOH), butyrate ($CH_3CH_2CH_2COOH$), hydrogen gas (H_2), carbon dioxide (CO_2), lactate ($CH_3CHOHCOOH$), formate ($HCOOH$), ethanol (CH_3CH_2OH), valeric acid ($CH_3CH_2CH_2CH_2COOH$), isovaleric acid ($(CH_3)_2CHCH_2COOH$), and caproic acid ($CH_3CH_2CH_2CH_2CH_2COOH$). The distribution of final product depends on the species of acidogenic bacteria and on the environmental conditions such as pH and temperature.

ObligateHydrogenProducingAcetogens(OHPA): Thisgroupconvertscompoundformed in the first stage into acetic acid and hydrogen. Low hydrogen pressure favours these reactions [Harper and Pohland,1986].

e.g. Propionate $CH_3CH_2COOH + 2H_2O \rightarrow CH_3COOH + CO_2 + 3H_2$

$\Delta Go = 76.1$ KJ/mole

Butyrate $CH_3CH_2CH_2COOH + 2H_2O \rightarrow 2CH_3COOH + 2H_2$

$\Delta Go = 48.1$KJ/mole

From the viewpoint of the thermodynamics, a negative value of free energy change is necessaryforanyreactiontoproceedwithoutinputofexternalenergy.Thistheoryapparently suggests that hydrogen producing acetogenic bacteria cannot obtain energy for growth from these reactions. However, the value of free energy change in the actual environment surrounding the bacteria, $\Delta G'$, is different from that of $\Delta Go'$ and depends on the concentrationsofsubstratesandproductsasfollows[HarperandPohland,1986]:

$$\Delta G' = \Delta Go' + RT_A \ln \frac{[P_1].[P_2]....}{[S_1].[S_2]....} \quad \text{...} (1)$$

Where,

$\Delta G'$ = free energy change at pH = 7 (kJ/mol),

$\Delta Go'$=standardfreeenergychangeatpH=7(kJ/mol), R =

gas constant = 0.082L.atm/mol.^{o}K,

T_A = temperature (^{o}K),

$[P_1].[P_2]...$ = product concentration (mol/L or atm), and

$[S_1].[S_2]...$ = substrate concentration (mol/L or atm).

Only low partial pressure of hydrogen can give negative values of ΔG' in above equations, because substrate concentration cannot be so high and acetate concentration is not so low in anaerobic reactors. This shows that extremely low partial pressure of hydrogen is essential for hydrogen producing acetogenic bacteria although they themselves produce hydrogen. Experimentally it was found that the hydrogen partial pressure higher than 5×10^{-3} atm ceased the degradation of propionate by hydrogen producing acetogenic bacteria [Hanaki et al., 1985]. Based on thermodynamics associated with this reactions Harper and Pohland [1985] indicated that propionic acid oxidation to acetate becomes favourable only at hydrogen partial pressure below 10^{-4} atm, while, butyric acid oxidation becomes favourable at 10^{-3} atm H_2 orbelow.

Hydrogen utilizing methanogenic bacteria can serve such a thermodynamically favourable conditions for hydrogen producing acetogenic bacteria in anaerobic reactors, thus, the activity of hydrogen producing acetogenic bacteria depends on the existence of methanogenic bacteria. Hydrogen utilizing methanogens receive hydrogen as a substrate from hydrogen producing acetogenic bacteria. The interrelationship between these two groups of bacteria is called interspecies hydrogen transfer, which also exists between acidogenic and methanogenic bacteria. Acidogenic bacteria produce more hydrogen and acetate than propionate or lactate and obtain more energy under low hydrogen partial pressure which is kept by methanogenic bacteria. The interspecies hydrogen transfer is favourable but not essential for acidogenic bacteria, while, it is indispensable for hydrogen producing acetogenic bacteria.

Methanogenic bacteria - methanogens: These bacteria produce methane. The doubling time of these bacteria is 2 - 10 days. These are further divided into two groups as:

a) Hydrogen utilisers(lithotrophs)

$CO_2 + 4H_2 \rightarrow CH_4 + 2H_2O$ convert ADP to ATP

b) Acetic acid users(acetotrophs)

$CH_3COOH \rightarrow CH_4 + CO_2$ produce 0.25 mole ofATP

The methane producing bacteria are strict anaerobes which are extremely sensitive to changes in temperature and pH. These bacteria are active in two temperature zones, namely, in the mesophilic range (30°C - 35°C) and in the thermophilic range (50°C - 60°C). However, anaerobic processes have been operated at 15°C successfully when sufficient residence time for these bacteria was provided.

The majority of methanogens in anaerobic wastewater treatment and natural anaerobic environment utilize hydrogen and single carbon compounds as substrates for methane production. In addition there are two known genera of methanogens which can utilize the two-carbon compound, acetic acid. These include species of *Methanosarcina* and *Methanothrix (Methanosaeta*). The *Methanothrix species* are unable to use hydrogen in combination with CO_2 and these are non-hydrogen-oxidizing acetotrophs (NHOA). In contrast, *Methanosarcina* can utilize H_2/CO_2 as well as acetate, carbon monoxide, methanol, and methylamines as growth substrates. Due to their ability to use both H_2/CO_2 and acetate, these bacteria are classified as Hydrogen Oxidizing Acetotrophs (HOA). Hydrogen-Oxidizing Methanogens (HOM) do not cleave acetate, but utilize H_2/CO_2 and formate as substrates [Harper and Pohland, 1986]. The HOA are unique in their capability to utilize multiple (one and/or two carbon) substrates. This ability affords a higher potential for survival when competing with sulfur reducing bacteria (SRB) and nitrate reducing bacteria (NRB) for hydrogen and acetate. At hydrogen partial pressure $>10^{-4}$atm, HOA use H_2/CO_2 in favour of acetate, whereas acetate cleavage by NHOA is unaffected by hydrogen.

NHOA have a much higher affinity for acetate than the HOA. NHOA may outcompete HOA at acetate concentrations below 50 mg/L, while above 250 mg/L acetate, the HOA are more competitive [Speece et al., 1983]. As a result of this comparative kinetics *Methanothrix*(NHOA) may be found in reactors with lower organic loading. *Methanosarcina* are more predominant in low retention time reactors such as in the lower reaches of plug flow anaerobic filters and in two phase reactor system.

Oxidation of reduced organic products to bicarbonate and acetate also occurs due to NRB and SRB. Higher organic waste conversion rates may be available through SRB than through methanogenesis. Moreover, SRB and NRB are not limited to one- and two-carbon substrates, as are methanogens. However, from process engineering perspective, such an approach has disadvantages, including the loss of energy available from methane and the production of hydrogen sulphide or ammonia. Since, sulphide and ammonia are much more soluble than methane, their dissolved components can contribute significantly to effluent COD [Harper andPohland, 1986]. However, this approach may hold possibilities for reducing propionic acid and hydrogen, as well as acetic acid in a stressed reactor, in order to more rapidly reestablish the equilibrium with the existing hydrogen removal system.

21.4 Factors Affecting AnaerobicDigestion

Development of anaerobic process technology is dependent on a better understanding of the factors that are associated with the stability of the biological processes involved. Process instability is usually indicated by a rapid increase in the concentration of volatile acids in the first stage of the reaction. Low pH with a concurrent reduction in methane gas production indicates the methanogenesis more susceptible to upset. Acclimatization of the microbes to a substrate may take 3 to 8 weeks. Sufficiently acclimated bacteria show greater stability towards stress-inducing events such as hydraulic overloads, fluctuations in temperature, volatile acid and ammonia concentrations, etc. Several environmental factors can affect anaerobic digestion such as specific growth rate, decay rate, gas production, substrate utilization, etc. The environmental factors of primary importance arediscussed below.

21.4.1 pH, Acidity andAlkalinity

Methanogenic microorganisms are susceptible to the minute changes in the pH values. Optimum pH range of 6.6–7.6 is considered favourable for the methane producing bacteria, which cannot tolerate the fluctuations. The non-methanogenic bacteria do not exhibit such strong sensitivity for environmental conditions and are able to function in a range of pH form 5–8.5. The pH maintained inside the reactor, due to the process results from the interaction of the carbon dioxide-bicarbonate buffering system and volatile acids-ammonia formed by the process. It is necessary to prevent the accumulation of acids to a level, which may become inhibitory to the methanogenic bacteria. For this, it is important that there should be sufficient buffering capacity present in the reactor, which may prevent the reactor from souring. Although, the carbonates and bicarbonates of sodium and calcium are required to be added to the digesters to provide buffering action, lime (Calcium hydroxide) is most commonly used for this purpose. Only the unionized volatile acids in the concentration range of 30 - 60 mg/L aretoxic.

21.4.2 Temperature

As in all biological processes, anaerobic processes are affected by temperature. The higher the temperature, higher is the microbial activity until an optimum temperature is reached. A further increase of the temperature beyond its optimum value results in steep decrease in activity. Anaerobic process can take place over a wide range of temperatures (4 – 60^0C). Onceaseffectivetemperatureisestablished,smallfluctuationscanresultinaprocessupset. Although most of the sludge digester are operated in the mesophilic range (30 – 40^0C), methanogenesis can occur at temperatures as low as 12 to 15^0C. The effect of increasing temperatureonbiochemicalreactionrateintherangeof4–25^0Cisprofound.

The optimum temperature for growth of anaerobic microorganisms is 35^0C or greater. Although anaerobic digesters have been reported to operate at substantially lower temperatures, such as 20^0C, anaerobic growth under these temperature conditions is slow requiring prolonged start-up time and difficulties in operation. In situations where reactor's operatingtemperatureislow,start-upwillbebenefitedifinitiatedatapproximately35^0C.At temperature of less than 25^0C, the digestion rate decreases sharply and conventional anaerobic reactors in operation at ambient temperatures in cooler climates may require detentiontimesofasmuchas12weeksforthetreatmentofsewagesludges.

Themajorityofindustrialdigestersystemsoperateinthemesophilicrangeof30–40^0C.Itis probablethatincreaseinmicrobialreactionratesattheelevatedtemperaturesofthermophilic processes (50 – 60^0C), and hence decrease in SRT may prove advantageous under some circumstances. However, lack of stability in thermophilic municipal waste treatment can occur. Thermophilic digestion is most practical where wastewater stream to be treated is dischargedatanextremelyhightemperatureandthedigesterispresentonsite.

Inpsycrophilic,mesophilic,orthermophilicranges,uniformityoftemperatureovertheentire vessel contents is of paramount importance to anaerobic digestion. Temperature change of evenafewdegreescanresultinamarkedupsetinmicrobialmetabolismandrapidalterations in reactions in the reactor and may necessitate several days for the recovery. A consistent temperature throughout the system can be provided by adequate mixing of the reactor by paddle, gas sparging, or flow over heatexchangers.

21.4.3 Nutrients

Anaerobicwastewatertreatmentprocessesareoftenusedforindustrialwastewithonlyminor amount of nutrients present. This might result in nutrient deficiency, unless additional nutrients are supplemented. Often the COD/N ratio and COD/N/P ratio is used to described the nutrient requirements. Optimum N/P ratio can be considered to be 7. The theoretical minimum COD/N –ratio is considered to be 350/7. A value around 400/7 is considered reasonable for high rate anaerobic processes (operated in SLR of 0.8 – 1.2 kg COD /kg VSS.d). For low rate processes (<0.5 kg COD /kg VSS.d) the COD/N-ratio has been observedtobeincreaseddramaticallytovaluesof1000/7ormore[VandenBergandLentz, 1980].

Otherthannitrogenandphosphorous,tracemetalsalsoareessentialforanaerobicprocesses. Thepresenceoftracemetalssuchasmolybdenum,selenium,tungstenandnickelisprobably necessary for the activity of several enzyme systems. When these trace elements are not presentinthewastewater,additionofnickel,cobalt,andmolybdenumcanincreasemethane production and allow greater volumes of wastewater to be effectively treated by decreasing the reactor residencetime.

21.4.4 InhibitorySubstances

Inhibition of the anaerobic digestion process can be mediated to varying degrees by toxic materials present in the system. These substances may be components of the influent wastewater or byproducts of the metabolic activities in the digester. Inhibitory toxic compoundsincludesulphides,consequentialintheprocessingofwastefromsourcessuchas molasses fermentation, petroleum refining and tanning industries. Volatile acid and other microbial products can accumulate and inhibit reactor-buffering capacity. Inhibition may also arise as the consequence of the increased levels of ammonia, alkali, and the alkaline earth metals, and heavy metals in thesystem.

VolatileAcidsInhibition:Anaerobicreactorinstabilityisgenerallyevidentbyamarkedand rapidincreaseinvolatilefattyacidsconcentrations;thisisfrequentlyindicativeofthefailure of the methanogenic population due to other environmental disruptions such as shock loadings, nutrient depletion or infiltration of inhibitory substances. Acetate has been described as the least toxic of the volatile acids, while propionate has often been implicated as a major effecter of digesterfailure.

The inhibition by the volatile acids at acidic pH values can be attributed to the existence of unionized VFAs in significant quantities in the system. The undissociated nature of these acids allow them to penetrate the bacterial cell membrane more efficiently than their ionized counterparts, and once assimilated, induce an intracellular decrease in pH and hence a decrease in microbial metabolic rate. The resulting VFA concentration in the reactor should be maintained below 500 mg/L at any point of time and preferably below 200 mg/L for optimum performance.

Ammonia – Nitrogen Inhibition: Although ammonia is an important buffer in anaerobic processes, high ammonia concentration can be a major cause of operational failure. In reactor system that has not previously been acclimated to high ammonia loadings, shock loadings of high ammonia concentration generally caused rapid production of VFAs such that the buffering capacity of the system may not be able to compensate for the decrease in pH. Further depression of alkalinity and reduction of pH may result in reactor failure. Inhibition is indicated by a decrease in gas production and an increase in volatile acid formation.

Sulphide Inhibition: The sulphate and other oxidized compounds of sulphur are easily reduced to sulphide under the conditions prevalent in anaerobic digesters. Sulphur-containing amino acids of protein can also undergo degradation to sulphide. These compounds are of significance when anaerobic treatment is considered for industrial processes which tend to produce large quantities of sulphides in their waste stream. These sulphides formed by the activity of reactor microorganisms may be soluble or insoluble, depending upon their associated cations. When the salts formed are insoluble, they have negligible effects on digestion. Iron addition, for example can suppress sulphide inhibition by removing S^{2-} ion from solution by precipitation.

Desulfovibrio and other sulphate-reducing genera form sulphides from sulphates and some of the fermentative microorganisms utilize the sulphur containing amino acids to produce sulphides. Sulphide concentration in excess of 200 mg/L in a digester at 35 ^{0}C, with continuous feeding and mixing, produced severe inhibitory effects including the complete cessation of gas production [Parkin and Speece, 1983]. All the heavy metals, with the exception of chromium, form insoluble sulphide salts and thus can be removed from solution by sulphide present in the system by precipitation. Free sulphide can also be eliminated as hydrogen sulphide by vigorous gas production.

Heavy Metals Inhibition: The most common agents of inhibition and failure of sewage sludgedigestersareidentifiedtobecontaminatingheavymetals.Heavymetalsinthesoluble state are in general regarded to be of more significance to reactor toxicity than are insoluble forms. Anaerobic digestion also reduces the valence states of some heavy metals. Both copperandironmaybereducedfromthetrivalenttodivalentstate.Thisreducesthequantity of the precipitating agent, such as sulphide, necessary for the removal of the metal ion from solution. The heavy metals can be removed from anaerobic systems by adsorption. Those digesters, such as the CSTR configuration; which are tolerant to the wastes containing high levelsofsuspendedsolidsareeffectiveinmetalremoval,providedsufficientadsorptionsites arepresent.Themetalslikecopper,chromium,nickel,leadcaninducetoxicityinthereactor when present in higher concentration, and acceptable concentration in the wastewater to be treated differs from metal tometal.

21.5 Merits of Anaerobic DecompositionProcess

It has been recognized that the anaerobic treatment is in many ways ideal for wastewater treatment and has several merits mentioned as below:

- A high degree of wastestabilization;
- A low production of excess biological sludge and this sludge can be directly dried on sludgedryingbedwithoutfurthertreatmentduetobetterdewateringability;
- Low nutrient requirements, hence anaerobic treatment is attractive for the treatment of wastewater where external nutrient addition isrequired;
- Nooxygenrequirement,hencesavinginpowerrequiredforsupplyofoxygeninaerobic methods;
- Production of valuable byproduct, methanegas;
- Organic loading on the system is not limited to oxygen supply hence higher loading rate as compared to aerobic processes can beapplied.
- Less land required as compared to many aerobicprocess.
- Non-feedconditionsforfewmonthsdonotaffectadverselytothesystemandthismakes it attractive option for seasonal industrial wastewatertreatment.

Quantity of biological solids produced in the anaerobic systems per unit weight of organic materialismuchlessthanthatinaerobicsystems.Thisisamajoradvantageoftheanaerobic process as the quantity of sludge for ultimate disposal is reduced. This is a result of conversion of volatile solids present, to the high energy level end products such as methane, carbon dioxide and water. Methane has a definite economic value as a fuel, and it is used as a source of energy for both heat and power in many installations.

Another major advantage is the loading potential. Aerobic processes are restricted in maximum organic loading rate by the inability to transfer oxygen at the rate sufficient to satisfy the oxygen demand of the systems. Such limitation in organic loading rate is not there for anaerobic processes. The stabilized sludge from anaerobic process may be free from strong or foul odours and can be used for land application as ultimate disposal because the digested sludge contains sufficient nutrients required for plants. Pathogens are also destroyed to a high degree during the thermophilic anaerobic process. Due to large retention time and consequent low growth rate, the cell yield is also extremely low; thus, most of the carbon in the waste is available for methanogenesis and under normal circumstances the yield of methane would, on an average, be 0.33 – 0.36 m^3per kg COD utilized at 35^oC and atmospheric pressure.

However, anaerobic treatment processes are not largely being implemented, because of many factors. Anaerobic microorganisms, especially methanogens have slow growth rate. At lower HRTs, the possibility of washout of biomass is more prominent due to higher upflow velocity. This makes it difficult to maintain the effective number of useful microorganisms in the system. To maintain the population of anaerobes, larger reactor volume or higher HRTs with low upflow velocity is required. This may ultimately provide longer SRTs more than 40 days for high rate systems. Thus, provision of larger reactor volume or higher HRTs ultimately leads to higher capital cost. Low synthesis / reaction rate hence, long start-up periods and difficulty in recovery from upset conditions are some of the notable disadvantages. Special attention is therefore required towards controlling the factors that affect process adversely; importantly among them being environmental factors such as, temperature, pH, and concentration of toxic substances. Hence, skill supervision is required for operating anaerobic reactor at optimal performance.

15 : Anaerobic Wastewater Treatment Processes

21.8 Design Procedure for UASB Reactor

The UASB reactor can be designed as circular or rectangular. Modular design can be preferred when the volume of reactor exceeds about 400 m^3. It is necessary to select proper range of operating parameters for design, such as OLR, SLR, superficial liquid upflow velocity (referred as liquid upflow velocity), and HRT. The literature recommendations for all these parameters and design procedure to account these recommendations are given below.

21.8.1 Organic Concentration and Loading

For COD concentration in the range 2 to 5 g/L, the performance of the reactor depends upon the loading rate and is independent of influent substrate concentration. For COD concentration greater than 5 g/L, it is recommended to dilute the wastewater to about 2 g COD/ L during primary start-up of the reactor. Once, the primary start-up of the reactor is over with granulation of sludge, loading rates can be increased in steps to bring the actual COD concentration of the wastewater. The loading above 1-2 kg COD/m^3.d is essential for proper functioning of the reactor. For primary start-up the optimal loading rates for getting high COD removal efficiency (about 90%) within short start-up time, coupled with generation of good quality granular sludge, are OLR between 2.0 and 3.6 kg COD/m^3.d and SLR between 0.15 and 0.25 kg COD/kg VSS.d (Ghangrekar et al., 1996). The OLR to be used for design of UASB reactor for different temperature is provided by Lettinga and Hulshoff (1991). In general, for temperature between 15 and 35^0C, the reactor can be designed for loading between 1.5 to 18 kg COD/ m^3.d. Lower OLR should be preferred for low temperature and higher OLR can be adopted for high temperature.

For sewage treatment, the design of reactor at higher loading rate is not possible due to limitations of upflow velocity, and maximum loading of about 2 to 3 kg COD/m^3.d can be adopted for design. Similarly, for high strength wastewater, such as distillery, satisfying minimum velocity criteria and maximum HRT limit is difficult. Therefore, categorization of wastewater based on COD concentration is necessary for generalizing the design procedure of UASB reactor to meet the recommended operating conditions to the maximum extent. Thus, the COD concentration of the wastewater is suitably divided in four categories. It has been proposed to adopt loading conditions as recommended in the Table 21.2, for design of UASB reactor depending on the average COD concentration of the raw wastewater. These loading rates recommended are suitable for temperature about 30^oC. For higher temperature, the loading rates can be slightly increased and for low temperature these design loading rates can be reduced.

Table 21.2 Recommended loading range for design of UASB reactor based on COD concentration at average flow

Category of wastewater	COD concentration, mg/L	OLR, Kg COD/ m^3.d	SLR, Kg COD/ kg VSS.d	HRT, Hours	Liquid Upflow Velocity, m/h	Expected Efficiency, %
Low strength	Up to 750	1.0 - 3.0	0.1 - 0.3	6 – 18	0.25 – 0.7	70 –75
Medium strength	750 – 3000	2.0 – 5.0	0.2 – 0.5	6 – 24	0.25 – 0.7	80 – 90
High strength	3000 – 10,000	5.0 – 10.0	0.2 – 0.6	6 – 24	0.15 – 0.7	75 – 85
Very high strength	> 10,000	5.0 – 15.0	0.2 – 1.0	> 24	---	65 – 75

(Source: Ghangrekar et al., 2003)

21.8.2 Reactor Volume

Based on the higher suitable value of OLR, for given COD concentration, the volume of reactor required is to be worked out as:

Volume = (Flow Rate x COD concentration)/OLR .... (2)

For the suitable SLR values for that COD range (Table 21.2), the volume of sludge required can be worked out considering the average concentration of VSS between 25 and 35 g/L for medium and high strength wastewater, and 15 to 25 g/L for low strength wastewater. This volume of sludge should be less than 50% of the reactor volume, worked out based on OLR, to avoid overloading of the reactor with respect to SLR. If the volume is not meeting the requirements, the OLR can be reduced to increase the volume. The volume of the reactor is thus, finalized to meet both the requirements. For this volume, the HRT should not be allowed to be less than 6 h for any type of wastewater and generally, it should be less than 18 h to reduce volume and hence, cost of the reactor. For very high strength of the wastewater, COD greater than 10,000 mg/L, it may not be possible to meet this requirement, hence, under such situation the HRT may be allowed to exceed even 24 h and as high as 200 h.

21.8.3 Superficial Liquid Upflow Velocity

Higher upflow velocities, favors better selective process for the sludge and improve mixing in the reactor. However, at very high upflow velocity, greater than 1.0 to 1.5 m/h, the inoculum may get washed out during start-up or during normal operation granules may get disintegrated, and the resulting fragments can easily wash out of the reactor. The maximum liquid upflow velocity allowed in design should not exceed 1.2–1.5 m/h. Upflow velocities as 0.25 to 0.8 m/h are favorable for granule growth and accumulation, during normal operation of the reactor and maximum upflow velocity up to 1.5 m/h at peak flow conditions for short duration can be used in design.

21.8.4 Reactor Height and Area

The reactor should be as tall as possible to reduce plan area and to reduce cost of land, GLS device, and influent distribution arrangement. The height should be sufficient to provide enough sludge bed height to avoid channelling and to keep liquid upflow velocity within maximum permissible limits. In order to minimise channelling the minimum height of the sludge bed should be about 1.5 to 2.5 m. For this reason, the minimum height of the reactor should be restricted to 4.0 m, to conveniently accommodate sludge bed, sludge blanket and GLS separator. The maximum height of the reactor can be about 8 m. The height of the reactor adopted in practice is usually between 4.5 and 8 m and 6 m is the typical height used for UASB reactors.

While designing, initially suitable height of the reactor (about 6m) can be chosen, and superficial liquid upflow velocity is to be worked out as height/ HRT. It is recommended to adopt upflow velocity of 0.7 m/h at average flow and 1.0 m/h to 1.2 m/h at peak flow. Accordingly, if the upflow velocity exceeds the maximum limits height of the reactor can be reduced in steps up to minimum of 4 to 4.5 m. If this is not possible in the applicable range of height, HRT shall be modified and fresh reactor volume and OLR shall be worked out. For low strength wastewater, the maximum liquid upflow velocity becomes limiting and for very high strength wastewater very low velocity (less than 0.1 m/h) is required while designing the UASB reactor. Under certain situations, the revised OLR may be less than the initial OLR recommended. It is advisable to allow lowering of OLR in such situations to control upflow velocity in the reactor for proper performance of the reactor.

After these iterations for volume and height, the plan area can be worked out and suitable dimensions of the reactor can be adopted. Generally, the maximum diameter or side length of single reactor should be kept less than 20 m. Before finalizing the dimensions of the reactors, it is necessary to consider the dimensions required for GLS separator, because to accommodate the GLS separator meeting all requirements, it may be necessary to alter height and plan area of the reactor.

21.8.5 Gas-Liquid-Solid (GLS)Separator

In order to achieve highest possible sludge hold-up under operational conditions, it is necessary to equip the UASB reactor with a GLS separator device. The main objective of this design is to facilitate the sludge return without help of any external energy and control device. The guidelines for shapes and design of GLS separator are given by Lettinga and Hulshoff(1991). The GLS should be designed to meet the requirements such as, provision of enough gas-water interface inside the gas dome, sufficient settling area outside the dome to control surface overflow rate; and sufficient aperture opening at bottom to avoid turbulence due to high inlet velocity of liquid in the settler, and to allow proper return of solid back to the reactor. Due attention has to be paid to the geometry of the unit and its hydraulics, to ensure proper working of the GLSseparator.

Design of GLS separator: The shape of the GLS device considered in design is presented in Figure 21.4. The gas-water interface inside the dome is considered at the depth Δh from top of the dome. In the beginning, the height of GLS separator can be considered as 25% of the total reactor height. For estimating initially the number of domes required the angle of dome with horizontal can be assumed as 45^o, and base width of dome (Wb) can be calculated as $2(h+\Delta h)/\tan\theta$. The Δh is to be calculated as $(Wt/2)\tan\theta$, and initially the top width (Wt) can be considered as 0.2 to 0.3 m. The number of domes required for given diameter (or width for rectangular reactor) can be calculated by dividing width or diameter by W_B, and rounding this number. Where, W_B=Wb+Wa, and Wa can be considered as 0.2 m initially. After deciding the number of domes, the flow rate shared by each dome, is to be estimated in proportion to the base area of each dome, including aperture width, to the total area of the reactor.

Aperture width at bottom of gas dome: The area of aperture (Ap) required can be computed based on the maximum inlet velocity of liquid to be allowed. This area can be estimated as flow rate per dome for rectangular reactor (or central dome in case of circular) divided by maximum velocity to be allowed. The maximum inlet velocity of 3 m/h is safe for medium and high strength wastewater and for low strength wastewater lower inlet velocity should be preferred. The width of aperture (Wa) is to be calculated as aperture area divided by length (or in case of circular reactor by diameter) of the reactor. It is recommended to use minimum aperture width of 0.2 m and if the width required is greater than 0.5 m, then increase the number of dome by one and repeat earlier steps till it is less than 0.5 m.

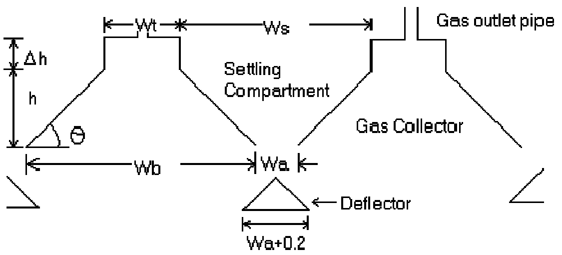


Figure 21.4. Details of the Gas-Liquid-Solid (GLS) Separator

Width at gas-water interface: The gas production expected in the reactor can be estimated based on the OLR selected for the design and expected COD removal efficiency in the range 70 to 90 percent. The methane production can be estimated as 0.35 m^3 /kg COD removed at ambient temperature and methane content of 70% in biogas. From this gas production the biogas collection per dome is to be worked out in proportion with percentage of area covered by the dome. The biogas loading at gas-water interface can be calculated as gas collection per dome divided by product of top width of gas collector (Wt) and length of the gas collector dome. The loading of biogas at gas-water interface should be kept less than 80 m^3gas/ m^2.d (about 3 m/h) (Ghangrekar et al., 2003). Initially the top width can be assumed as 0.3 m and for this width if the biogas loading is less than 3.0 m/h then adopt 0.3 m as top width. If the biogas loading is greater than 3.0 m/h, calculate the top width required. Generally, top width of 0.3 to 0.7 m can be adopted in design with maximum of 1.0 m. When even with maximum top width, if biogas loading is greater than 3.0 m/h reduce the height of GLS separation device to 20% and repeat the earlier steps of GLS separator design, with fresh number of domes. Even with reduction in height of GLS separator if these checks are not satisfying, provide additional layer of gas collector dome. When two or more layer of gas collectors are used the height of each layer can be 15 to 20% of the overall reactor height, with minimum height of each layer as 1.2 m and maximum up to 1.5 to 2.0 m. The fresh biogas collection per dome is to be worked out and further steps are repeated until all design conditions are satisfied.

Check for Surface overflow rate: The width of the water surface (Ws) available for settling of solids for each gas dome, at top of the reactor, can be calculated as difference of base width of dome ($W_B = W_b + W_a$) and Wt. The corresponding surface overflow rate is calculated as hydraulic flow rate per dome divided by product of length (or diameter) and Ws. It is recommended that the surface overflow rate for effective settling of solids back to thereactorshouldbelessthan20m^3/m^2.dataverageflowandshouldbelessthan36m^3/m^2.d under peak flow conditions. If the calculated surface overflow rate is meeting these criteria the design of the GLS separator is final. When it is exceeding the limits recommended, it is advisable to reduce the height of the reactor, thus, for same volume of the reactor more plan areawillbeavailable.Whentheheightofthereactorisreducedallearlierstepsfordesignof GLSseparatorshouldberepeatedtosatisfyalldesigncriterions.Theminimumheightofthe reactor should be restricted to 4.0 m (preferably 4.5 m). Once, all the design criteria are satisfied the angle of inclination of the gas collector dome with horizontal (θ) can be calculated as $\theta = \tan^{-1}[\ 2h/\ (Wb - Wt)]$.

Baffleofsufficientoverlap(0.1to0.2m)shouldbeprovidedbelowthegascollectorinorder to avoid entry of biogas in the settling compartment. The diameter of the gas exhaust pipes should be sufficient to guarantee easy removal of the biogas from the gas collection cap, particularlyincaseoffoaming.Generally,lowerreactorheightisrequiredforUASBreactor treating sewage. Under certain situation, particularly for very low strength of wastewater, evenwithreductionofheighttotheminimummaynotmeetalldesignrequirements.Insuch cases the OLR adopted for design can be reduced to provide greater volume of the reactor and hence more plan area to meet the entire designcriterion.

21.8.6 Effluent CollectionSystem

The effluent has to leave the UASB reactor via number of launders distributed over entire area discharging to main launder provided at periphery of the reactor. The effluent launders can be designed in such a way that the weir loading (m^3/m.d) should not exceed the design criteria of secondary settling tank (*i.e.*185 m^3/m.d). The width of the launders may be minimum 0.20 m to facilitate maintenance. The depth of the launder can be worked out as open channel flow. Additional depth of 0.10 to 0.15 m shall be provided to facilitate free flow. On both sides of the launders ‘V’ notches shall be used. When effluent launders are provided with scum baffles, the ‘V’ notches will be protected from clogging as the baffles retain the floating materials. A scum layer may form at the top of reactor and sludge accumulation can occur in the launder hence, periodical cleaning of launders and removal of scum should be carried out.

21.8.7 Design of Feed Inlet System

It is important to establish optimum contact between the sludge available inside the reactor and wastewater admitted, and to avoid channeling of the wastewater through sludge bed. Hence, proper design of inlet distribution system is necessary. Depending on topography, pumping arrangement, and likelihood blocking of inlet pipes, one could provide either (i) gravity feed from top (preferred for wastewater with high suspended fraction), or (ii) pumped feed from bottom through manifold and laterals (preferred in case of soluble industrial wastewaters). The rough guidelines for the number of feed inlet points required in UASB reactor is presented by Lettinga and Hulshoff (1991) for different concentration of the sludge inside the reactor and applicable loading rates. In general, the area to be served by each feed inlet point should be between 1 and 3 m^2. Lower area per inlet point (1 m^2) is to be adopted for reactor designed for OLR of about 1 kg COD/m^3.d, and higher area (2 to 3 m^2) per inlet point can be provided to the reactor designed for OLR greater than 2 kg COD/m^3.d. Apart from the number of feed inlet points, the minimum and maximum outflow velocity through the nozzles should also be given due consideration while designing. This outflow velocity through nozzles can be kept between 0.5 and 4.0 m/s. The equation of ‘condition for maximum power transfer through nozzle’ can be used for working out nozzle or inlet pipes diameter. The clogging of the nozzles may represent serious problem resulting in uneven distribution of the wastewater over reactor bottom, particularly when treating partially soluble wastewater. Hence, arrangements should be made for cleaning or flushing the inlet system.

21.8.8 Other Requirements

It is necessary to keep provision for removal of excess sludge from the reactor. Although, the excess sludge is wasted from about middle height of the reactor, it is also necessary to make arrangement at bottom of the reactor. In addition, 5 to 6 numbers of valves should be provided over reactor height to facilitate sampling of the sludge. For treating high strength wastewateritisrecommendedtoapplyeffluentrecycle,inordertodiluteCODconcentration and to improve contact between sludge and wastewater. For treating wastewater with COD concentration greater than 4 - 5 g/L, it is recommended to apply dilution during start-up, for proper granulation of sludge inside UASB reactor. Auxiliary equipment has to be installed for addition of essential nutrients, and alkalinity for control of pH of the influent. The other equipments to be provided are for measurement of pH, temperature, influent flow rate, and gas productionrate.

Example: 1

DesignanUASBreactorfortreatmentof4MLDsewagehavingBODof200mg/LandCOD of500mg/L.Theaverageminimumtemperatureofwastewaterinwinteris about20^{o}Cand maximumtemperatureinsummeris35^{o}C.Thewastewatercontains80mg/Lsulphate.

Solution

<u>Reactor Volume (V)</u>

RangeforHRTis6to18h

V = Q x HRT

ProvideHRTof8h

$V = 4 \times 10^3 x \ \frac{1}{24} x 8$

$= 1333.33 m^3$

<u>Check for OLR</u>

Range for OLR = 1 to 3 kg COD/ m^3.day

$$OLR = \frac{\text{flow per reactor X COD}}{\text{volume of each reactor}}$$

$$= \frac{4 \times 10^3 \times 500 \times 10^{-3}}{1333.33}$$

= 1.5 kg COD/ m^3.day

<u>Check for SLR</u>

Range for SLR = 0.1 to 0.3 kg COD/kg VSS/day

$$\text{SLR} = \frac{\text{flow per reactor X COD}}{\text{volume of sludge x VSS inreactor}}$$

Volumeoccupiedbysludgebedshouldbeabout50%ofreactorvolume

Assume VSS in reactor sludge = 25g/l

$$\text{SLR} = \frac{4 \times 10^{3} \times 500 \times 10^{-3}}{1333.33 \times 0.5 \times 25}$$

$$= 0.12 \text{ kg COD/kg VSS.day}$$

Check for MCRT

Range for MCRT = 40 - 100 days

$$\text{MCRT} = \frac{\text{mass of inoculumsludge}}{\text{massof sludge wasted per day}}$$

Assume sludge in effluent to be 100 mg/L

$$= \frac{1333.33 \times 0.5 \times 25}{4 \times 10^{3} \times 100 \times 10^{-3}}$$

$$= 41.67 \text{ days}$$

Height of the reactor

Let us provide height of the reactor H = 4.5 m.

Check for upflow velocity

Upflowvelocity=H/HRT=4.5/8=0.562m/h(lessthan0.7m/h)

Area of thereactor

$$A = \frac{\text{volume of reactor}}{\text{height of reactor}}$$

$$= \frac{1333.33}{4.5} = 296.296 \text{ m}^2$$

ProvidelengthofthereactorL=19m,hencewidthB=15.6m

Design of GLSseparator

Height of dome = 0.25 x ht. of reactor

= 0.25 x 4.5

= 1.125 m

Provide 1.20 m height of the dome and 0.3 m free board above the water surface for gas collection.

Provide max liquid velocity at aperture i.e. inlet of the settler = 3 m/h

Area of opening at inlet of settler = $\frac{4000}{3 \times 24}$

= 55.56 m^2

Total width of opening required = $\frac{\text{Areaofopening}}{\text{width of reactor}}$

= $\frac{55.56}{15.6}$ = 3.56 m

Provide width of each gap = 0.4 m

No of gaps = $\frac{3.56}{0.4}$ = 8.9 say 9, provide 9 number of domes which will make 8 openings in the middle of the domes and two opening along the side wall.

Width of each aperture opening will be 0.395 m and along the wall it will be 0.198 m.

Henceprovidedeflectorbeamof0.59mbelowtheapertureopeningand0.3mbasewidth Provide 0.3 m width at top of thedome

Totalwidthofbaseofdomes=length–widthofeachgapxno.ofgaps–topwidthxno.of domes

= 19 – 0.395 x 9 – 0.3 x 9

= 12.745 m

Width of base of each dome = $\frac{12.745}{9}$ = 1.416 m

Angle of inclination =$\tan^{-1}\left(\frac{1.2}{0.71 - 0.15}\right) = 64.98^0$

Gas production

Methane production in litres = 1.28 x T (^{0}K) per kg of COD removed

= 1.28 x (273+30) per kg of COD removed

= 387.84 L per kg of COD removed

Let the COD removal efficiency of the system be 75%.

Hence, kg of COD removed = flow per reactor x COD x 0.75

$= 4 \text{ x } 10^{3} \text{x } 500 \text{ x } 10^{-3} \text{x } 0.75 \ /_{\text{day}}$

= 1500 kg/day

The total COD removed in the reactor = 1500 kg/day

But not all the organic matter present in the influent is carbonaceous. The influent also consistsofsulphateswhicharereducedtosulphidesandconsumeabout0.67kgofCODper kg ofsulphate

$SO_4^{2-} \rightarrow S^{2-}$

Assuming sulphate removal of 80%, the total sulphate reduction

$= 0.8 \text{ x } 4000 \text{ x } 80 \text{ x } 10^{-3}$

= 256 kg/day

COD consumed in sulphate reduction = 256 x 0.67 = 171.52 kg/day

Hence COD available for methane production = 1500 – 171.52 = 1328.48 kg/day

Also some portion of biogas will remain in soluble form in the reactor effluent due to high partialpressureofbiogasinsidethereactor.Typicallyabout16mg/Lofmethanewillbelost along with theeffluent.

Methane that can be collected = $1328.48 \text{ x } 0.38 – 4000 \text{ x } 16 \text{ x } 10^{-3} = 440.822 m^3$/day

Alsogascollectionefficiencyofthedomeswillbeabout85to90%,henceactuallymethane collected at 85% efficiency will be = $374.69 m^3$/day

<u>Check for biogas loading at gas-water interface</u>

Total biogas produced assuming methane content to be 70% = $\frac{1328.48 \text{x} 0.38}{0.70} = 721 \ m^3$/day.

Max gas loading rate = $3 \ m^3/m^2/h$

Area required = $\frac{\text{Volume of biogas produced}}{\text{Max gas loading}}$

$= \frac{721}{3 \text{ x} 24}$

$= 10 m^2$

Total top width = $\frac{\text{Area required}}{\text{width of UASB}}$

$= \frac{10}{15.6}$ = 0.64 m. Hence, the width required for each dome = 0.07 m which is less than 0.3 m provided.

Check for surface overflow rate (SOR)

Width available in the settling compartment i.e. outside the domes

= (19 – 0.3*9) = 16.3

Hence, SOR = 4000/(16.3*15.6) = 15.73 $m^3/m^2.d$ (Less than 20 $m^3/m^2.d$)

Questions

1. Explainthereactionsequenceinvolvedintheanaerobictreatmentofwastewater.
2. Describedifferenttypesofbacteriaandtheirroleinanaerobicdegradationoforganic matter to final endproduct.
3. Discussthefactorsthatcanaffecttheanaerobicreactorperformanceadversely.
4. Describeadvantagesanddisadvantagesofanaerobictreatmentofliquidwaste.
5. Describedifferenttypesofreactorsusedforanaerobictreatmentofwastewaters.
6. Whatishighrateanaerobicprocess?Namedifferenthighrateanaerobicreactors.
7. Explain different types of filters used in anaerobictreatment.
8. WiththehelpofschematicexplainUASBreactoranditsworking.
9. WhatisGLSseparator?WhatarethedesignguidelinesforGLSseparator?
10. Describe advantages of sludge granulation in UASBreactor.
11. Whyposttreatmentisnecessaryforanaerobicreactoreffluents?Whatposttreatment youwillrecommendaftertreatmentofsewageinUASBreactor?
12. Describe organic loading rates used for design of UASB reactor. How reactor height is important for proper functioning of UASBreactor?
13. DesignaUASBreactorfortreatmentof2MLDofsewagehavingCODof500mg/L and BOD of 250 mg/L. Make suitable assumptions for thedesign.

Answer

Q.13.

Assume HRT = 8 hr
Volume required = 666.67 m^3; provide 2 reactors of height of 5.6 m and L x W = 10 x 6 m.

16 : Sludge Management

22.1 Introduction

In the context of wastewater treatment residual is used to refer "sludge". The term sludge refers to the solids that are settled and separated during wastewater treatment. It is necessary to treat properly or dispose the sludge generated during the various stages of wastewater treatment like primary sedimentation, secondary sedimentation and sludge generated from advanced (tertiary) treatment, if any. The quantity of sludge generated depends upon the degree of treatment or quality of treated effluent required i.e., higher the degree of wastewater treatment, the larger the quantity of sludge to be treated and handled. Because of strict rules and regulations involving the handling and disposal of sludge, it has become necessary to reduce the volume of sludge in order to reduce the operating costs (approximately 50% of the plant cost) of treatment plants. Hence a properly designed and efficiently operated sludge processing and disposal system is essential to the overall success of the wastewater treatment plant. The sludge generated during the wastewater treatment can be classified into threecategories:

Primary Sludge: Sludge settled in primary settling tanks comes under this category which contains 3% to 7% solids out of which approximately 60% to 80% are organic. Primary sludge solids are usually gray in color, slimy, fairly coarse, and with highly obnoxious odors. This sludge is difficult to dewater without treatment, hence digestion is necessary. This type of sludge can be digested readily by aerobic or anaerobic bacteria under favorable operating conditions.

Secondary Sludge: This type of sludge from secondary settling tanks has commonly a brownish, flocculent appearance and an earthy odor. It consists mainly of microorganism containing 75% to 90% organic fraction and remaining inert materials. The organic matter may be assumed to have a specific gravity of 1.01 to 1.05, depending on its source, whereas the inorganic particles have high a specific gravity of about2.5.

Tertiary Sludge: The nature of sludge from the tertiary (advanced) treatment process depends on the unit process followed like membrane processes or chemical methods, etc. Chemical sludge from phosphorus removal is difficult to handle and treat. Tertiary sludge from biological nitrification and denitrification is similar to waste activated sludge.

The sludge is generated in the wastewater treatment plant in the form of already present settleable solids, when settled in the PST, and in the form of biological cell mass, generated in the biological secondary treatment that settled in SST. The water content of the sludge is very high, and solids constitute very small part of it. Therefore before final disposal further treatment is required for this sludge to reduce water content and oxygen demand. Sludge is stabilized to (i) reduce pathogens, (ii) eliminate odours, (iii) inhibit, reduce, or eliminate the potential for decomposition, and (iv) improve dewatering characteristics of the sludge to reduce volume for disposal. There are four means to eliminate this nuisance condition through stabilization. They are (1) biological reduction of volatile solids, (2) chemical oxidation of volatile solids, (3) addition of chemicals to make conditions not suitable for bacterial growth, (4) application of heat to disinfect or sterile thesludge.

Disposal of the sludge presents problems due to (i) the solids present are mostly organic and undergo decomposition, and (ii) volume of the sludge is many times than the solids constituents. Hence, further treatment is required for reducing volume of the sludge, stabilizing organic matter present in the sludge, and improving its filtration ability for easy dewatering. The reduction in volume of the sludge can be achieved by thickening, dewatering and drying; and stabilization of organic matter can be obtained by employing digestion (aerobic or anaerobic), incineration, composting, heat treatment, chlorine oxidation or lime stabilization. The flow sheet for biological sludge treatment is presented in the Figure22.1.

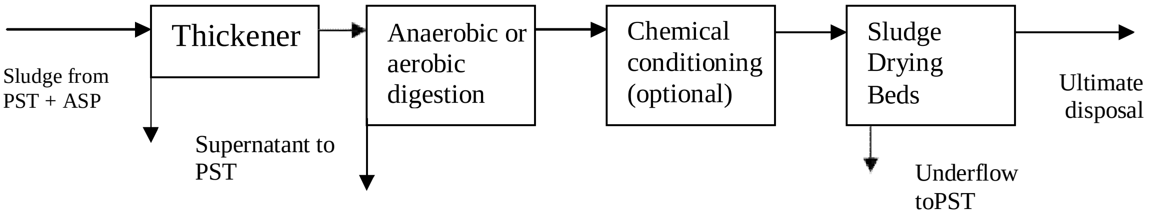


Figure 22.1 Flow sheet for biological sludgetreatment

Organic sludge originates from various sources such as PST, SST of ASP, SST of TF, etc. PST andSSTsludgeusuallyhave65%-75%volatilesolidswhilesludgefromTFofSSTwillhave 45 to 70% volatile solids. The dry solids content of primary and secondary sludge after thickening is about 4 to 6% and specific gravity of 1.01. The separation of water is difficultfrom undigested sludge. Fuel value of undigested sludge is about 6500 to 8000 BTU/lb (15100 to 18600 KJ/kg of dry solids). After digestion sludge have volatile solids of about 32 to 48%, dry solids of 8 to 13%, and specific gravity of 1.03 to 1.05 and fuel value is about 8100 to 9300 KJ/kg of dry solids. Digested sludge is easy to dewater. Approximately 99.8 % of coliform are destroyed from sewage sludge. Generally PST and SST sludge is thickened before digestion to reduce volume of sludge. Sludge treatment is an important aspect of wastewater treatment and account for 40 to 45 % of the capital and operatingcosts.

22.2 Sludge Thickening

Sludge thickening or dewatering is adopted for reducing the volume of sludge and increasing the solid contents. This will help in following:

(i) increasing the loading on the digester, requiring lesser digestervolume,
(ii) increase feed solids concentration to vacuumfilters,
(iii) economize transport and handling cost of sludge within the plant and finaldisposal,
(iv) minimize land required and handling cost for final disposal of the digested sludge on land,and
(v) save fuel if incineration ispracticed.

In sludge thickeners, greater amount of water is removed from the sludge than what could obtain from sedimentation tank. This reduces overall volume of the sludge considerably. The thickening of the sludge can be achieved either by gravity thickening, application of air floatation or by centrifugation.

22.2.1 Gravitythickening

Gravity thickening is accomplished in a tank similar in design to a sedimentation tank. This is most commonly used for concentrating the sludge for achieving saving in the digester volume and sludge handing cost. This is used for primary sludge and for combine primary and secondary sludge, and it is not suitable for ASP sludge alone. When the ASP sludge is more than 40% (weight ratio) of the total combined sludge, gravity thickening is not effective and other methods of thickening have to beconsidered.

Gravity thickeners can be operated either as continuous flow or fill and draw type, with or without chemical addition. The thickened sludge is withdrawn from the bottom of the tank and pumped to the digester. The supernatant is returned to the PST. Use of slow stirring improves efficiency. Continuous feed tanks are circular in shape with central feeding and overflow at the periphery. The side water depth is kept about 3.0 m. Due to relatively high concentration of the solids, as compared to PST or SST, the settling in thickeners will follow hindered settling in the beginning and compaction at later stage. Concentration of the underflow solids is governed by the depth of sludge blanket up to 1 m beyond which there is very little influence of the blanket. Thickeners are designed for hydraulic loading of 20 to 25 m^3/m^2.d. Loading rates lesser than 12 m^3/m^2.d are likely to give very high solids concentration, which may require dilution with plant effluent for transporting. The underflow solid concentration will increase with increase in detention time, and detention time of about 24 h will produce maximum compaction. During peak condition, lesser detention time is allowed to keep the sludge blanket sufficiently below the overflow weirs to prevent excessive solids carryover. The surface loading rates for various types of sludges are given in the Table22.1.

Table 22.1 Surface loading rates and solid concentration typically achieved in thickeners

Type of sludge	Solid surface loading (kg/m^2.day)	Thickened sludge solids concentration, % (g/L)
Separate sludge		
Primary	90 – 140	5 – 10 (50 to 100)
Activated	25 – 30	2.5 – 3.0 (25 to 30)
Trickling filter	40 – 45	7 – 9 (70 to 90)
Combined sludge		
Primary + Activated	30 – 50	4 – 8 (40 to 80)
Primary + Trickling filter	50 – 60	7 – 9 (70 to 90)

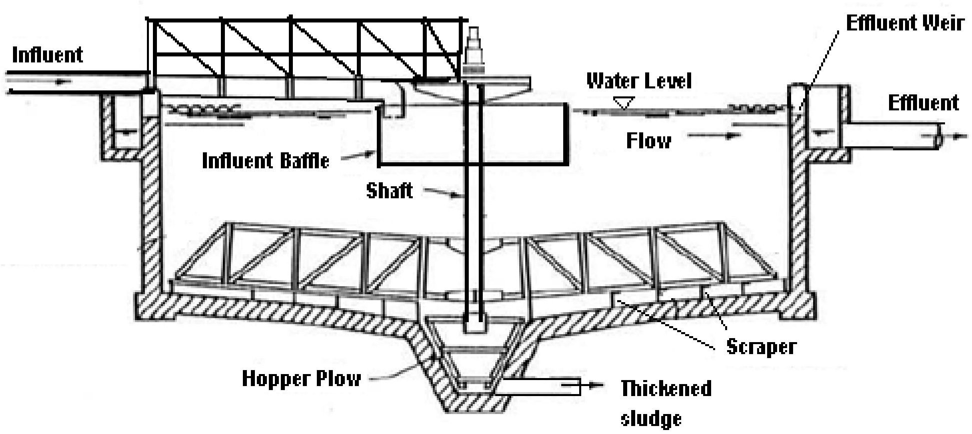


Figure 22.2 Schematic diagram of a gravity thickening unit

22.2.2 Air floatation

By applying air under pressure or vacuum the thickening of the sludge can be achieved. This is normally preferred for ASP sludge. This requires additional equipment, power for operation, skilled supervision for operation and maintenance, hence it is costly. However, better removal of oil and grease, solids, and odour control are the advantages offered by this method. Addition of alum, polyelectrolytes can increase the efficiency of the flotation unit. Alum will increase the sludge but polyelectrolyte will not increase the solids concentration but improves solids capture from 90 to 98%. The floatation units can be of two types (i) pressure type and (ii) vacuum type.

Pressure type floatation unit: In pressure type floatation unit, a portion of the subnatant is pressurized from 3 to 5 kg/cm^2 and then saturated with air in the pressure tank. The effluent from the pressure tank is mixed with influent sludge immediately before it is released into flotation tank (Figure 22.3). Once the pressure is released, excess dissolve air rises up in the form of extremely small air bubbles, attaching themselves to the sludge particles. This imparts buoyancy to the sludge particles and forms sludge blanket at the top, which is skimmed off, while the unrecycled subnatant is returned to the wastewater treatment plant. Dissolved air floatation (DAF) offers significant advantages in thickening light sludge such as activated sludge.

Typical design data of a DAF unit is given below:

1) Air pressure required: 280-550kPa
2) Retention tank pressure: 3-5kg/cm^2
3) Solid loading rate: 10-20 kgsolid/(m^2.d)

DAF thickening produces about 4% solids with a solids recovery of 85%. The sludge volume index, SVI, is also an important factor for DAF operation.

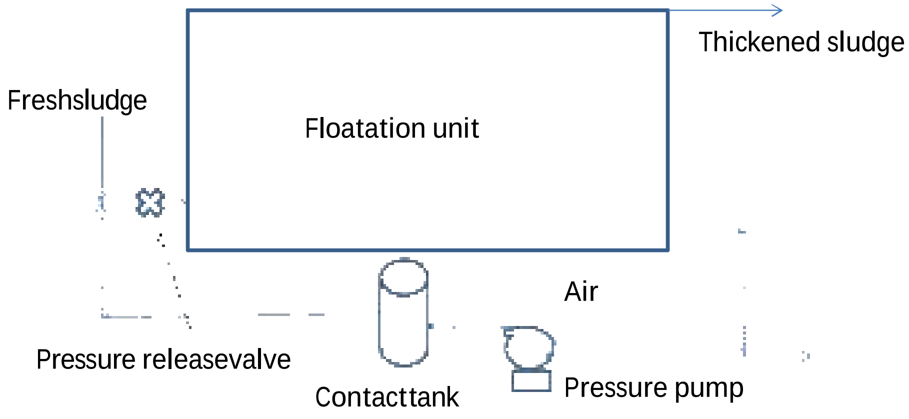


Figure 22.3 Working of pressure type air floatation unit

***Vacuum type floatation*:** The vacuum type floatation unit employs the addition of air to saturation and applying vacuum to the unit to release the air bubbles which float the solids to the surface. The solids concentrated at the surface are skimmed off.

22.2.3 Centrifugation

Thickening by centrifugation is used only when the land available is limited and sludge characteristics will not permit adoption of other methods. This will require high maintenance and operational cost. A centrifuge acts both ways to thicken and to dewater sludge. The centrifuge process separates liquid and solid by the influence of centrifugal force which is typically 50 to 300 times that of gravity.

22.3 Anaerobic Sludge Digestion

22.3.1 Process Fundamental

In anaerobic digestion process the organic material, in mixture of primary settled sludge and biological sludge from secondary clarifier, is converted to CH_4 and CO_2 under anaerobic conditions. This is carried out in an air tight reactor in absence of oxygen. Sludge is introduced continuously or intermittently and retained in the reactor for varying periods of time. Two basic processes involved in anaerobic digestion are liquifaction and gasification. The stabilized sludge which is withdrawn continuously or intermittently from the process, is non putrescible, and its pathogen content is also greatly reduced.

Anaerobic digestion is defined as being biological oxidation of degradable organic sludge by microbes under anaerobic condition. It occurs in absence of oxygen and organic matter acts as food source for microorganisms. Most microbes used in this digestion are obligate anaerobes or facultative type. This process is employed for treatment of the organic sludge.

During oxidation of organic matter anaerobically following reaction occurs

$$\text{Organicmatter} \xrightarrow[\text{(Anaerobic bacteria)}]{} CO_2 + CH_4 + \text{new cell} + \text{energy for cells} + \underset{(H_2S,\ H_2,\ N_2\text{etc.})}{\text{Other products}}$$

Microbial action by anaerobic bacteria consists of three stages as 1) liquification of solids, 2) digestion of soluble solids, 3) gas production. Organic acid forming heterotrophs use complex organic substrate such as carbohydrates, proteins, fats, oils and their degradation products and produce organic acids. The breakdown of three major organic matters is shown below:

Carbohydrates → Simplesugars → Alcohols,aldehydes → Organic acids

Proteins → Aminoacids → Organic acids +NH_3

Fatsandoils → organicacids

Most of these organic acids forming bacteria are facultative anaerobes easily found in soil and works in relatively wide pH range. Methane producing heterotrophs, obligately anaerobes, use organic acids in narrow pH range of 6.7 to 7.4 to produced CO_2 and CH_4.

$$\text{Organicacids} \longrightarrow \underset{(55\text{-}75\%)}{CO_2} + \underset{(35\text{-}45\%)}{CH_4} + H_2S,\ H_2,\ N_2 \text{ etc. in traces}$$

The first group of microorganisms hydrolyzes the complex organic substances to soluble end products and is called as hydrolytic bacteria. The second group of microorganisms called acidogenic bacteria converts the product of first group of bacteria into simple end product primarily VFA and alcohols. The third group called methanogenic bacteria converts the produced acid by the second group into methane and carbon dioxide. The reactor contentshould be free from oxygen. Alkalinity present in the reactor should be sufficient for proper functioning of the digester to maintain the pH between 6.5 to 8.0. Temperature has got tremendous effect in the functioning of a digester. It has been established that two types of bacteria, mesophilic (20 to 40 ^{o}C) and thermophilic (45 to 65 ^{o}C) are responsible for biodegradation. Therefore, the digester can be operated either at mesophilic or thermophilic temperature range. Places where the temperature is less than 20 ^{o}C, the digesters are required to be heated externally to bring the temperature to the mesophilic range.

22.3.2 Advantages and disadvantages of anaerobic digestion

Advantages

- Methane recovered can be used as alternate fuelsource.
- Reduce production of landfill greenhouse gases when otherwise these untreated sludge is disposed on landfill, which then broken down anaerobically to release methane into atmosphere.
- Reduction in volume of sludge and improving dewatering characteristics of the sludge makes it easy todry.
- Reduces odour/ fliesproblem.
- Low operating cost, since energy is not require to supply oxygen being anaerobic process.

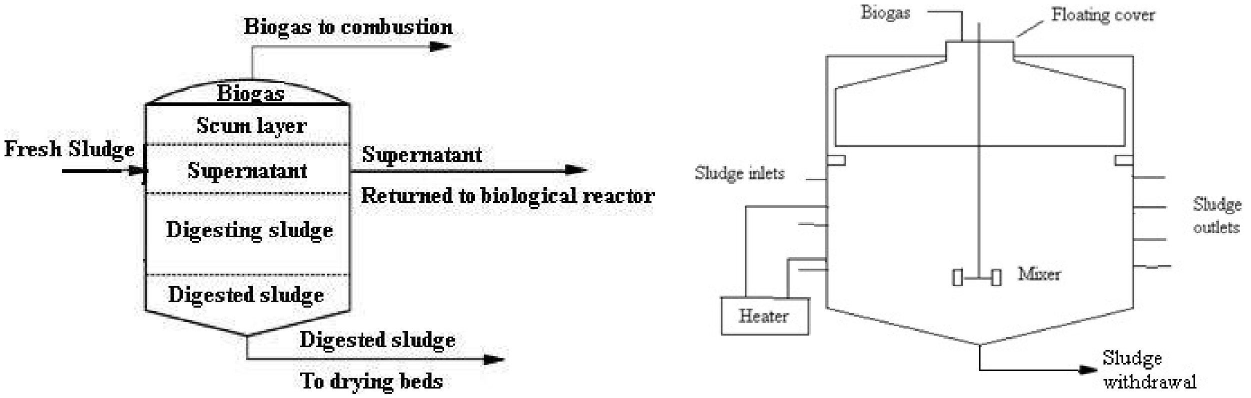
Disadvantages

- Accumulation of heavy metal and recalcitrant contaminants in thesludge.
- Narrow operating temperature controlrange.
- When heating is to be provided safely handling is required with electrical grid based heat management.

22.3.2 Types of anaerobicdigesters

Two types of digesters are used in practice, standard rate or low rate and high rate digester (Figure 22.4). In the standard rate digestion process, the contents of the digester are usually unheated and unmixed. In this acidification, methane fermentation and sludge thickening takes place in single tank. Standard or low rate digesters have intermittent mixing, intermittent sludge feeding and intermittent sludge withdrawal (Figure 22.4). Detention time for this process vary from 30 to 60 days. When sludge is not being mixed, the digester contents undergo thermal stratification. About 50% of the capacity of the digester remains unutilized due to the stratification and lack of mixing. Hence, longer detention time is required for complete stabilization of the sludge, which results in large capacity requirement for the digester. Suggested digestion period for different temperature for low rate digesters are givenbelow:

Temperature, ^{o}C	10	16	21	27	32	38
Digestion period, days	75	56	42	30	25	24



(a) Conventional Standardratedigester (b) High rate digester, continuous flow stirredtank, single stage process

Figure 22.4 Anaerobic digesters

In high rate digestion process, the contents of the digester are heated and completely mixed. Continuous or intermittent sludge feeding and withdrawal is practiced in these digesters. The required detention time is 15 days or less. Two stage digestion is also practiced where two tanks are provided. The first tank is provided for acidification and second for methane formation and thickening and supernatant formation. In single stage digester mixing is stopped and the contents are allowed to stratify before digested sludge and supernatant are withdrawn.

The digestion tanks are circular and diameter ranges between 6 and 45 m. The depth of water ranges between 7.5 and 14 m. The minimum slope of the tank floor is 1 vertical to 4 horizontal the solid loading used for design is 0.5 to 1.6 kg VSS/m^3.d. The design criteria for the digesters are given in the Table 22.2. Generally digesters are operated at mesophilic temperature (35 ^{0}C optimum) and in cold weather digester is heated to near optimum temperature. For thermophilic operation digesters are heated at 55 ^{0}C.

Table 22.2 Design criteria for anaerobic digesters

Parameter	Low rate digestion (Intermittent mixing)	High rate digestion (Heating and mixing)
Volatile solids loading, kg VSS/m^3.d	0.6 – 1.6	2.4 – 6.4
Solid retention time, days	30 – 40	10 – 20
Hydraulic retention time	30 – 40	10 – 20
% Solids in feed sludge (dry basis)	2 to 5	4 to 6
% Solids in underflow (digested sludge)	4 to 8	4 to 6
Gas production, m^3/kg of VSS added	0.65	0.5
Diameter, m	6 – 45	6 – 45
Height, m	7 – 14	7 – 14

22.3.2.1 Low ratedigesters

Conventional low rate digesters are single stage digesters and may have floating covers or fixed cover. Most municipal plants have floating type covers. They are having diameter ranging from 4.5 m to 38 m. Side water depth (SWD) is generally 6 to 9 m and free board of 0.6 to 0.75 m is provided. When fresh sludge is being added digested sludge is recycled along with that and added at middle of digester. When sludge is added once every day no withdrawal of supernatant and sludge is done. The digester sludge is withdrawn once every two weeks. During rainy days when sludge cannot be applied on sludge drying beds it is not withdrawn from digester till suitable weather conditionsfavors.

For fixed cover digester there is limit for accommodating fresh sludge and digested sludge and also for biogas. During fresh sludge addition biogas will get compressed and maximum allowable pressure inside is 0.203 m of water column. When sludge is withdrawn, biogas will expandandpressurewillbedecreasedwithminimumallowablepressureof76mmofwater column. Fixed cover digesters will have more problems of grease floating and drying. They are generally used for population less than 10000 persons. Recycled pump used to provide mixing should have capacity to pump one tank volume in 30 min. In this way mixing is performed for 1 h every day. Generally low rate digesters (conventional) without mixing will face problem of acidification.

22.3.2.2 High ratedigesters

High rate digesters usually have fixed cover. Mixing is continuous hence entire digester volume is active, which is only about 50% in case of conventional digesters. Due to mixing, better contact between fresh sludge and active microbes occurs and hence, higher loading and less retention time is provided in this. When fresh sludge is being added, the digested sludge may be displaced to holding tank where supernatant is separated. Alternatively mixing is stopped, stratification forms and sludge is withdrawn and fresh sludge added and mixing isrestored.

Mixing system is separate than sludge recycling. This is achieved by recycling gas or impeller mounted with a draft tube in the centre or impeller without draft tube. Gas mixing has advantage as there are no mechanical parts involved in mixing. Figure 22.5 shows different types of mixing systems in anaerobic digesters such as a) Gas recycle and draft tube, b) Gas recycle and gas injection, c) Impeller and draft tube, and d) Impeller (Reynolds, 1996).

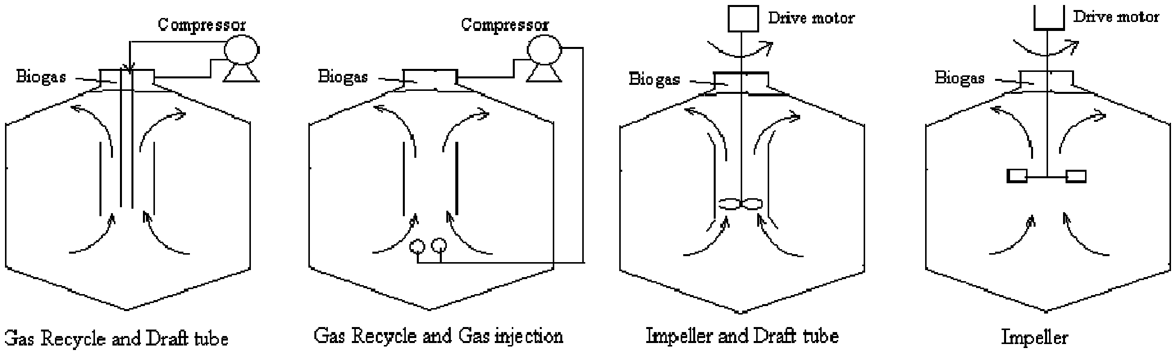


Figure 22.5 Types of mixing systems in anaerobic digesters

22.3.2.3 Two stage digester

Generally this type of digester is provided when population served ranged from 30,000 to 50,000. In first stage, mainly liquefaction of organic solids, digestion of soluble organic materials and gasification occurs. First stage is usually high rate digester with fixed cover and continuous mixing is preferred (Figure 22.6). In second stage, some gasification occurs however main use is supernatant separation, gas storage and digested sludge storage. This second stage is usually conventional digester type with floating cover and it is provided with intermittent mixing. Organic loading on first stage is much more than secondstage.

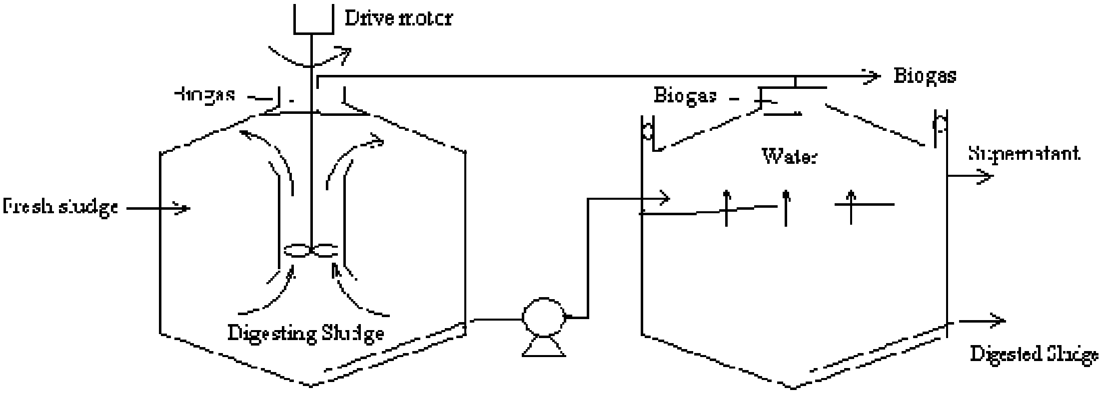


Figure 22.6 Two stage digester

22.3.3 Operation ofDigesters

Generally digesters are operated at temperature of 30 to 37 ^{0}C and optimum pH of 7 to 7.2. The wide operating pH range could be 6.7 to 7.4. Lime may be added to control the pH. Drop in pH inhibits methanogenesis and reduces methane production. The digester gas produced from the process may be used for heating purpose. Heavy metals may inhibit digestion process so that it must be eliminated at source. Supernatant liquor has BOD_5 of about 2000 mg/L and SS of 1000 mg/L and it is recycled back to PST. Usually acidification will not occur if the dry solids added or withdrawn daily do not exceed 3 to 5 % of the dry solids in the digester. The degree of the digestion is measured by VSS reduction and amount of gasproduced.

22.3.4 Digester Volume

For design consideration, total digester volume is a function of the fresh sludge added daily, volume of digested sludge produced daily and for required digestion time, volume of storage of digested sludge, gas storage, and volume of supernatant liquor. Volume of gas storage is relatively small as compared to others. If the supernatant liquid is removed as it is produced, the volume of remaining digesting sludge versus digestion time is parabolic function. The average volume of digested sludge is the initial volume minus 2/3 of the difference between initial and final volume (Fair et al. 1968).

Average volume of digested sludge $V_{avg} = V_1 - \frac{2}{3}(V_1 - V_2)$ …(1)

Where V_1 and V_2 = volume of fresh sludge added daily and volume of digested sludge produced daily, respectively (m^3/day).

The reduction in volume of sludge during digestion is mainly due to the release of water from the sludge solids and partly due to destruction and conversion of solids to biogas.

Therefore, total volume of sludge in digesters

$Vs = V_{avg} * t_d + V_2 * t_s$ … (2)

Where, Vs = Total sludge volume, m^3

V_2 = volume of digested sludge, m^3/d

V_{avg} = Average volume of digesting sludge, m^3/d

t_d = time for required digestion, days

t_s = time provided for sludge storage, days

The sludge volume normally occupies bottom half of the digester and supernatant and biogas occupies top half, hence

$V_t = 2V_s$ … (3)

Where, V_t = total digester volume, m^3

The digesters can also be designed based on typical mean cell residence time and organic loading rates (kg VSS/m^3.day).

The mean cell residence time, $\theta_c = \frac{X}{\Delta X}$ … (4)

Where, X = kg of dry solids in the digester and ΔX = kg of dry solids produced per day in the digested sludge, which is equal to the sludge wasted per day.

The SRT in digester is equal to HRT because of no recycling. There is minimum θ_c below which digestion will not occur which depends on temperature.

Table 22.3 Suggested values of θ_c at different temperature (McCarty, 1964).

Temperature, ^{0}C	$\theta_{c,}$ days
18	11
24	8
30	6
35	4
40	4

Actual value of θ_c used in design is 2.5 times higher since it is a critical parameter. Therefore, volume of High rate digesters

$V = Q^* \theta_c = Q^* \theta_H$

Where, Q = Fresh sludge volume, m^3/day

θ_c = designed mean cell residence time, days

θ_H = Hydraulic retention time, days

Example 1

A low rate digester is to be designed for waste sludge generated from activated sludge process treating sewage generated from 25000 persons. The fresh sludge has 0.11 kg dry solids/capita-day (VS = 70 % of ds). The dry solids (ds) is 5% of the sludge and specific gravity is 1.01. During digestion 65% of VS are destroyed and fixed solids remained unchanged. The digested sludge has 7% ds and a wet specific gravity is 1.03. Operating temperature of digester is 35^{0}C and sludge storage time is 45 days. Determine the digester volume required. Assume digestion time of 23days.

Solution

From given data,

Solids in incoming sludge = 25000*0.11 = 2750 kg/day.

Volatile solids = 2750 * 0.7 = 1925 kg/day

Fixed solids = 2750 * 0.3 = 825 kg/day

VSS destroyed = 1925 *0.65 = 1251 kg/day

The digested sludge solids represent output.

Output = Input – decrease due toreaction

VSS in digested sludge = 1925-1251 = 674 kg/day in digested sludge

Hence total solids in digested sludge = Volatile solids+ Fixed solids

= 674 + 825 = 1499 kg/day

Volume of digestedsludge= $\frac{1499}{0.07*1.03*1000}$ = 20.79 m^3/day

Fresh sludgevolume= $\frac{2750}{0.05*1.01*1000}$ = 54.46 m^3/day

Average volume of digesting sludge $V_{avg} = V_1 - (2/3)(V_1 - V_2)$

= 54.46 - 0.67 (54.46 - 20.79) = 31.9 m^3/day

Total volume of digested sludge in digestion = (31.9 * 23) + (20.79 * 45) = 1669.25 m^3

Hence, volume of digester = 2 * 1669.25 = 3338.5 m^3

17 : Sludge Management

22.4 Aerobic digestion

Aerobic digestion is affected by biosolids temperature, rate of biosolids oxidation, biosolids loading rate, system oxygen requirements, biosolids age, and biosolids characteristics. The process converts organic sludge solids to carbon dioxide, ammonia, and water by aerobic bacteria with reduction of volatile solids, pathogens, and offensive order. It can be used to treat only (i) waste activated sludge, (2) mixture of ASP sludge (or trickling filter sludge) and PST sludge, and (iii) and waste sludge of ASP designed without PST. It is similar to ASP with HRT of 10 to 12 days. The oxygen requirement in aerobic digestion for the complete oxidation of the BOD is about 2 kg/kg of cells. To ensure proper operation, the contents of the aerobic digestion should be well mixed. During this extended aeration, the microorganisms enter a phase (the endogenous stage) where materials previously stored by the cell oxidize, reducing the biologically degradable organic matter. During this endogenous stage, food supplies to microbial life are depleted to the point where the microorganisms begin to consume their own protoplasm, oxidizing it to carbon dioxide, water, and ammonia. As the digestion process continues, the ammonia is further converted to nitrates. Eventually, the oxygen uptake rate levels off and the biosolids matter is reduced to inorganic matter and relatively stable volatilesolids.

In a conventional aerobic digester, concentration of influent VSS must not be more than 3% for retention times of 15 to 20 days. In the batch basis, the digester is filled with raw sludge and aerated for 2 to 3 weeks, then stopped. The supernatant is decanted and the settled solids are removed. For the semi batch basis, raw sludge is added every couple of days; the supernatant is decanted periodically, and the settled solids are held in the digester for a long time before being removed.

22.5 Sludge Conditioning

Sludge is conditioned to improve its dewatering characteristics. Two methods are commonly used for sludge conditioning (i) addition of chemicals and (ii) heat treatment. Chemical conditioning results in coagulation of the solids and release of the absorbed water. Conditioning isusedinadvanceofvacuumfiltrationandcentrifugation.Chemicalsusedincludeferric chloride, lime, alum and organic polymers. The chemical dosage required is determined in the laboratory test. The sludge which is difficult to dewater requires higher dose.

Heat Treatment of sludge is both stabilization and a conditioning process. This involves heating of sludge for short period (30 min) under pressure (1.0 to 1.4 MN/m^2). The temperature is kept in the range of 140 to 200 ^oC. The treatment coagulates solids, breaks down the gel structure and reduces the water affinity of sludge solids. As a result the sludge is sterilized, deodorized, and is dewatered readily on vacuum filter or filter presses, without addition of chemicals. The heat treatment process is most applicable to biological sludges that may be difficult to stabilize or condition by other means. The high capital costs of equipment generally limit its use in the large plants.

22.6 SludgeDewatering

The digestion of the primary or mixed sludge will bring down the water content to about 90%; however, treatment is necessary to reduce the water content further. When digested sludge is applied on the sludge drying beds, the water content of the sludge can be reduced to around 70%. Presence of excess oil and grease will interfere with this process. Sludge drying beds require large land area (nearly 40% of the total area required for sewage treatment plant), hence at the places where land is not available other alternatives such as, mechanical dewatering on vacuum filters, filter press or centrifuge followed by heat drying or incineration could be used after sludge conditioning. In India, most of the parts of the country there is favourable climate for open sludge drying, hence sludge drying beds are preferred as an economical way and easy to manage.

22.6.1 Sludge DryingBeds

This is used where land available is adequate and the dried sludge is used for soil conditioning. The sludge is applied on the bed of sand, which is supported on gravel. Major portion of the liquid drains off in the first few hours after which drying occur due to evaporation. Sludge cake shrinks, producing cracks which further accelerate evaporation from the sludge surface. In dry region generally the sludge will get dried within two weeks. The drying period will depend on sunshine, rainfall, wind velocity, and relative humidity, apart from sludge characteristics. Under adverse weather condition, it may take up to four weeks. The sludge drying beds should be located at least 100 m away from houses to avoid smelling problem.

22.6.2 Design Criteria for Sludge DryingBeds

Area of beds: It depends on the volume of the sludge to be dried, cycle time required to retain sludge for dewatering, drying and removal of dried sludge, and making the sand bed ready for next application. Cycle time for dewatering and drying depends on the applied depth of the sludge. The cycle time depends on the climatic conditions, temperature, humidity, dry or wet weather, wind velocity, etc. and also depends on the sludge characteristics. The land requirement can be substantial with the value of 0.1 to 0.25 m^2/capita for anaerobically digested sludge under unfavourable conditions (CPHEEO, 1993). Generally the cycle time is up to 2 weeks for warmer climate and 3 to 6 weeks in unfavorable conditions.

The specifications for preparation of sludge drying beds are as follows:

Underdrains: It is made from the open joined vitrified clay pipe or tiles of at least 10 cm diameter. Pipe should not be laid more than 6 m apart from each other. Arrangement should be made to return the drained water to primary sedimentation tank.

Gravel: Gravel covers the under-drainage system. Graded gravel is placed around the under drains in layers up to 30 cm, with minimum of 15 cm above under drains (Figure 22.7). At least top 3 cm layer of gravel is of 3 to 6 mm size.

Sand: Sand of effective size 0.5 to 0.75 mm and uniformity coefficient not greater than 4 is used. The depth of the sand may vary from 20 to 30 cm.

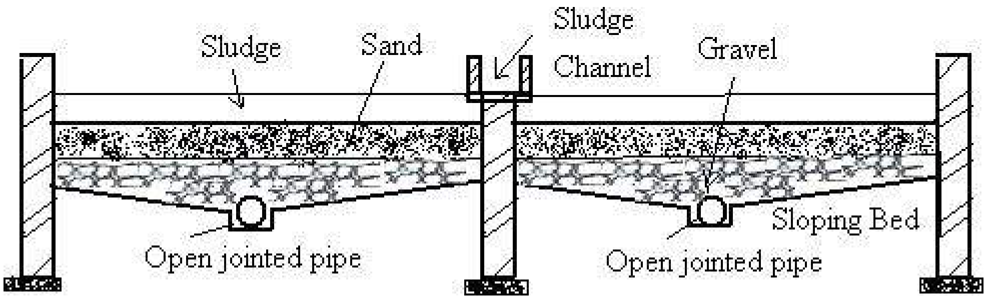


Figure 22.7 Sectional elevation of sludge drying beds

Dimensions: Sludge drying beds are commonly 6 to 8 m wide and 30 m long. With the bed slope of 0.5% the length should not exceed 30 m for single sludge application point. For longer length of sludge bed multiple sludge entry should be used to reduce the length of wet sludge travel. Free board of at least 0.4 m isused.

Sludge Inlet: Pipe of minimum 20 cm diameter should be used for sludge inlet pipe. This pipe should discharge sludge at minimum height of 0.3 m above the sand bed. Splash plates should be provided at discharge points to spread the sludge uniformly over the bed and prevent erosion of the sand bed.

Removal of Sludge: Dried sludge cake is removed by shovel or forks when the moisture content is less than 70%. When the moisture content is less than 40% the sludge is suitable for grinding. Some sand always clings to the bottom of the sludge cake. When the depth of the bed will get reduced to 10 cm due this loss, fresh sand of desired specification is added to match the original depth. After removal of the dried sludge the sand bed is prepared by leveling for next cycle of sludge application.

Example: 2

Design a sludge drying bed for drying the digested sludge volume of 229 m^3/day. Consider the dewatering, drying and sludge removal cycle of 10 days.

Solution

Provide the sand depth of 0.3 m and depth of sludge application as 0.3 m.

Total area required for the bed = 229 x 10 / 0.3 = 7633 m^2

Provide each bed of width 8.0 m and length of 30 m,

Therefore number of beds required = 7633/(8*30) = 31.8

Hence provide 32 beds of size 30 m x 8 m each.

22.7 Incineration

Biosolids incineration is the complete destruction of biosolids by heat. It is not actually a means of disposal, but used as a volume reduction to ash. Incinerating biosolids is not only a beneficial process; it produces a beneficial reuse product: biosolids ash. Compared to several other biosolids management options (especially land filling or lagoons), biosolids incineration presents some obviousadvantages:

- Considerably reduces area required for the sludge management as compared to lagoons or landburial
- reduces biosolids volume andweight
- provides immediate reduction of sludgevolume
- avoids transportation costs when it is on-siteincineration

A key factor in incineration is the solids content of the sludge. Before incineration, the sludge must be dewatered (often by centrifugation), because even though the heat value of sludge is relatively high, excessive water content consumes auxiliary fuel to maintain incinerator combustion.

22.8 Final Disposal of Sludge

Final disposal of the sludge from the treatment plant generally involves some form of land disposal. The most common methods of land disposal include spreading on land, lagooning, dumping, and landfilling.

Spreading on land: Dewatered and composted sludge can be disposed off by spreading over farm lands, and plowing under after it has dried. Wet dewatered sludge can be incorporated into soil directly by injection. The humus in the sludge conditions the soil and improves its moisture retentiveness.

Lagooning: It is an economical mode of disposal in remote area. A lagoon is an earthen basin into which untreated or digested sludge is deposited. Stabilization of untreated sludge can be carried out in a lagoon which gives objectionable odours. The stabilized sludge settles to the bottom of the lagoon and accumulates. Excess liquid from the lagoon, if there is any, is returned to the wastewater treatment plant at PST. Sludge may be stored indefinitely in a lagoon, or it may be removed periodically after draining anddrying.

Dumping: Dumping in an abandoned mine quarry is a suitable disposal method only for the sludges and solids that have been stabilized, so that no decomposition or nuisance condition will results. Digested sludge, clean grit and incinerator residue can be disposed off safely by this method.

Landfilling: A sanitary landfill can be used for disposal of sludge, grease, grit and other solids, whether it is stabilized or not. The sanitary landfill method is most suitable if it is also used for disposal of the other solid wastes of the community. In a sanitary landfill, the wastes are deposited in a designated area, compacted in place with a tractor or roller and covered with a 30 cm layer of clean soil. With daily coverage of the newly deposited wastes, nuisance condition such as odour and flies are minimized.

Questions

1. What are the means by which the nuisance conditions associated with untreated sludge can beeliminated?
2. Describe advantages of sludge thickening and explain gravitythickening.
3. Describe sludge thickening by air floatationunit.
4. Why it is necessary to digest the sludge generated from sewage treatment plant before disposal?
5. Explain anaerobic and aerobic digestion of thesludge.
6. Stating necessity, describe sludgeconditioning.
7. With the help of sketch, explain two stagedigester.
8. What are different types of anaerobic digesters? Describe it withsketch.
9. Design a low rate digester for waste sewage sludge generated from aerobic secondary treatment of sewage generated from 40000 persons. The fresh sludge has 0.13 kg dry solids/capita.day (VS = 70 % of ds). The dry solids (ds) is 5% of the sludge and specific gravity is 1.01. During digestion 2/3 of VS are destroyed and fixed solids are remaining unchanged. The digested sludge has 6% ds and a wet specific gravity is 1.03. Operating temperature of digester is 32^0C for which the digestion time of 25 days is required. Consider sludge storage time of 50 days. Determine the digester volumerequired.
10. Design a sludge drying bed for drying 400 m^3/d of sewage sludge. Provide detail specifications for thebed.
11. Describe various options available for final disposal of the dewateredsludge.
12. Design high rate digester for ASP for a city of total population of 30000 generating sewage. The feed to the digester (primary and secondary sludge) is 56.1 m^3/day for the operating temperature of 34^oC. Determine the volume of thedigester.

Answers

Q. 9: Volume of digester = 7663.78 m^3

Q. 10: Provide 56 beds of size 30 m x 8 m each.

Q. 12: Total volume = 561 m^3

18 : Tertiary Wastewater Treatment

23.1 Introduction

Secondary treatment removes 85 to 95 percent of BOD and TSS and minor portions of nitrogen, phosphorus, and heavy metals. Tertiary treatment is the next wastewater treatment process after secondary treatment. This treatment is sometimes called as the final or advanced treatment and consists of removing the organic load left after secondary treatment for removal of nutrients from sewage and particularly to kill the pathogenic bacteria. The effluents from secondary sewage treatment plants contain both nitrogen (N) and phosphorus (P). N and P are ingredients in all fertilizers. When excess amounts of N and P are discharged, plant growth in the receiving waters may be accelerated which results in eutrophication in the water body receiving such waste. Algae growth may be stimulated causing blooms which are toxic to fish life as well as aesthetically unpleasing. Secondary treated effluent also contains suspended, dissolved, and colloidal constituents which may be required to be removed for stipulated reuse or disposal of the treated effluent.

The purpose of tertiary treatment is to provide a final treatment stage to raise the effluent quality before it is discharged to the receiving environment such as sea, river, lake, ground, etc., or to raise the treated water quality to such a level to make it suitable for intended reuse. This step removes different types of pollutants such as organic matter, SS, nutrients, pathogens, and heavy metals that secondary treatment is not able to remove. Wastewater effluent becomes even cleaner in this treatment process through the use of stronger and more advanced treatment systems. It includes sedimentation, coagulations, membrane processes, filtration, ion exchange, activated carbon adsorption, electrodialysis, nitrification and denitrification, etc. (Figure 23.1). Tertiary treatment is costly as compared to primary and secondary treatment methods.

23.2 Need of Tertiary treatment

Tertiary treatment may be provided to the secondary effluent for one or more of the following contaminant further.

- To remove total suspended solids and organic matter those are present in effluents after secondarytreatment.
- To remove specific organic and inorganic constituents from industrial effluent to make it suitable forreuse.

- To make treated wastewater suitable for land application purpose or directly discharge it into the water bodies like rivers, lakes,etc.
- To remove residual nutrients beyond what can be accomplished by earlier treatment methods.
- To remove pathogens from the secondary treatedeffluents.
- To reduce total dissolved solids (TDS) from the secondary treated effluent to meet reuse qualitystandards.

One or more of the unit operation/ process mentioned in Figure 23.1 will be used for achieving this tertiary treatment.

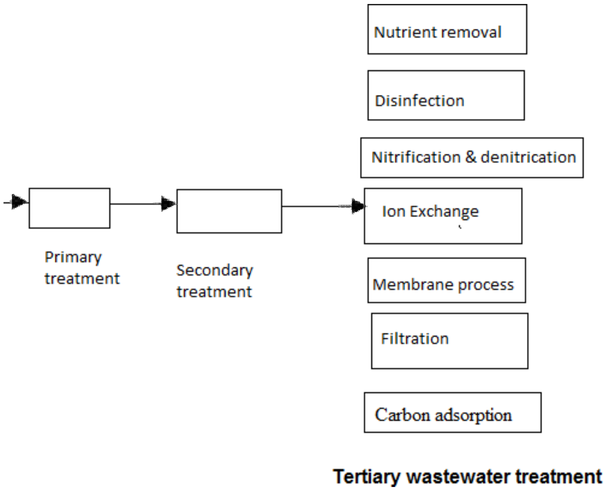


Figure 23.1 Process involved in tertiary wastewater treatment

23.3 Tertiary Treatments

In advanced wastewater treatment, treatment options or methods are dependent upon the characteristics of effluent to be obtained after secondary treatment to satisfy further use or disposal of treated wastewater.

23.3.1 Nitrogen removal

Wastewater containing nutrients includes sewage, agriculture runoff and many of the industrial effluents. The nutrients of most concerned are N and P. The principal nitrogen compounds in domestic sewage are proteins, amines, amino acids, and urea. Ammonia nitrogen in sewage results from the bacterial decomposition of these organic constituents.

The nitrogen compounds results from the biological decomposition of proteins and from urea discharged in body waste. This nitrogen is in complex organic molecules and is referred simply as organic nitrogen. Organic nitrogen may be biologically converted to free ammonia (NH_3^0) or to the ammonium ion (NH_4^+) by one of several different metabolic pathways. These two exists in equilibriumas

$$NH_4^+ \rightleftharpoons NH_3 + H^+$$

Ammonia nitrogen is the most reduced nitrogen compound found in wastewater, which can be biologically oxidized to nitrate if molecular oxygen is present (under aerobic condition). In wastewater, the predominant forms of nitrogen are organic nitrogen and ammonia. The nitrification may takes place in biological treatment units provided the treatment periods are long enough. Generally, for the HRT used in secondary treatment conversion of organic nitrogen to ammonia is significant and nitrification may not be significant. Because of oxygen demand exerted by ammonia (about 4.6 mg of O_2 per mg of NH^+-N oxidized) and due to other environmental factors, removal of ammonia may be required. The most common processes for removal of ammonia from wastewater are

i) Airstripping,
ii) Biological nitrification anddenitrification.

23.3.1.1 Airstripping

It consists of converting ammonium to gaseous phase and then dispersing the liquid in air, thus allowing transfer of the ammonia from wastewater to the air. The gaseous phase NH_3^0 and aqueous phase NH_4^+ exist together in equilibrium. The relative abundance of these phases depends upon both the pH and the temperature of the wastewater. The pH must be greater than 11 for complete conversion to NH_3. Since, this pH is greater than pH of normal wastewater, adjustmentofpHisnecessarypriortoairstripping.Additionoflimeisthemostcommonmeans for raising the pH. Enough lime must be added to precipitate the alkalinity and to add the excess OH^- ions for pH adjustment.

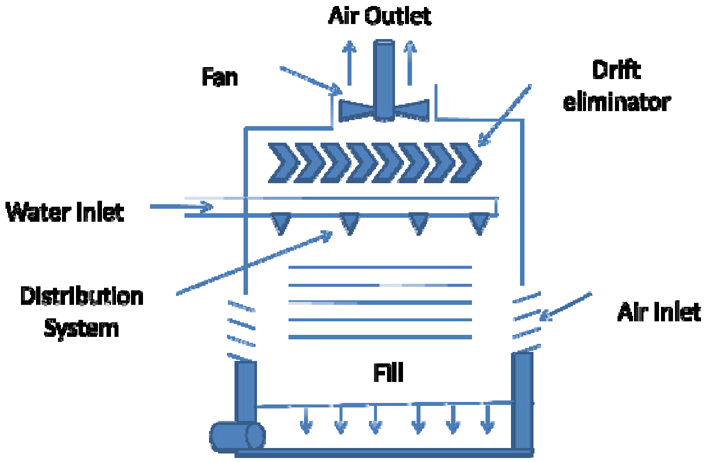


Figure 23.2 Spray tower

The most important and efficient reactor for air stripping is counter current spray tower. Larger quantity of air is required, and fan must be installed to draw air through tower (Figure 23.2). Packing is provided to minimize the film resistance to gas transfer by continuously forming, splashing and reforming drops.

The air to wastewater ratio ranging from 2000 to 6000 m^3 of air/m^3 of wastewater is used for design. Air requirement is more at lower temperature. Tower depths are generally less than 7.5 m, and hydraulic loading vary from 40 to 46 L/min.m^2 of tower. The limitation to this process occurs at temperature close to freezing temperature. Drastic reduction in efficiency is observed and preheating of gas is not possible because of highvolume.

The other problems associated with this include noise and air pollution and scaling of the packing media. Noise pollution problem is caused by roar of the fan. The odor problem is due to dispersion of ammonia gas in atmosphere, due to which this technique may not be permitted at some location. This problem can be minimized by locating the facility away from the populated area. Precipitation of calcium carbonate scale on the packing media as a result of wastewater softening can be minimized by the use of smooth surface PVC pipe as packing material. The occasional cleaning of packing media is stillrequired.

23.3.1.2 Biological Nitrification and denitrification

Bacteria remove ammonia nitrogen from wastewater by a two step biological processes: nitrification followed by denitrification to covert it finally to gaseous nitrogen. In this gaseous form N_2 is inert and does not react with the wastewater itself or with other constituents present in wastewater. Since, treated wastewater is likely to be saturated with molecular nitrogen; the produced N_2 is simply released to the atmosphere. These two steps involved require different environmental conditions and hence generally they are carried out in separate reactors.

Nitrification

It has important role in nitrogen removal from wastewater during treatment. The biological conversion of ammonium to nitrate nitrogen is called Nitrification. It is autotrophic process i.e. energy for bacterial growth is derived by oxidation of nitrogen compounds such as ammonia. In this process, the cell yield per unit substrate removal is smaller than heterotrophs. Nitrification is a two-step process. In first step, bacteria known as *Nitrosomonas*can convert ammonia and ammonium to nitrite. These bacteria known as nitrifiers are strictly aerobes. This process is limited by the relatively slow growth rate of *Nitrosomonas*. Next, bacteria called *Nitrobacter*finish the conversion of nitrite tonitrate.

First step: $NH_4^+ + 3/2O_2 \xrightarrow{\textit{Nitrosomonas}} N0_2^- + 2H^+ + H_2O$
(Nitrite)

Second step: $N0_2^- + 1/2O_2 \xrightarrow{\textit{Nitrobacter}} N0_3^-$
(Nitrate)

Overall reaction: $NH_4^+ + 2O_2 \longrightarrow N0_3^- + 2H^+ + H_2O$

*Nitrosomonas*and *Nitrobacter*use the energy derived from the reactions for cell growth and maintenance. Some of ammonium ions are assimilated into cell tissues. Neglecting this ammonium ion used in cell synthesis the O_2 required to oxidize ammonia to nitrate is 4.57 mg O_2/mg ammonium nitrogen. If the ammonium used in cell, O_2 required is considered it is 4.3 mg O_2/mg ammonium nitrogen and about 7.14 mg of alkalinity is needed to neutralize the H^+produced.

Nitrification may be used to prevent oxygen depletion from nitrogenous demand in the receiving water. Nitrification requires a long retention time, a low food to microorganism ratio (F/M), a high mean cell residence time (MCRT), and adequate alkalinity. Wastewater temperature and pH affects the rate of nitrification.

Under favourable conditions, carbon oxidation and nitrification may occur in a single reactor called single stage under favourable conditions. In separate stage carbon oxidation and nitrification occur in different reactors. It can be accomplished in both suspended and attached growth process such as trickling filter, ASP, rotating disc biological contactor (RBC), SBR, etc. Also, nitrifying organisms are present in almost all aerobic process sludge but they are less in number. In nitrification, when ratio of BOD_5 to TKN is greater than 5, the process is called as combined carbon oxidation and nitrification, whereas, when ratio of BOD_5 to TKN is less than 3, the process is called as separate stage carbon oxidation and nitrification.

For single stage carbon oxidation and nitrification, plug flow, completely mixed and extended aeration ASP can be used. Oxidation ditch is also one of option for this process. The attached growth processes like TF and RBC are commonly used. Nitrification can be achieved by reducing applied loading rate, increasing the mean cell residence time (θ_c) and airsupply.

Nitrifying bacteria are sensitive organisms. A variety of organic and inorganic agents can inhibit the growth and action of these organisms. High concentration of ammonia and nitrous acid can be inhibitory. The effect of pH is also significant with optimal range of 7.5 to 8.6. The system acclimatize to lower pH can also work successfully. The temperature also has considerable impact on growth of the nitrifying bacteria. Dissolved oxygen concentration above 1 mg/L is essential for nitrification. Below this DO, oxygen becomes the limiting nutrients and nitrification slows down orceases.

Denitrification

In some applications, such as discharge of effluent into enclosed water bodies or recycle to water supplies, nitrification may not be sufficient. When nitrogen removal is required, one of the available methods is to follow biological nitrification with denitrification. Denitrification is accomplished under anaerobic or near anaerobic conditions by facultative heterotrophic bacteria commonly found in wastewater. Nitrates are removed by two mechanisms: (1) conversion of NO_3 to N_2 gas by bacterial metabolism and (2) conversion of NO_3 to nitrogen contained in cell mass which may be removed by settling. Denitrification occurs when oxygen levels are depleted and nitrate becomes the primary electron acceptor source formicroorganisms.

Nitrate, $N0_3^-$ → Nitrite, $N0_2^-$ Nitric → oxide, NO Nitrous oxide, → N_2O Nitrogen, N_2 →

Denitrifying bacteria are facultative organisms, they can use either dissolved oxygen or nitrate as an oxygen source for metabolism and oxidation of organic matter. This is carried out by hetetrophic bacteria such as *pseudomonas, spirillum, lactobacillus, bacillus, microaoccus,* etc. For reduction to occur, the DO level must be near to zero, and carbon supply must be available to the bacteria. Because of low carbon content is required for the previous nitrification step, carbon must be added before denitrification can proceed. A small amount of primary effluent, bypassed around secondary and nitrification reactor can be used to supply the carbon. However, the unnitrified compounds in this water will be unaffected by the denitrification process and will appear in effluent. When complete nitrogen removal is required, an external source of carbon containing no nitrogen will be required. The most commonly used external source of nitrogen is methanol. When methanol is added the reaction is

$N0_3^- + 5/6CH_3OH \longrightarrow \frac{1}{2} N_2 + 5/6\ CO_2 + 7/6\ H_20 + OH^-$

For treatment plant above 3 mg/L of methanol is required for each milligram per litre of nitrate, making this process an expensive. Alkalinity is generated in this process. Denitrification can be carried out as attached growth (anaerobic filter) and suspended growth process (expanded bed or UASB reactor).

23.3.2 Phosphorusremoval

Normally secondary treatment can only remove 1-2 mg/l of phosphorus, so a large excess of phosphorus is discharged in the final effluent, causing eutrophication of lakes and natural water bodies. Generally it appears as orthophosphate, polyphosphate and organically bound phosphorus. Phosphorus is required for microbes for cell synthesis and energy transport, maintenance as well as it is stored for subsequent use by microbes. During secondary treatment process about 10 to 30 % of influent phosphorus is removed (Metcalf & Eddy, 2003). Phosphate removal is currently achieved largely by chemical precipitation, which is expensive and causes an increase of sludge volume by up to 40%. An alternative is the biological phosphate removal (BPR), which is accomplished by sequencing and producing the appropriate environmental condition in thereactors.

Acinetobacter organisms are helpful for removal of phosphorus from effluent. Under anaerobic conditions, phosphorus accumulating organisms (PAO) assimilate fermentation products (i.e. volatile fatty acids) into storage products within the cells with the concomitant release of phosphorus from stored polyphosphates (Gray, 2005). Release of phosphorus occurs under anoxic condition. The BPR requires both aerobic and anaerobic zones in reactors for efficient treatment. Generally, lime precipitation is most commonly used for phosphorus removal. Phosphorus is removed in the waste sludge from thesystem.

Treatment technologies presently available for phosphorus removal include (de-Bashan and Bashan, 2004):

A) Physical:
  a) filtration for particulatephosphorus
  b) membranetechnologies
B) Chemical:
  a) precipitation
  b) other (mainly physical-chemicaladsorption)
C) Biological
a) assimilation
b) enhanced biological phosphorus removal(EBPR)

23.3.3. Ion exchange

Ion Exchange can be used in wastewater treatment plants to swap one ion for another for the purpose of demineralization. The widest application of this process is in domestic water softening, where sodium ions are removed on cation exchange resin and chlorides are removed on anion exchange resin. Ion exchange is a unit process in which ions are removed out of an aqueous solution and is replaced by another ionic species. The basic principle behind ion exchange is that a weakly bound ion can preferably be displaced by a stronger binding ion. This effect is called the principle of selectivity. A more selective ion binds more strongly than a less selectiveion.Theeffectofselectivitycanbeusedtoremovedistinctionsfromwaterandto replace them with others. It can be operated in a batch or continuous mode and has been used for removal of nitrogen, heavy metals and TDS in wastewater applications. It has also been used selectively to remove specific impurities and to recover valuable trace metals like chromium, nickel, copper, lead and cadmium from industrial waste discharges.

A number of naturally occurring minerals have ion exchange properties. Among them the notable ones are aluminium silicate minerals, which are called zeolites. Synthetic zeolites have been prepared using solutions of sodium silicate and sodium aluminate. Alternatively synthetic ion-exchange resins composed of organic polymer with attached functional groups such as $–SO_3^- H^+$ (strongly acidic cation exchange resins), or $- COO^- - H^+$ (weakly acidic cation exchangeresins or $-N^+(CH_3)_3OH^-$ (strongly basic anion exchange resins) can be used. Synthetic and industrially produced ion exchange resins consist of small, porous beads that are insoluble in water and organic solvents. The most widely used base-materials are polystyrene andpolyacrylate.

23.3.4 Membrane process

Membrane technology can be used to treat a variety of wastes, including sanitary landfill leachate containing both organic and inorganic chemical species, water-soluble oil wastes used in metal fabricating and manufacturing industries, solvent-water mixtures, and oil-water mixtures generated during washing operations at metal fabricating facilities. Depending upon the material used for membrane, nature of driving force and separation mechanism, the membrane processes can be classified into sub-processes such as electrodialysis (ED) or electrodialysis reversal (EDR), microfiltration (MF), ultrafiltration (UF), nanofiltration (NF), reverse osmosis(RO).

MF, UF, NF and RO use pressure to transport water across the membrane. MF removes particulate matter while RO removes many solutes as water permeate through the membrane. MF membranes have the largest pore size and typically reject large particles and various microorganisms. UF membranes have smaller pores than MF membranes and, therefore, in addition to large particles and microorganisms, they can reject bacteria and soluble macromolecules such as proteins. RO membranes are effectively non-porous and, therefore, exclude particles and even many low molar mass species such as salt ions, organics, etc. NF membranes are relatively new and are sometimes called loose RO membranes. They are porous membranes, but since the pores are of the order of ten angstroms or less, they exhibit performance between that of RO and UF membranes. Different membrane processes are being used in water treatment since 1960.

The driving forces for the use of membrane technology are increased regulatory pressure to provide better quality water, increased demand of water requiring exploitation of low quality water resources as source and development and commercialization of membrane processes. Application of membrane processes for wastewater treatment is increasing worldwide day by day with reduction in cost of process and increased water charges. Membrane processes are used for about 70 % of the total installed capacity of desalination worldwide and percentage is increasing (for drinking water). In Japan, more than 30 MLD wastewater is used as toilet flushing water in building. It has application in many industrial wastewater treatment plants to produce reusable quality water from the effluents.

23.3.4.1 Types of membrane operation

The membrane is defined as the thin film separating two phases and acting as a selective barrier to the transport matter. Chemical potential difference exists between the two phases (Figure 23.3). ***Retentate*** contains non permeating species (Figure 23.4). ***Permeate*** forms the produced water from the membrane filtration. Membrane operation is recommended nomenclature than membrane process. The relevant main membrane operations used in water treatment are summarized in Table23.1.

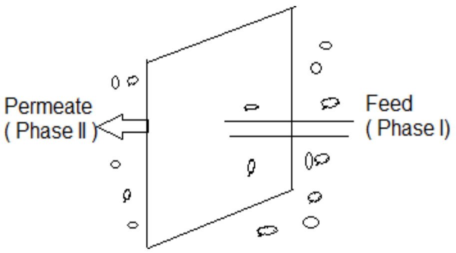


Figure 23.3 Mechanism of membrane operation

Table 23.1 Technically relevant main membrane operations in water treatment

Membrane	**Driving force**	**Mechanism of separation**	**Membrane structure**	**Phases in contact**
Microfiltration (MF)	pressure	Sieve	Macropores	Liquid-liquid
Ultrafiltration (UF)	pressure	Sieve	Mesopores	Liquid-liquid
Nanofiltration (NF)	pressure	Sieve + solution + diffusion + exclusion	Micropores	Liquid-liquid
Reverse Osmosis	pressure	Solution/diffusion + exclusion	Dense	Liquid-liquid

***Advantages of membrane operation**:*

- Separation takes place at ambient temperature without phasechange.
- Separation takes place without accumulation of product inside membrane (unlike ion exchange resins, which needsreplacement/regulations).
- No need of chemical additives forseparations.

Figure 23.4 Three end modules

23.3.4 .2 Membrane operations

The transmembrane flux for each element is given by

Flux = Force * concentration * mobility

In most cases, concentration will vary with distance from the membrane surface along the boundary layer. Hence above equation is local equation where local forces are used to estimate gradient of chemical potential dμ/dx of every component that can be transported.

The variation of chemical potential of component ‘i’ can be expressed as a sum of the terms

$$d\mu_i = RTd\,ln\,a_i + V_i\,dP + Z_iFd\Psi$$

where, activity a_i (product of concentration by activity coefficient) is not under the control of operator. The pressure P and electric potential Ψ can be varied in order to improve the separation between the mobile components. The applied pressure of every component is proportional to its molar volume V_i. Electric field acts on every ionic species according to its valencyZ_i and does not affect non-ionic species.

23.3.4.3 Reverse Osmosis

Reverse osmosis (RO) is a most commonly used membrane filtration method that removes many types of large molecules and ions from effluents by applying pressure to the effluents when it is on one side of a selective membrane. RO is used to remove specific dissolved organic constituents from wastewater remaining after advanced treatment with depth filtration or MF. RO system can operate at 90 % efficiency or better with respect to TDS. In addition, it also removes residual organic molecules, turbidity, bacteria andviruses.

The solvent of the solution is transferred through a dense membrane tailored to retain salts and low molecules weight solutes. When concentrated saline solution is separated from pure water by such a membrane, difference in chemical potential tends to promote the diffusion of water from the diluted compartment to the concentrated compartment in order to equalize concentration. At equilibrium, the difference in the levels between two compartments corresponds to the osmotic pressure of the saline solution. The demineralization of water can be accomplished using thin, microporous membranes. There are two basic modes of operation in use. One system uses pressure to drive water through the membrane against the force of osmotic pressure and is called RO. The pressure applied is several orders of magnitude in excess of natural osmotic pressure. The membrane commonly used in reverse osmosis is composed of cellulose acetate and is about 100 μm thick. The film contains microscopic openings that allow water molecules to pass through but retain dissolved solids by either molecular sieving (or by some other mechanism which is not yet completely understood). The process results in a concentrated solution of the ions on the pressure side of the membrane and product water relatively free of ions on the other side of themembrane.

For the producing pure water from the saline solution, osmotic pressure of the solution must be exceeded in the brine. For economically viable flows, at least twice the osmotic pressure must be exerted e.g. for seawater pressure of 5 -8 MPa is used in practice.

22.3.4.4 Nanofiltration (NF)

Nanofiltration (NF) is a rapidly advancing membrane separation technique for water and wastewater treatment as well as concentration/separation of antibiotics and pharmaceuticals due to its unique charge-based repulsion property and high rate of permeation. This process is also called as low pressure RO or membrane softening. It lies between RO and ultrafiltration in terms of selectivity of membranes, which is designed for removal of multivalent ions (Ca, Mg, etc.) in softening operation. Monovalent ions are poorly rejected. Hence, osmotic back pressure is much lower than that in RO. The operating pressure used in NF is typically 0.5 to 1.5 MPa.

It is a pressure driven process wherein the pore size of the membrane is in the range of 0.5-1 nm. Due to the lower operating pressure and higher flow rates, nanofiltration is inexpensive when compared to reverse osmosis. NF membranes allow partial permeation of monovalent salts such as sodium chloride while rejecting bivalent salts and hardness to a greater extent from aqueous solutions. NF can lower TDS and hardness, reduce color and odor, and remove heavy metal ions from groundwater. Other possible applications include treatment of effluents from textile dyeing, bulk drug and chemical process industries.

NF process is useful for raw water containing TDS in the range 700-1200 ppm and has exhibited two advantages over RO. NF allows sufficient permeation of essential salts while keeping the total dissolved solids (TDS) in accordance to BIS drinking water standards, whereas RO removes almost all the minerals essential for the human body. NF can be operated at lower pressure while RO requires higher operating pressure and therefore higher running cost.

23.3.4.5 Ultrafiltration(UF)

It is a clarification and disinfection membrane operation. UF membranes are porous and allow only coarser solutes (macromolecules) to be rejected. All types of microorganisms as viruses and bacteria and all types of particles can be removed by this process. Since, the low molecular solutes are not retained by UF, osmotic back pressure can be neglected and operating pressure is kept low as 50- 500 KPa.

23.3.4.6 Microfiltration(MF)

Microfiltration (MF) membranes are having 0.1 µm or more pore size. It is generally used for particulate matter removal. The pressure used in this process is similar to that of UF.

23.3.5 Classification ofmembranes

23.3.5.1 ***According to separation mechanism:*** Three mechanisms exist for separation of ions and other particulate matter from the membranes, and accordingly the membranes are classified as below:

- Sieve effect: In this mechanism, separation is based on difference in pore size e.g. MF,UF.
- Solution-diffusion mechanism: In this mechanism, separation is based on difference in the solubility and diffusivity of materials in the membrane e.g.RO.
- Electrochemical effect: In this mechanism, separation is based on difference in the charges of the species to be separated e.g. ED(electrodialysis).

The classification based on separation mechanism leads to three main classes of membranes.

i) Porousmembranes

Fixed pores are present in these membranes. These pores are sub-divided into three types' viz.

- Macropores: Theses are larger than 50nm
- Mesopores- These are in the range of 20 to 50nm
- Micropores- These are in the size less than 2nm.

MF and UF are porous membranes while, NF could be classified in an intermediateclass between porous and non porous membranes. Since, solution diffusion and even electrochemical effect have to introduce in equilibrium of masstransfer.

ii) Non Porousmembranes

These are dense media membranes. The diffusion of species takes place in the free volume which is present between the macromolecular chains of the membrane material. RO is non porous membrane.

iii) Ion exchangemembranes

These are specific types of non porous membranes. They consist of highly swollen gels carrying fixed positive or negative charges. A membrane with fixed positive charges (e.g. $-NR_3^+$) is called as anion exchange membranes, whereas cation exchange membranes have fixed negative charges (e.g. $–SO^{3-}$).

23.3.5.2 Classification based on morphology

i) Asymmetricmembranes

These are anisotropic membranes prepared from the same material (Figure 23.5 a).

ii) Compositemembranes

These are anisotropic membranes where top layer and sub-layer originate from different materials. Each layer can be optimized independently. Generally porous layer is asymmetric membranes.

The anisotropic membrane consists of very thin top layer called skin supported by a thicker and more porous supporting sub-layer (Figure 23.5 b). Skin has main function of membrane. Overall flux and selectivity depends upon the structure of skin. Skin has thickness in the range of 0.1 to 0.5 µm, which is about 1% of the thickness of porous sub-layer. Supporting layer offers negligible resistance to mass transfer and imparts mechanical strength to the membrane forming integral part of the membrane.

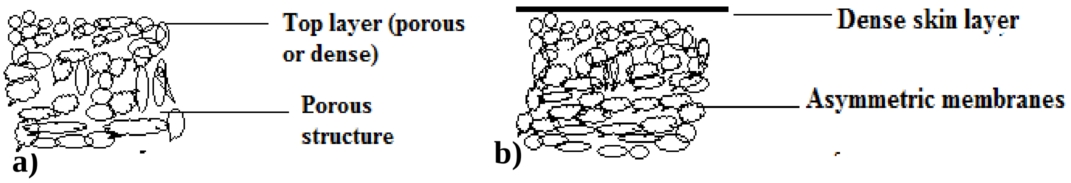


Figure 23.5 a) Asymmetric membrane and b) Composite membrane.

2.3.3.5.3 Classification based ongeometry

According to the geometry, the membranes can be classified as flat sheet membrane and cylindrical membrane. The cylindrical membrane can be further classified as (a) Tubular membranes having internal diameter >3 mm, and (b) Hollow fiber membranes having internal diameter <3 mm. They are available in market with outer diameter ranging from 80 to 500 µm. They are used in RO with larger diameter. Also hallow fiber membranes are used in MF and UF in which they are also called as ‘capillary membranes’.

2.3.3.5.4 Classification based on chemical nature

On the basis of chemical nature of the material used for membrane these are classified as: organic (polymers) membrane and inorganic membranes made from metals, ceramics, glasses, etc.

a) Polymer (organic)membranes

Cellulose and its derivatives are more widely used. These hydrophilic polymers are low cost and they are having low tendencies for adsorption. Polyamides, a hydrophilic polymers, are second type of polymer (aromatic polyamides) used for making membrane after cellulose diacetate. It has better thermal, chemical and hydraulic stability than cellulose esters. Amide group(-CO-NH-) cannot tolerate exposure to trace of Cl_2, hence they are not suitable for chlorinated water. Poly acrylonitrile (PAN) are used in UF but not in RO.

Polysulphone (PSF) and Poly ethersulphone (PES) - These two are not hydrophilic and hence they have high adsorption tendency. They have very good thermal, chemical and mechanical stability. Polytetrafluoro ethylene (PTFE), Polyvinylidene fluoride (PVDF), Polyethylene (PE), Polycarbonate (PC) or isotactic Polypropylene (PP) are hydrophilic polymers that are used for membranes.

b) Inorganicmembranes

These membranes have superior thermal, chemical and mechanical stability relative to polymer materials. The disadvantage of this membrane is that they are brittle and more expensive. Ceramic membranes are oxides, nitrides or carbides of metals such as Al, Zr, Ti, etc.

19 : Tertiary Wastewater Treatment

23.3.6 Membranemodule

A single operational unit into which membranes are engineered for used is referred as module. It consists of membranes, pressure support structure, feed inlet and concentrated outlet ports, and permeate draw-off points.

Module configuration: This refers to the packing of the membrane in the module so that it can be installed in the system. Common configurationsinclude:

1) Plate andframe
2) Tubular
3) Spirallywound
4) Hollow finefiber
5) Rotating disc and cylindermodules

1) Plate and framemodule

This module is made from stacked flat sheet membranes and support plates. Plate and frame and tubular membrane are not commonly used for municipal water treatment but are more appropriate for certain waste or food processing applications where there is a high fouling tendency. Feed circulates between membranes of two adjacent plates with thickness of liquid sheet in the range of 0.5 to 3 mm. The packing density of plate and frame module is about 100 to 400 m^2/m^3. Plate ensures mechanical support to membranes and at the same time the drainage of the permeate. Plates can be corrugated on the feed side to improve mass transfer (Figure23.6).

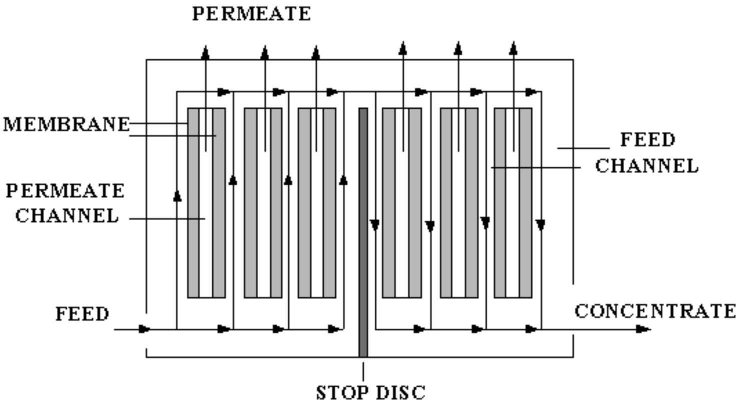


Figure 23.6 Plate and frame module

2) Spirallywound

An envelope of two flat sheet membranes enclosing a flexible porous sheet (permeate collector) is sealed on three of its edges (Figure 23.7). The open end of the membrane envelope is attached around a tube with holes which provide a route for permeate to flow out. The membrane is wound up around the centre tube to form a cylindrical element. Water that has passed through the membrane in service flows towards the centre tube through the porous support. Several sandwiches are thus fastened and separated from one another by a feed side spacer. This spacer maintains open channel flow for feed and also fulfills important function of inducing turbulence, and reducing concentration polarization. The feed flows parallel to permeate tube axis. The diameter of each element is up to 300 mm and length up to 1.5 m. About 2 to 6 elements are inserted in a single cylindrical pressure vessel. This module is more compact and packing density is about 700 to 1000 m^2/m^3. Head loss in this type is lower than that of plate and framemodule.

Flux rates of approximately 0.1 m^3/m^2.day are typical for spiral wound and tubular systems, and considerably lower flux rates are achieved in hollow fiber units. However much more membrane area is available in module packed with hallow fibers than similar size spiral wound modules. Hence, product water per module unit is approximately same. Spiral wound module is more sensitive to clogging than open channel flat sheet systems. It cannot be used without pretreatment. The modules are used in parallel to provide the necessary capacity and in series to increase efficiency.

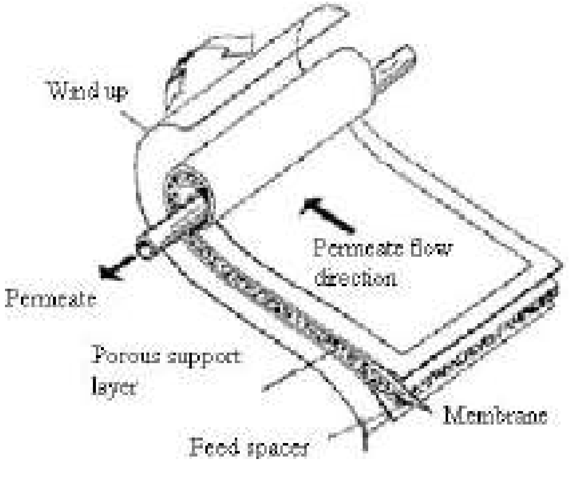


Figure 23.7 Spiral wound type module

3) Tubular module

In this module, membrane is cast on the inside wall of porous support tubes. Tubes are having internal diameter of 6 to 40 mm. They are placed in PVC sleeve or stainless steel for smaller applications (Figure 23.8). This module is more expensive and packing density is low, and it is around 300 m^2/m^3. It does not need fine pre-filtration of the feed. Generally these modules are adapted to the treatment of very viscous fluids.

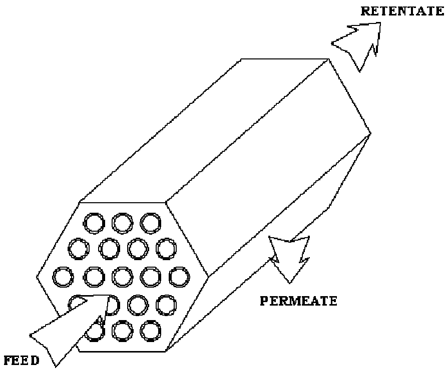


Figure 23.8 Schematic drawing of tubular module

4) HollowFiber

These fibers (Figure 23.9) are gathered in bundles of several thousands. Flow of feed can be either from inside of the fiber or from outside. The hollow-fiber module has highest packing density of about 1000 m^2/m^3 in UF and up to 10000 m^2/m^3 in RO, and it is very compact. Operating velocities are normally low and modules are even operated without recirculation (dead end mode). It has back flushing capabilities for cleaning due to self supporting nature of fibres.

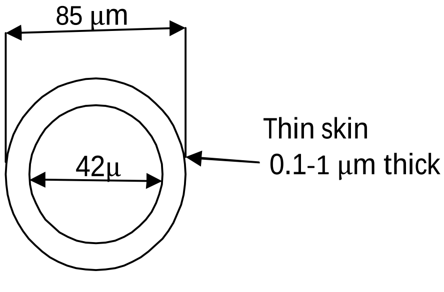


Figure 23.9 Fiber cross section

5) *Rotating disc and cylindermodules*

They promote secondary flows to help depolarise the solute and particle build up at the membrane solution interface. Improved performance in pressure driven filtration in terms of flux is observed in this case. High energy consumption for rotation and difficulties in maintenance are few limitations of this modules.

23.3.7 Electrodialysis

Electrodialysis (ED) is a membrane process, during which ions are transported through semi permeable ion selective membrane, under the influence of an electric potential. This process may be operated on continuous or batch mode. This process uses series of membranes made from ion exchange resins. These membranes will separately transfer ions. One membrane is cation permeable, i.e. it will pass cations (positive ions) but will rejects anions (negative ions), while other membrane is anion permeable, i.e. it will pass anions but it will rejects cations. When parallel channels are constructed by alternating membranes and an electric current is passed across them, an electrodialysis cell is formed as shown in Figure 23.10. Cations are drawn toward the cathode (-ve), passing through the cation selective membrane but being stopped by the anions selective membrane. The opposite action occurs with anions. This results in ions being removed from one channel and concentrated in the adjoiningchannel.

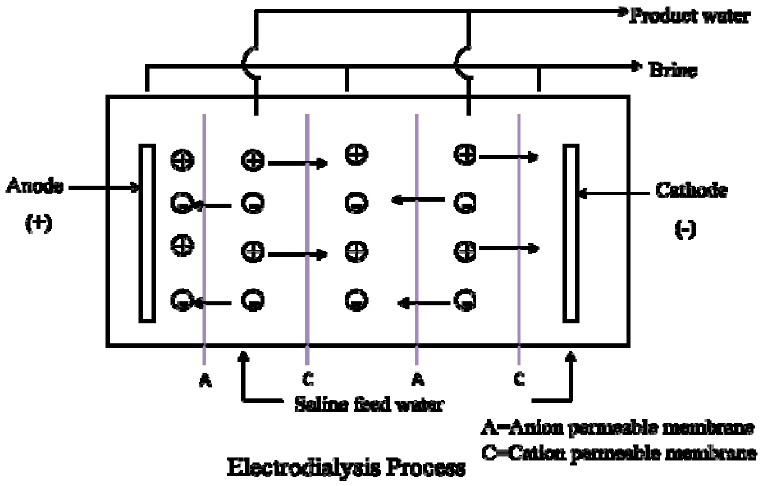


Figure 23.10 Mechanism of electrodialysis

The membranes in electrodialysis are approximately 0.5 mm thick and are separated by porous spaces about 1 mm thick. Water flows through the porous spaces. Several membranes and spaces are sandwiched together into an electrodialysis cell. A contact time of 10 to 20 sec is required for about 60 % efficiency. Cells can be used in parallel to meet flow requirement or used in series to enhance efficiency. Under ideal conditions, 90 % efficiency can be obtained with ions being concentrated in 10 % of thewater.

Both RO and electrodialysis requires a high degree of treatment prior to their application. Suspended solids removal is absolute necessary, and dissolved organics should be removed to prevent fouling. Adjustment of pH to slightly acidic range may be necessary to prevent inorganic precipitation. RO generally produces a higher quality effluent than electrodialysis, although at a higher cost.

Usually about 10 % of the feed volume is required to wash the membrane continuously. The problems associated with this process for wastewater reclaimation includes membrane clogging, chemical precipitation of salts on membrane and fouling by residual organic matter in wastewater treatmentplant.

23.3.8 Granular MediaFiltration

Granular media filtration is employed to remove those suspended and colloidal solids which are carried over from secondary unit processes. This process is sub-divided into two types, *viz*. depth filtration and surface filtration. These can be designed and operated under gravity flow condition (like rapid sand filter) or pressure filtration in a closed reactor.

i) Depth filtration: It is a unit operation that involves the removal of particulate material suspended in effluent of biological and chemical treatment by passing the effluent through a filter bed comprised of a granular medium. It is also used as a pretreatment step for membrane filtration. Filter bed with multimedia of different specific gravity and effective size will be used for facilitating filtration at the greater depths of filter bed. It is commonly used to achieve supplemental removal of suspended solids from wastewater of primary and secondarytreatment.

ii) ***Surface filtration:*** It involves the removal of particulate material suspended in a liquid by mechanical sieving by passing the liquid through a thin filter medium. The filter media in this case is having finer effective size and more uniformity coefficient to make the media graded enough to offer finer pore size.

23.3.9 Disinfection

Primary, secondary and even tertiary treatment cannot be expected to remove 100 percent of the incoming pathogen load and as a result, many organisms still remain in the treated wastewater stream. Thus, whenever wastewater effluents are discharged to receiving waters bodies like river, etc. which may be used for water supply, swimming or shell fishing, the reduction of bacterial numbers to minimize health hazards is a very desirable goal. Disinfection is treatment of the effluent for the partial destruction of all pathogens. Another term that is sometimes also used in describing the complete destruction of microorganisms is called as sterilization. Sterilization is the destruction of all microorganisms; while disinfection indicates the destruction of all disease causing microorganisms. Generally, no attempt is made in wastewater treatment to obtain sterilization. However, disinfection process applied to wastewaters will result in a substantial reduction of all microbes so that bacterial numbers are reduced to a safe level.

Disinfection is most commonly accomplished by the use of 1) chemical agent, 2) physical agent, and 3) radiation. Chemical agent used for treatment includes chlorine, iodine, ozone, phenolic compounds, alcohols, synthetic detergents, hydrogen peroxide and various alkalies and acids. Physical disinfectants include heat, light and sound waves. Chlorination, UV light application, ozonation are few techniques used in disinfections. The later consideration is an issue with the use of ozone and UV radiations where no residual is measured. The time of contact and concentration of disinfectant are two main important factors in disinfection process. There are a number of chemicals and processes that will disinfect wastewater, but none are universally applicable.

23.3.10 Coagulation andflocculation

Coagulation is the process by which colloidal particles and very fine solid suspensions initially present in a wastewater are combined into larger agglomerates that can be separated via flocculation, sedimentation, filtration, centrifugation or other separation methods. The objective of coagulation is to alter these particles in such a way as to allow them to adhere to each other. Coagulation is commonly achieved by adding different types of chemicals (coagulants) to the wastewater to promote destabilization of the colloid dispersion and agglomeration of the resulting individual colloidalparticles.

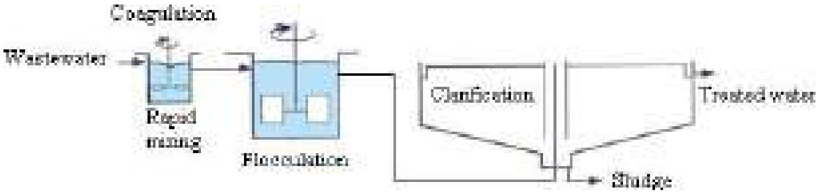


Figure 23.11 Coagulation and flocculation

The usual coagulant is alum [$Al_2(SO_4)_3 \cdot 18H_2O$], though $FeCl_3$, $FeSO_4$ and other coagulants, such as polyelectrolytes, can also be used. Coagulation depends on the rapid reaction through which the coagulant destabilizes the colloids. Coagulation requires the rapid dispersion of the coagulant throughout the wastewater. This is accomplished through very intense agitation in rapid mixing (Figure 23.11). High shear rate is useful forcoagulation.

Flocculation refers to the process by which destabilized particles actually assembled into larger aggregates so that they can be separated from the wastewater in the following clarifier. Flocculation depends on the frequency of collision of the destabilized colloids to form larger floc particles. The frequency of collision depends on the intensity of the agitation and the shear rate. However, if agitation intensity is too high, shear rate may results into breakup of the floc just formed. Therefore, moderate shear rates and agitation intensities (10 to 75 s^{-1}) are used in flocculators.

After flocculation the water is sent to clarifier, where conditions favourable for sedimentation of the flocs are maintained. The settled floc forms a sludge, which need separate disposal. The clear supernatant is collected from the effluent weir provided at the periphery of the clarifier, which is reasonably free from the colloidal solids. If further reduction of colloidal solids is desired, generally sand filter is provided after clarification to produce treated water with very lowturbidity.

23.3.11 Polishingponds

Wastewater treatment ponds may be used as a practical and economical method for upgrading existing secondary treatment facilities to obtain improved organic, suspended solids, nutrients, heavy metals and pathogens removal. Although both aerobic and aerobic-anaerobic ponds can be used as primary ponds for treatment of wastewater the tertiary treatment ponds will be shallow ponds to ensure complete aerobicity in the pond and to achieve more bacterial die-off rate. Ponds used for polishing purposes are lightly loaded ponds in terms of organic matter load and are subjected to the same operating conditions as those used for primary or secondary treatment.

23.3.12 Otherprocesses

Other tertiary treatment processes includes distillation, filtration, coagulation, carbon adsorption, flocculation, lime precipitation, etc. Depending upon the characteristics of effluent to be obtained after secondary treatment, few of these may be used.

Questions

1. Describe tertiary treatmentsystem.
2. Why it is necessary to provide tertiary treatment system during wastewatertreatment.
3. Define: Ion exchange, Reverse Osmosis,Electrodialysis
4. Distinguish between disinfection andsterilization.
5. Write a short note on nitrification anddenitrificaton.
6. What is filtration in advanced wastewater treatment? Describe different types of filtration process.
7. Define membrane process. With neat figure explain mechanism of membraneprocesses.
8. Classify the membraneprocesses.
9. Write a short note on membrane module configuration. Explain any two withdiagram.
10. Distinguish between inorganic and organic polymermembranes.
11. Write a short note on separation mechanism in membraneprocesses.
12. Define reverse osmosis. Explain it inbrief.
13. Write a short note on airstripping.
14. What is coagulation and flocculation? Describe in brief with neatsketch.