19. AEROBIC SECONDARY TREATMENT OF WASTEWATER

19.1 Activated Sludge Process

Conventional biological treatment of wastewater under aerobic conditions includes activated sludge process (ASP) and Trickling Filter. The ASP was developed in England in 1914. The activated sludge process consists of an aeration tank, where organic matter is stabilized by the action of bacteria under aeration and a secondary sedimentation tank (SST), where the biological cell mass is separated from the effluent of aeration tank and the settle sludge is recycled partly to the aeration tank and remaining is wasted (Figure 19.1). Recycling is necessary for activated sludge process. The aeration conditions are achieved by the use of diffused or mechanical aeration.

Diffusers are provided at the tank bottom, and mechanical aerators are provided at the surface of water, either floating or on fixed support. Settled raw wastewater and the returned sludge enter the head of the tank, and cross the tank following the spiral flow pattern, in case of diffuse air aeration, or get completely mixed in case of completely mixed reactor. The air supply may be tapered along the length in case of plug flow aeration tank, to match the quantity of oxygen demand. The effluent is settled in the settling tank and the sludge is returned at a desired rate.

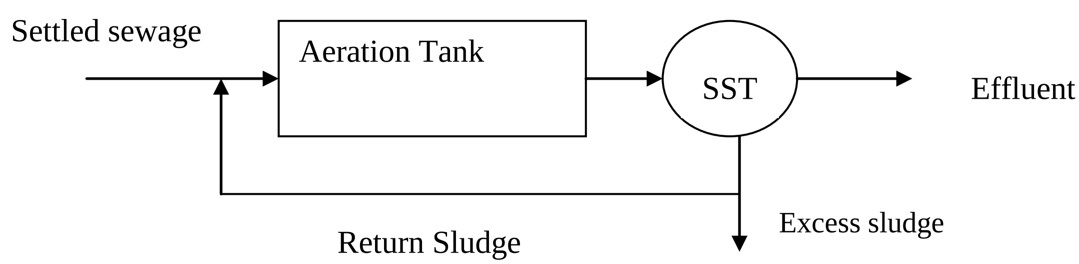


Figure 19.1 Conventional Activated Sludge Process

Loading Rate: The organic matter loading rate applied to the reactor is quantified as kg of BOD applied per unit volume of the reactor per day, called as volumetric loading rate, or kg of BOD applied per day per unit mass of microorganisms present in the reactor (i.e. in the aeration tank), called as organic loading rate or F/M. This can be calculated as stated below:

Volumetric loading = Q x BOD x 10^{-3}/ Vol

Where, BOD = Influent BOD_5 to aeration tank, mg/L

Q = Flow rate, m^3/day

Vol. = Volume of aeration Tank, m^3

Organic Loading Rate, F/M = Q x BOD / (V x X_t)

Where, X_t = MLVSS concentration in the aeration tank, mg/L

The F/M ratio is the main factor controlling BOD removal. Lower F/M values will give higher BOD removal. The F/M can be varied by varying MLVSS concentration in the aeration tank.

Solid Retention Time (SRT) or Mean Cell Residence Time (MCRT): The performance of the ASP in terms of organic matter removal depends on the duration for which the microbial mass is retained in the system. The retention of the sludge depends on the settling rate of the sludge in the SST. If sludge settles well in the SST proper recirculation of the sludge in aeration tank is possible, this will help in maintaining desired SRT in the system. Otherwise, if the sludge has poor settling properties, it will not settle in the SST and recirculation of the sludge will be difficult and this may reduce the SRT in the system. The SRT can be estimated as stated below:

$$SRT = \frac{\text{kg of MLVSS in aeration Tank}}{(\text{kg of VSS wasted per day} + \text{kg of VSS lost in effluent per day})}$$

Generally, the VSS lost in the effluent are neglected as this is very small amount as compared to artificial wasting of sludge carried out from the sludge recycle line or from aeration tank.

Sludge Volume Index: The quantity of the return sludge is determined on volumetric basis. The sludge volume index (SVI) is the volume of the sludge in mL for one gram of dry weight of suspended solids (SS), measured after 30 minutes of settling. The SVI varies from 50 to 150 mL/ g of SS. Lower SVI indicates better settling of sludge.

Quantity of Return Sludge: Usually solid concentration of about 1500 to 3000 mg/L (MLVSS 80% of MLSS) is maintained for conventional ASP and 3000 to 6000 mg/L for completely mixed ASP. Accordingly the quantity of return sludge is determined to maintain this concentration. The sludge return ratio is usually 20 to 50%. The F/M ratio is kept as 0.2 to 0.4 for conventional ASP and 0.2 to 0.6 for completely mixed ASP.

Sludge Bulking: The sludge which does not settle well in sedimentation tank is called as bulking sludge. It may be due to either (a) the growth of filamentous microorganisms which do not allow desirable compaction; or (b) due to the production of non-filamentous highly hydrated biomass. There are many reasons for sludge bulking. The presence of toxic substances in influent, lowering of temperature, insufficient aeration, and shock loading can also cause sludge bulking. Proper supply of air and proper design to maintain endogenous growth phase of metabolism will not produce bulking of sludge. The sludge bulking can be controlled by restoring proper air supply, eliminating shock loading to the reactor, or by increasing temperature of the wastewater or by small hypochlorite dosing to the return sludge line to avoid the growth of filamentous hygroscopic microorganisms.

Mixing Conditions: The aeration tank can be of plug flow type or completely mixed type. In the plug flow tank, the F/M and oxygen demand will be highest at the inlet end of the aeration tank and it will then progressively decrease. In the completely mix system, the F/M and oxygen demand will be uniform throughout the tank.

Flow Scheme: Sewage addition may be done at a single point at the inlet end of the tank or it may be at several points along the aeration tank. The sludge return is carried out from the underflow of the settling tank to the aeration tank. The sludge wastage can be done from return sludge line or from aeration tank itself. Sludge wasting from the aeration tank will have better control over the process, however higher sludge waste volume need to be handled in this case due to lower concentration as compared to when wasting is done from underflow of SST. The compressed air may be applied uniformly along the whole length of the tank or it may be tapered from the head of the aeration tank to its end.

19.1.1 Aeration in ASP

Aeration units can be classified as:

1) Diffused Air Units
2) Mechanical Aeration Units
3) Combined Mechanical and diffused air units.

19.1.1.1 Diffused air aeration

In diffused air aeration, compressed air is blown through diffusers. The tanks of these units are generally in the form of narrow rectangular channels. The air diffusers are provided at the bottom of tank. The air before passing through diffusers must be passed through air filter to remove dirt. The required pressure is maintained by means of air compressors.

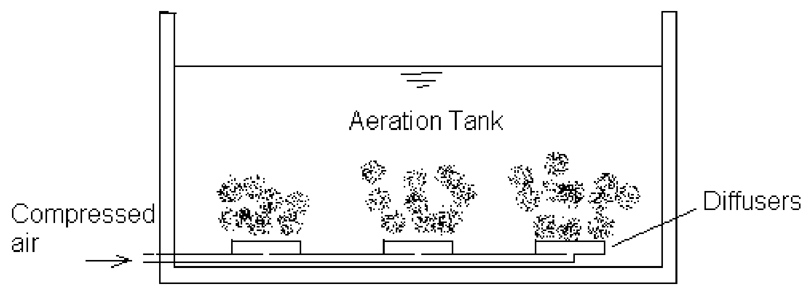


Figure 19.2. Typical air diffusers arrangement

Types of air diffusers

a) <u>Jet diffusers</u>: These diffusers give direct stream of air in the form of jet downward and strike against a small bowl kept just below the nozzle of the jet. The air flashes over the surface of the bowl and escapes in the form of fine bubbles.

b) <u>Porous diffusers</u>: Manufactured in the form of tubes and plates from grains of crushed quartz, aluminum oxide or carbon fused to form a porous structure. These are tile shaped or tubular shape. 10 to 20 % area of the tank is covered with porous tiles. The supply of air is done through pipeline laid in the floor of the tank and is controlled by the valves. Depending upon the size of the air bubbles these can be classified as fine or medium bubble diffused-air aeration device.

In common practice, porous dome type air diffusers of 10 to 20 cm diameter are used. These are directly fixed on the top of C.I. main pipes laid at the bottom of the aeration tanks. These are cheap in initial as well as maintenance cost.

Air Supply: Normally air is supplied under pressure of 0.55 to 0.7 kg/cm^2. The quantity of air supplied varies from 1.25 to 9.50 m^3/m^3 of sewage depending on the strength of the sewage to be treated and degree of treatment desired. The oxygen transfer capacity of the aerators depends on the size of air bubbles, for fine bubble oxygen transfer capabilities of aeration device is 0.7 to 1.4

kg O_2/KW.h. For medium bubble it is 0.6 to 1.0 kg O_2/KW.h, and for coarse bubble it is 0.3 to 0.9 kg O_2/KW.h.

19.1.1.2 Mechanical Aeration Unit

The main objective of mechanical aeration is to bring every time new surface of wastewater in contact with air. In diffuse aeration only 5 to 12% of the total quantity of the air compressed is utilized for oxidation and rest of the air is provided for mixing. Hence, mechanical aeration was developed. For this surface aerators either fixed or floating type can be used (Figure 19.3). The rectangular aeration tanks are divided into square tank and each square section is provided with one mixer. The impeller are so adjusted that when electric motors starts, they suck the sewage from the centre, with or without tube support, and throw it in the form of a thin spray over the surface of the wastewater. When the wastewater is sprayed in the air more surface area of wastewater is brought in contact with the air and hence aeration will occur at accelerated rate. Detention period of the aeration tank treating sewage is usually 5 to 8 hours. The volume of aeration tank should be worked out considering the return sludge volume.

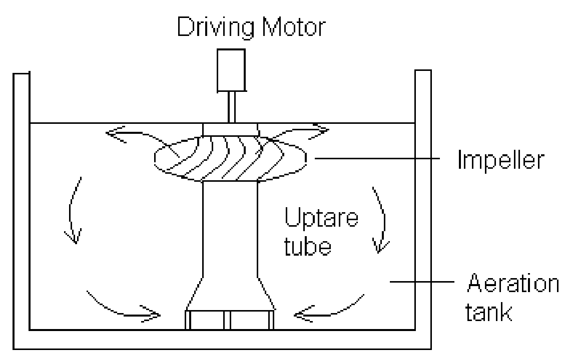


Figure 19.3 Typical arrangement of the surface aerator supported on conical bottom tube

19.1.2 Types of Activated Sludge Process

19.1.2.1 Conventional aeration

In conventional ASP the flow model in aeration tank is plug flow type. Both the influent wastewater and recycled sludge enter at the head of the tank and are aerated for about 5 to 6 hours for sewage treatment (Figure 19.4). The influent and recycled sludge are mixed by the action of the diffusers or mechanical aerators. Rate of aeration is constant throughout the length of the tank. During the aeration period the adsorption, flocculation and oxidation of organic matter takes place. The F/M ratio of 0.2 to 0.4 kg BOD/kg VSS.d and volumetric loading rate of 0.3 to 0.6 kg BOD/m^3.d is used for designing this type of ASP. Lower mixed liquor suspended solids (MLSS) concentration is maintained in the aeration tank of the order of 1500 to 3000 mg/L and mean cell residence time of 5 to 15 days is maintained. The hydraulic retention time (HRT) of 4 to 8 h is required for sewage treatment. Higher HRT may be required for treatment of industrial wastewater having higher BOD concentration. The sludge recirculation ratio is generally in the range of 0.25 to 0.5.

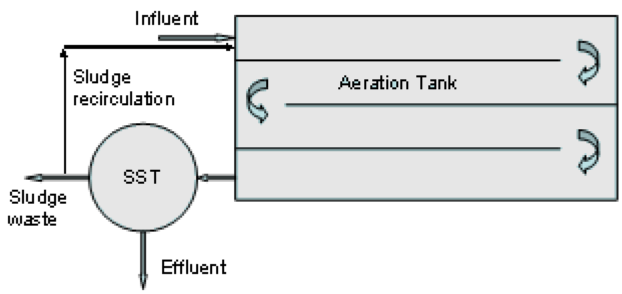


Figure 19.4 Conventional activated sludge process

19.1.2.2 Tapered Aeration

In plug flow type aeration tank BOD load is maximum at the inlet and it reduces as wastewater moves towards the effluent end. Hence, accordingly in tapered aeration maximum air is applied at the beginning and it is reduced in steps towards end, hence it is called as tapered aeration (Figure 19.5). By tapered aeration the efficiency of the aeration unit will be increased and it will also result in overall economy. The F/M ratio and volumetric loading rate of 0.2 to 0.4 kg BOD/kg VSS.d and 0.3 to 0.6 kg BOD/m^3.d, respectively, are adopted in design. Other design recommendation are mean cell residence time of 5 to 15 days, MLSS of 1500 to 3000 mg/L, HRT of 4 to 8 h and sludge recirculation ratio of 0.25 to 0.5. Although, the design loading rates are similar to conventional ASP, tapered aeration gives better performance.

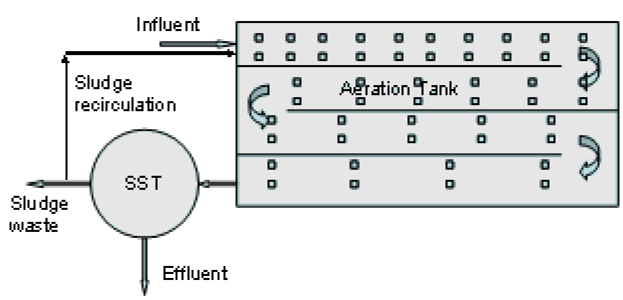


Figure 19.5 Tapered aeration activated sludge process

19.1.2.3 Step aeration

If the sewage is added at more than one point along the aeration channel, the process is called as step aeration (Figure 19.6). This will reduce the load on returned sludge. The aeration is uniform throughout the tank. The F/M ratio and volumetric loading rate of 0.2 to 0.4 kg BOD/kg VSS.d and 0.6 to 1.0 kg BOD/m^3.d, respectively, are adopted in design. Other design recommendation are mean cell residence time of 5 to 15 days, MLSS of 2000 to 3500 mg/L, HRT of 3 to 5 h and sludge recirculation ratio of 0.25 to 0.75. In step aeration the design loading rates are slightly higher than conventional ASP. Because of reduction of organic load on the return sludge it gives better performance.

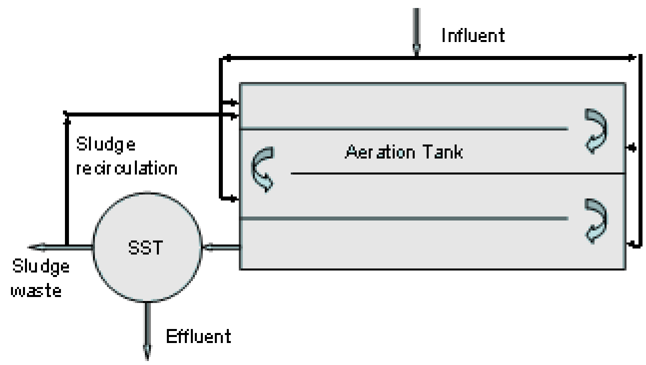


Figure 19.6 Step aeration activated sludge process

19.1.2.4 Completely mixed

In this type of aeration tank completely mixed flow regime is used. The wastewater is distributed along with return sludge uniformly from one side of the tank and effluent is collected at other end of the tank (Figure 19.7). The F/M ratio of 0.2 to 0.6 kg BOD/kg VSS.d and volumetric loading of 0.8 to 2.0 kg BOD/m^3.d is used for designing this type of ASP. Higher mixed liquor suspended solids (MLSS) is maintained in the aeration tank of the order of 3000 to 6000 mg/L and mean cell residence time of 5 to 15 days is maintained. The hydraulic retention time (HRT) of 3 to 5 h is required for sewage treatment. Higher HRT may be required for treatment of industrial wastewater having higher BOD concentration. The sludge recirculation ratio is generally in the range of 0.25 to 1.0. This type of ASP has better capability to handle fluctuations in organic matter concentration and if for some time any toxic compound appears in the influent in slight concentration the performance will not be seriously affected. Due to this property completely mixed ASP is being preferred in the industries where fluctuation in wastewater characteristics is common.

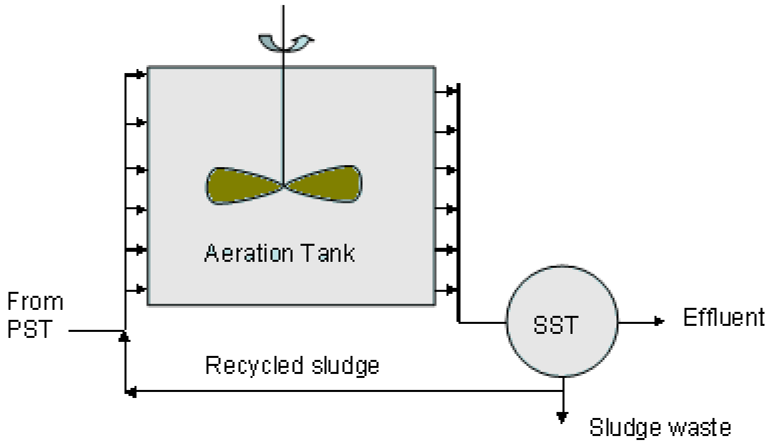


Figure 19.7 Complete mixed activated sludge process

19.1.2.5 Contact Stabilization

It is developed to take advantage of the absorptive properties of activated sludge. The BOD removal in ASP occurs in two phases, in the first phase absorption and second phase of oxidation. The absorptive phase requires 30 to 40 minutes, and during this phase most of the colloidal, finely divided suspended solids and dissolved organic matter get absorbed on the activated sludge. Oxidation of organic matter then occurs. In contact stabilization these two phases are separated out and they occur in two separate tanks (Figure 19.8). The settled wastewater is mixed with re-aerated activated sludge and aerated in the contact tank for 30 to 90 min. During this period the organic matter is absorbed on the sludge flocs. The sludge with absorbed organic matter is separated from the wastewater in the SST. A portion of the sludge is wasted to maintain requisite MLVSS concentration in the aeration tank. The return sludge is aerated before sending it to aeration tank for 3 to 6 h in sludge aeration tank, where the absorbed organic matter is oxidized to produce energy and new cells.

The aeration volume requirement in this case is approximately 50% of the conventional ASP. It is thus possible to enhance the capacity of the existing ASP by converting it to contact stabilization. Minor change in piping and aeration will be required in this case. Contact stabilization is effective for treatment of sewage; however, its use to the industrial wastewater may be limited when the organic matter present in the wastewater is mostly in the dissolved form. Existing treatment plant can be upgraded by changing the piping and providing partition in the aeration tank. This modification will enhance the capacity of the existing plant. This is effective for sewage treatment because of presence of organic matter in colloidal form in the sewage. Contact stabilization may not be that effective for the treatment of wastewater when the organic matter is present only in soluble form.

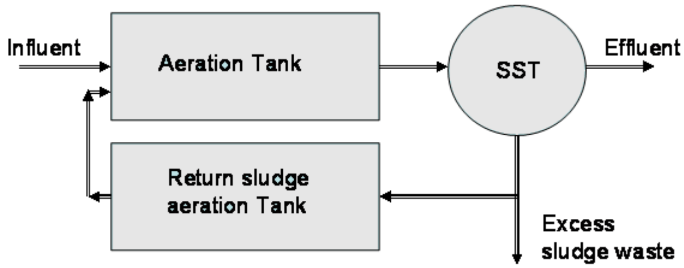


Figure 19.8 Contact stabilization activated sludge process

19.1.2.6 Extended Aeration

In extended aeration process, low organic loading rate (F/M) and long aeration time is used to operate the process at endogenous respiration phase of the growth curve. Since, the cells undergo endogenous respiration, the excess sludge generated in this process is low and the sludge can directly be applied on the sand drying beds where aerobic digestion and dewatering of the sludge occurs. The primary sedimentation can be eliminated when extended aeration process is used to simplify the operation of sludge handling. This type of activated sludge process is suitable for small capacity plant, such as package sewage treatment plant or industrial wastewater treatment plant of small capacity of less than 3000 m^3/day. This process simplifies the sludge treatment and separate sludge thickening and digestion is not required. The aeration tank in this case is generally completely mixed type.

Lower F/M ratio of 0.05 to 0.15 kg BOD/kg VSS.d and volumetric loading of 0.1 to 0.4 kg BOD/m^3.d is used for designing extended aeration ASP. Mixed liquor suspended solids (MLSS) concentration of the order of 3000 to 6000 mg/L and mean cell residence time of 20 to 30 days is maintained. Higher mean cell residence time is necessary to maintain endogenous growth phase of microorganisms. The hydraulic retention time (HRT) of 18 to 36 h is required. The sludge recirculation ratio is generally in the range of 0.75 to 1.5.

19.1.2.7 The Oxidation ditch

It is particular type of extended aeration process, where aeration tank is constructed in the ditch shape (oval shape) as shown in the Figure 19.9. The aeration tank consists of a ring shaped channel 1.0 to 1.5 m deep and of suitable width forming a trapezoidal or rectangular channel cross-section. An aeration rotor, consisting of Kessener brush, is placed across the ditch to provide aeration and wastewater circulation at velocity of about 0.3 to 0.6 m/s.

The oxidation ditch can be operated as intermittent with fill and draw cycles consisting of (a) closing inlet valve and aerating the wastewater for duration equal to design detention time, (b) stopping aeration and circulation device and allowing the sludge to settle down in the ditch itself, (c) Opening the inlet and outlet valve allowing the incoming wastewater to displace the clarified effluent. In case of continuous operation, called as **Carrousel process**, it is operated as a flow through system where wastewater is continuously admitted. The vertically mounted mechanical aerators are used to provide oxygen supply and at the same time to provide sufficient horizontal velocity for not allowing the cells to settle at the bottom of the ditch. Separate sedimentation tank is used to settle the sludge and the settled sludge is re-circulated to maintain necessary MLVSS in the oxidation ditch. The excess sludge generation in oxidation ditch is less than the conventional ASP and can be directly applied to the sand-bed for drying.

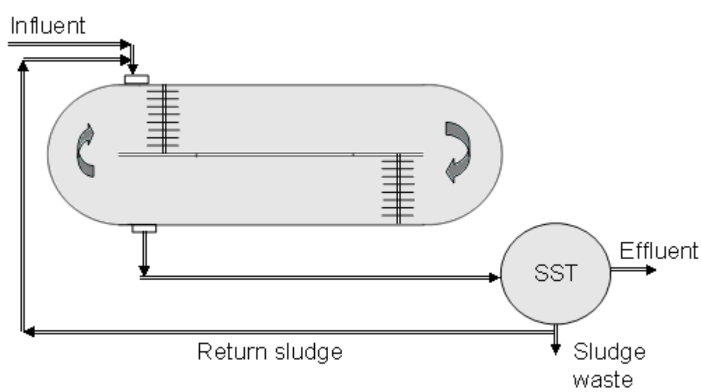


Figure 19.9 Oxidation ditch

19.1.2.8 Sequencing batch reactor (SBR)

A sequencing batch reactor (SBR) is used in small package plants and also for centralized treatment of sewage. The SBR system consists of a single completely mixed reactor in which all the steps of the activated sludge process occurs (Figure 19.10). The reactor basin is filled within a short duration and then aerated for a certain period of time. After the aeration cycle is complete, the cells are allowed to settle for a duration of 0.5 h and effluent is decanted from the top of the unit which takes about 0.5 h. Decanting of supernatant is carried out by either fixed or floating decanter mechanism. When the decanting cycle is complete, the reactor is again filled with raw sewage and the process is repeated. An idle step occurs between the decant and the fill phases. The time of idle step varies based on the influent flow rate and the operating strategy. During this phase, a small amount of activated sludge is wasted from the bottom of the SBR basin. A large equalization basin is required in this process, since the influent flow must be contained while the reactor is in the aerating cycle.

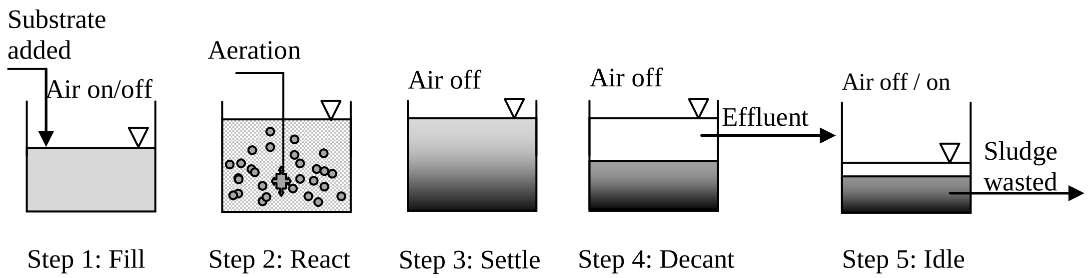


Figure 19.10 Operation cycles of sequencing batch reactor

This process is popular because entire process uses one reactor basin. In areas where there is a limited amount of space, treatment takes place in a single basin instead of multiple basins, allowing for a smaller footprint. In the effluent low total-suspended-solid values of less than 10 mg/L can be achieved consistently through the use of effective decanters that eliminate the need for a separate clarifier. The treatment cycle can be adjusted to undergo aerobic, anaerobic and anoxic conditions in order to achieve biological nutrient removal, including nitrification, denitrification and some phosphorus removal.

19.1.3 Limitations of ASP

For treatment of wastewater with high organic matter concentration, say if the resulting COD concentration in the aeration tank after dilution is in few thousands mg/L, then it will produce biomass of about 50% of the COD concentration. With original biomass concentration plus the generated biomass, the total biomass concentration in the system will be higher. This may pose the difficulty of operating ASP such as uniform aerating the system at such high biomass concentration, and settling and recirculation of the sludge. Hence, this process is not recommended for first stage treatment of high concentrated organic wastewaters.

19.1.4 Kinetics of the Bacterial Growth in Activated Sludge Process

During oxidation of organic matter in ASP following reaction occurs

$$\underset{\text{(organic matter)}}{COHNS} + O_2 + \text{nutrients} \xrightarrow[\text{(bacteria)}]{} CO_2 + NH_3 + \underset{\text{(new cell)}}{C_5H_7O_2N} + \text{Other products}$$

Under endogenous respiration the reaction is

$$\underset{\underset{113}{\text{(cell)}}}{C_5H_7O_2N} + \underset{160}{5\,O_2} \xrightarrow[\text{(bacteria)}]{} 5CO_2 + 2H_2O + NH_3 + \text{energy}$$

The above equation for endogenous respiration tells that for 1 unit mass of cell 160/113 = 1.42 times oxygen is required.

The biomass is the matter of interest rather than the number of organisms for the mixed cultures in the activated sludge process. The rate of biomass increase during the log growth phase is directly proportional to the initial biomass concentration, which is represented by the following first order equation

$$\frac{\mathrm{d}X}{\mathrm{d}t} = \mu X \tag{1}$$

Where $\frac{\mathrm{d}X}{\mathrm{d}t}$ = growth rate of biomass (g/m^3.d)

X = biomass concentration (g/m^3)

μ = specific growth rate constant (d^{-1}). It is the mass of the cells produced per unit mass of the cells present per unit time

If the biomass concentration is X_o, at time t = 0, then integrating Eq. (1),

$$\int_{X_0}^{X} \frac{\mathrm{d}X}{X} = \int_0^t \mu \, \mathrm{d}t$$

$$ln \frac{X}{X_0} = \mu t$$

$$X = X_0 \, e^{\mu t} \tag{2}$$

The exponential growth rate of the bacteria (Eq. 2) occurs as long as there is no change in the biomass composition or environmental condition.

Monod (1949) showed experimentally that the biomass growth rate is a function of biomass concentration and limiting nutrient concentration. The Monod's equation for biomass growth rate is expressed as

$$\mu = \mu_{\mathrm{m}} \frac{S}{K_{\mathrm{s}} + S} \tag{3}$$

Where S = limiting substrate concentration (g/m^3)
μ_{m} = maximum biomass growth rate (d^{-1})
K_{s} = half saturation constant, i.e. substrate concentration at one half maximum growth rate (concentration of S when $\mu = \mu_{\mathrm{m}}/2$, g/m^3)

Eq. (3) assumes only the growth of the microorganisms. However, there is simultaneous die-off of microorganisms. Therefore, an endogenous decay is used to take account of die-off. Hence, Eq. (1) becomes

$$\frac{\mathrm{d}X}{\mathrm{d}t} = \mu X - k_{\mathrm{d}} X$$

$$\frac{\mathrm{d}X}{\mathrm{d}t} = \left(\frac{\mu_{\mathrm{m}} S}{K_{\mathrm{s}} + S}\right) X - k_{\mathrm{d}} X \tag{4}$$

Where k_{d} = endogenous decay rate (d^{-1}). The k_{d} value is in the range of 0.04 to 0.075 per day, typically 0.06 per day.

If all the substrate (organic food, S) could be converted to biomass, then the substrate utilization rate is

$$-\frac{\mathrm{d}S}{\mathrm{d}t} = \frac{\mathrm{d}X}{\mathrm{d}t} \tag{5}$$

However, all the substrates cannot be converted to biomass because of catabolic reaction i.e., energy generation from oxidation of biomass is must for supporting anabolic reaction (biomass synthesis) in the conversion process. Therefore, a yield coefficient ($Y < 1$) is introduced such that the substrate utilization rate is higher than the biomass growth rate.

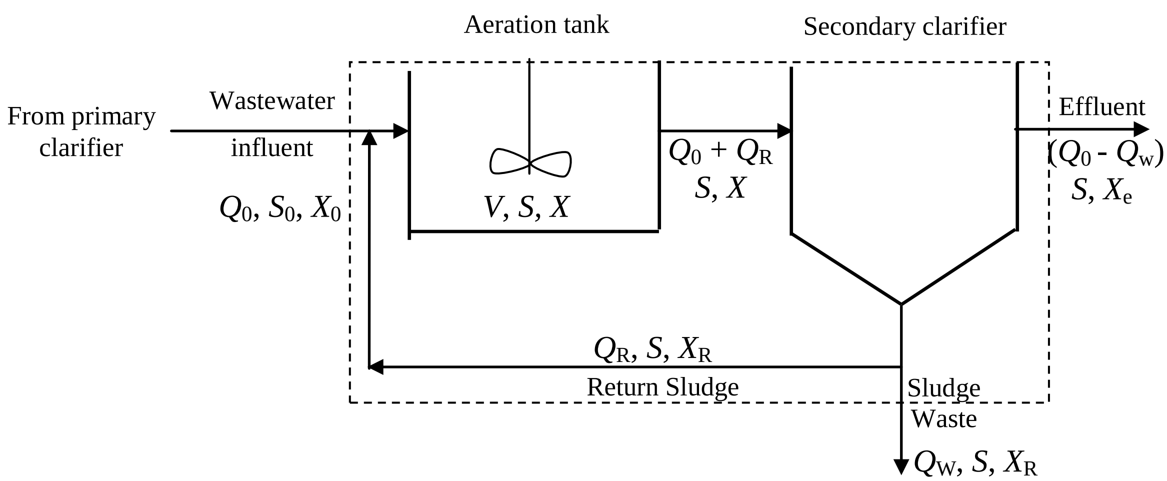
$$-\frac{dS}{dt} = \frac{1}{Y}\frac{dX}{dt} \tag{6}$$

$$-\frac{dS}{dt} = \frac{1}{Y}\frac{\mu_m SX}{k_S + S} \tag{7}$$

Where Y = yield coefficient i.e., fraction of substrate converted to biomass, (g/m^3 of biomass) / (g/m^3 of substrate). The value of Y typically varies from 0.4 to 0.8 mg VSS/mg BOD (0.25 to 0.4 mg VSS/mg COD) in aerobic systems.

19.1.5 Process Analysis of Completely Mixed Reactor with Sludge Recycle

Kinetic models, which have been proposed to describe the activated sludge process, have been developed on the basis of steady-state conditions within the treatment system. The completely mixed reactor with sludge recycle is considered in the following discussion as a model for activated sludge process. The schematic flow diagram shown in Figure 19.11 includes the



nomenclature used in the following mass balance equations.

Figure 19.11 Typical flow scheme for a completely mixed activated sludge system

The mass balance equations used to develop the kinetic models is based on the following assumptions:

- The biomass concentration in the influent is negligible.
- There is complete mixing in the aeration tank.
- The substrate concentration in the influent wastewater remains constant.

- Waste stabilization occurs only in the aeration tank. All reactions take place in the aeration basin so that the substrate in the aeration basin is of the same concentration as the substrate in the secondary clarifier and in the effluent.
- There is no microbial degradation of organic matter and no biomass growth in the secondary clarifier.
- Steady state conditions prevail throughout the system.
- The volume used for calculation of mean cell residence time includes volume of the aeration tank only.

Biomass mass balance

A mass balance for the microorganisms in the completely mixed reactor (Figure 19.11) can be written as follows:

$$\begin{array}{c} \text{Net rate of change in} \\ \text{biomass inside the} \\ \text{system boundary} \end{array} = \begin{array}{c} \text{Rate at which} \\ \text{biomass enters in} \\ \text{the system} \end{array} - \begin{array}{c} \text{Rate at which} \\ \text{biomass leaves} \\ \text{the system} \end{array} \qquad (8)$$

The above mass balance statement can be simplified to

$$\text{Accumulation} = \begin{array}{c} \text{Inflow of} \\ \text{biomass} \end{array} + \begin{array}{c} \text{Net growth} \\ \text{of biomass} \end{array} - \begin{array}{c} \text{Outflow of} \\ \text{biomass} \end{array} \qquad (9)$$

It is assumed that steady state condition prevails in the system; hence accumulation of biomass in the system will be zero. Therefore:

$$\begin{array}{c} \text{Influent} \\ \text{biomass} \end{array} + \begin{array}{c} \text{Biomass} \\ \text{production} \end{array} = \begin{array}{c} \text{Effluent} \\ \text{biomass} \end{array} + \begin{array}{c} \text{Wasted} \\ \text{biomass} \end{array} \qquad (10)$$

$$Q_0 X_0 + V \frac{\mathrm{d}X}{\mathrm{d}t} = (Q_0 - Q_\mathrm{W}) X_\mathrm{e} + Q_\mathrm{W} X_\mathrm{R} \qquad (11)$$

where Q_0 = Influent flow rate (m^3/d)

X_0 = Influent biomass concentration (g/m^3)

V = Volume of the aeration basin (m^3)

Q_W = Flow rate of waste sludge (m^3/d)

X_e = Effluent biomass concentration (g/m^3)

X_R = Biomass concentration in the return sludge (g/m^3)

It is assumed that the biomass concentration in the influent wastewater and in the effluent from the clarifier is negligible, i.e., $X_0 = X_e = 0$. Therefore, Eq. 11 becomes

$$V \frac{dX}{dt} = Q_W X_R \tag{12}$$

Substituting Eq. 4 in Eq. 12,

$$V \left[\left(\frac{\mu_m S}{K_s + S}\right) X - k_d X\right] = Q_W X_R \tag{13}$$

$$\left(\frac{\mu_m S}{K_s + S}\right) = \frac{Q_W X_R}{VX} + k_d \tag{14}$$

If r'_g is net growth of microorganisms, then from equation 13, $r'_g = Q_w X_R/V$

Or we can write $Q_w X_R/V.X = r'_g/X$ (15)

Also, $r'_g = -Y.r_{su} - k_d.X$ (16)

Where, r_{su} is the substrate utilization rate, mass/unit volume.time

Substituting in Eq. 15.

$$Q_w X_R/V.X = -Y.r_{su}/X - k_d \tag{17}$$

The left hand side of the equation is the reciprocal of the mean cell residence time θ_c

Therefore, $1/\theta_c = -(Y.r_{su}/X) - k_d$ (18)

Now, $r_{su} = -Q(So - S)/V = (So - S)/\theta$ (19)

where θ = hydraulic retention time (d)

So = Influent substrate concentration

S = Effluent substrate concentration

From Eq. 19 and Eq. 18

$$1/\theta_c = [Y(So - S)/\theta.X] - k_d \tag{20}$$

Solving for X and substituting $\theta = V/Q$

$$V = \frac{Q.\theta_c.Y(So - S)}{X(1 + k_d.\theta_c)} \tag{21}$$

Equation 21 is used for calculating volume of the aeration tank when the kinetic coefficients are known.

Substrate mass balance

A mass balance for the substrate in the completely mixed reactor (Figure 19.11) using the control volume of the aeration basin and the clarifier can be written as follows:

$$\begin{array}{c}\text{Net rate of change in}\\ \text{substrate inside the}\\ \text{system boundary}\end{array} = \begin{array}{c}\text{Rate at which}\\ \text{substrate enters}\\ \text{in the system}\end{array} - \begin{array}{c}\text{Rate at which}\\ \text{substrate leaves}\\ \text{the system}\end{array} \tag{22}$$

Considering steady state condition prevailing in the system, the above mass balance for the substrate can be simplified to

$$\begin{array}{c}\text{Inflow of}\\ \text{substrate}\end{array} - \begin{array}{c}\text{Consumption}\\ \text{of substrate}\end{array} = \begin{array}{c}\text{Outflow of}\\ \text{substrate}\end{array} + \begin{array}{c}\text{Wasted}\\ \text{substrate}\end{array} \tag{23}$$

$$Q_0\,S_0 - V\,\frac{\mathrm{d}S}{\mathrm{d}t} = (Q_0 - Q_\mathrm{W})\,S + Q_\mathrm{W}\,S \tag{24}$$

Where, S_0 = substrate concentration in the influent (g/m^3)

Substituting Eq. 7 in Eq. 24

$$Q_0\,S_0 + V\left[\frac{1}{Y}\left(\frac{\mu_m S X}{k_S + S}\right)\right] = (Q_0 - Q_\mathrm{W})\,S + Q_\mathrm{W}\,S \tag{25}$$

Rearranging Eq. 25, we get

$$\frac{\mu_m SX}{k_S + S} = \frac{Q_0 Y}{VX}(S_0 - S) \tag{26}$$

Rearranging after combining with Eq. 14

$$S = \frac{K_s(1 + k_d.\theta_c)}{\theta_c(YK - k_d) - 1} \tag{27}$$

Where K = μ_m/Y i.e., it is maximum rate of substrate utilization per unit mass of microorganism.

Hydraulic retention time (HRT)

The hydraulic retention time is calculated as

$$\theta = \frac{V}{Q_0} \tag{28}$$

The usual practice is to keep the detention period between 5 to 8 hours while treating sewage. The volume of aeration tank is also decided by considering the return sludge, which is about 25 to 50% of the wastewater volume.

Mean cell residence time (MCRT)

The mean cell residence time (MCRT) of microorganisms in the system is the length of time the microorganisms stay in the process. This is also called the solids retention time (SRT) or the sludge age. This is expressed as

$$\theta_C = \text{total biomass in the aeration basin/biomass wasted per unit time (d)}$$

$$\theta_C = \frac{VX}{Q_W X_R + (Q_0 - Q_W)X_e} \tag{29}$$

As the value of X_e is negligible, Eq. 29 reduces to

$$\theta_C = \frac{VX}{Q_W X_R} \tag{30}$$

The SRT is higher than the HRT as a fraction of the sludge is recycled back to the aeration basin.

The F/M ratio

The food to microorganism (*F*/*M*) ratio is one of the significant design and operational parameters of activated sludge systems. A balance between substrate consumption and biomass generation helps in achieving system equilibrium. The *F*/*M* ratio is responsible for the decomposition of organic matter. The type of activated sludge system can be defined by its *F*/*M* ratio as below:

- Extended aeration, $0.05 < F/M < 0.15$
- Conventional activated sludge system, $0.2 < F/M < 0.4$
- Completely mixed, $0.2 < F/M < 0.6$
- High rate, $0.4 < F/M < 1.5$

The F/M ratio, kg BOD_5/kg MLVSS.d, is determined as follows:

$$F/M = \frac{\left[\text{BOD of wastewater } (\text{g/m}^3)\right]\left[\text{Influent flow rate } (\text{m}^3/\text{d})\right]}{[\text{Reactor volume } (\text{m}^3)]\,[\text{Reactor biomass } (\text{g/m}^3)]} \tag{31}$$

$$F/M = \frac{S_0\, Q_0}{V\, X} \tag{32}$$

Substituting Eq. 21 into Eq. 26

$$F/M = \frac{S_0}{\theta\, X} \tag{33}$$

Excess sludge wasting

The excess sludge remaining in the secondary clarifier after being recycled to the aeration basin has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity increases with increase in *F*/*M* ratio and decreases with increase in temperature. The excess sludge wasting can be accomplished either from the sludge wasting line or directly from the aeration basin as mixed liquor. Although sludge wasting from sludge return line is conventional, it is more desirable to waste the excess sludge from the aeration basin for better plant control. Sludge wasting from aeration basin is also beneficial for subsequent sludge thickening operations, as higher solid concentrations can be achieved when dilute mixed liquor is thickened rather than the concentrated sludge.

The excess sludge generation under steady state may be estimated from Eq. 29 or from following equation:

$$P_x = Y_{obs}\, Q_0\, (S_0 - S)\, x\, 10^{-3} \qquad (34)$$

Where, P_x = net waste activated sludge produced each day, kg/d

Y_{obs} = Observed sludge yield = $Y/(1 + k_d\, .\, \theta_c)$

Sludge recycling

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settleability and thickening in the secondary clarifier. The recirculation ratio is estimated as stated below considering the mass of microorganisms entering aeration tank and leaving the aeration tank:

$$\frac{Q_R}{Q} = \frac{X}{X_R - X} \qquad (35)$$

Where, Q_R is recycle rate, Q is the flow rate of wastewater, X is MLVSS in aeration tank, and X_R is VSS concentration in return sludge. The sludge setteleability is determined by sludge volume index (SVI). If it is assumed that sedimentation of suspended solids in laboratory is similar to that in the secondary clarifier, then X_R = (VSS/SS ratio)10^6/SVI. Values of SVI between 50 and 150 mL/g indicate good settling of the suspended solids. The X_R value may not be taken as more than 10000 g/m^3 unless separate thickeners are provided to concentrate the settled solids or secondary clarifier is designed to have a higher value.

Oxygen requirement

Oxygen is used as an electron acceptor in the energy metabolism of the aerobic heterotrophic microorganisms present in the activated sludge process. Oxygen is required in the activated sludge process for oxidation of the influent organic matter along with cell growth and endogenous respiration of the microorganisms. The aeration equipments must be capable of maintaining a dissolved oxygen level of about 2 mg/L in the aeration basin while providing thorough mixing of the solid and liquid phase.

The oxygen requirement for an activated sludge system can be estimated by knowing the ultimate BOD of the wastewater and the amount of biomass wasted from the system each day (Metcalf and Eddy, 2003). If all the substrate removed by the microorganisms is totally oxidized for energy purpose, then the total oxygen requirement is calculated as:

$$\text{Total } O_2 \text{ requirement } \left(\frac{g}{d}\right) = \frac{Q\,(S_0 - S)}{f} \quad (35)$$

Where f = ratio of BOD_5 to ultimate BOD

But, all the substrate oxidized is not used for energy. A portion of the substrate is utilized for synthesis of new biomass. As it is assumed that the system is under steady state condition, there is no accumulation of biomass and the amount of biomass produced is equal to the amount of biomass wasted. Therefore, the equivalent amount of substrate synthesized to new biomass is not oxidized in the system and exerts no oxygen demand. The oxygen requirement for oxidizing 1 unit of biomass = 1.42 units. The oxygen requirement for oxidation of biomass produced as a result of substrate utilization is required to be subtracted from the theoretical oxygen requirement given by Eq. 35 to get the actual oxygen requirement.

$$\text{Total } O_2 \text{ requirement (g/d)} = \frac{Q\,(S_0 - S)}{f} - 1.42\, Q_W X_R \quad (36)$$

The above equations (Eq. 36) do not account for nitrification oxygen requirements. The carbonaceous oxygen requirement is only considered in these equations. When nitrification has to be considered, the oxygen requirement will be:

$$\text{Total } O_2 \text{ requirement (g/d)} = \frac{Q\,(S_0 - S)}{f} - 1.42\, Q_W X_R + 4.57\, Q(N_o - N)$$

Where, N_o is the influent TKN concentration, mg/L, N is the effluent TKN concentration, mg/L and 4.57 is the conversion factor for amount of oxygen required for complete oxidation of TKN.

The air supply in aeration tank must be adequate to:

- Satisfy the BOD of the wastewater
- Satisfy the endogenous respiration of the microorganisms
- Provide adequate mixing (15 to 30 $KW/10^3\ m^3$) to keep biomass in suspension.

- Maintain minimum DO of 1 to 2 mg/L throughout the aeration tank.

Typical air requirement for conventional ASP is 30 to 55 m^3/kg of BOD removed. For fine air bubble diffusers it is 24 to 36 m^3/kg of BOD removed. For extended aeration ASP the air requirement is higher of the order of 75 to 115 m^3/kg of BOD removed. To meet the peak demand the safety factor of 2 should be used while designing aeration equipment.

Example:1

Example: Design conventional ASP to treat soluble wastewater from bottle washing plant contains a soluble organic waste having a COD of 300 mg/l. From extensive laboratory studies the BOD_5/COD ratio was found to be 0.60. The average flow rate of 1.0 MLD is to be treated so that effluent SS and BOD_5 should be less than 20 mg/L. Consider following conditions is applicable.

1. Influent VSS are neglegible
2. Return sludge concentration = 8000 mg/L as SS = 6400 mg/L as VSS
3. MLSS = 2500 mg/l
4. MLVSS/MLSS = 0.8
5. Mean cell residence time θc = 8 days
6. Y = 0.50 kg cells/ kg substrate (COD) consumed, K_d =0.06 d^{-1}
7. It is estimated that 80% of effluent solids are biodegradable consider BODu=COD for solids and BOD_5 = 0.6 COD

Determine reactor volume, sludge washing rate, recirculation ratio, and hydraulic retention time for the reactor. Also determine specific utilisation & F/M ratio.

1. Estimate concentration of soluble BOD_5 in the effluent considering effluent SS concentration = 20 mg/L

   Out of which 80% biodegradable

   Hence biodegradable effluent solids= 20×0.8 = 16 mg/l

   COD of solids = 1.42×16 = 22.72 mg/L

   BOD_5 of effluent solids = 22.72×0.6 =13.63 mg/L

Effluent soluble BOD_5 = 20-13.63 = 6.37 mg/L

2. Treatment efficiency

Efficiency based on soluble BOD

Soluble effluent COD = $\frac{6.37}{0.6}$ = 10.62 mg/L

Efficiency

$$\frac{300 - 10.62}{300} \times 100 = 96.46\%$$

Efficiency based on total COD removal

Total COD = Total BOD/0.6 = 20/0.6 = 33.33 mg/L

Efficiency = 300-33.33/300=88.88%

Reactor volume: (Influent BOD5 = 0.6 * 300 = 180 mg/L)

$$V = \frac{Y.Q.\theta_c(S_0 - S)}{X(1 + kd.\theta_c)}$$

$$\frac{0.5 \times 10^3 \times 8(180 - 10.62)}{2500 \times 0.8(1 + 0.06 \times 8)} = 391.05$$

Sludge production rate

$$Y_{obs} = \frac{Y}{1 + kd.\theta c} = \frac{0.5}{1 + 0.06 \times 8} = 0.338 \frac{kg\ VSS}{kg\ COD\ removal}$$

Biomass production rate= Y_{obs}.Q (S_0-S)

$$= 0.338 \times 1000 \times \frac{(180-10.62)}{1000} = \frac{\text{kg}}{\text{VSS}}/\text{Day}$$

Sludge washing rate from recycle line

$$\theta_c = \frac{V.X}{QwXr}$$

$$Q_w = \frac{391.05 \times 2000}{8 \times 6400} = 15.27\ m^3/d$$

Sludge wasted in mass (kg)

Wasted sludge = $\frac{8000}{1000} \times 15.27 = 121.25 \frac{kg}{day} \approx \frac{97}{0.8} = 122\ kgss/d$

Recycle ratio

$Qr.Xr = (Q+Q_R)\ X$

$$\frac{Qr}{Q} = \frac{X}{X_r - X} = \frac{2000}{6400 - 2000} = 0.45$$

Hydraulic retention time=

$$\frac{V}{Q} = \frac{391.05}{1000} = 0.39\ day = 9.38h$$

Specific substrate utilisation rate

$$U = \frac{S_0 - S}{\theta.X} = \frac{(300 - 10.6)}{0.39 \times 2000} = 0.37\frac{mg\ COD\ utilised}{mg\ VSS.d}$$

$$F/_M = \frac{S_0}{\theta.X} = \frac{300}{0.39 \times 2000} = 0.385\frac{mgCOD}{mg\ VSS.d} \quad (\frac{F}{M} \times efficiency = 0.37 = U)$$

Oxygen requirment = $\frac{Q(S_0 - S)}{0.6} - 1.42(px)$

Example 2

Design a complete mixed activated sludge process aeration tank for treatment of 4 MLD wastewater having BOD concentration of 180 mg/L. The effluent should have soluble BOD of 20 mg/L or less. Consider the following:

MLVSS/MLSS = 0.8

Return sludge SS concentration = 10000 mg/L

MLVSS in aeration tank = 3500 mg/L

Mean cell residence time adopted in design is 10 days

Solution

a) Treatment efficiency based on soluble BOD

$\eta = (180 - 20)*100/180 = 88.89\%$

b) Calculation of reactor volume, Q = 4 MLD = 4000 m^3/d, Y = 0.5 mg/mg, k_d = 0.06 per day

$$V = \frac{Q.\theta_c.Y((So - S)}{X(1 + k_d.\theta_c)}$$

Therefore,

$$V = \frac{4000 \text{ x } 10 \text{ x } 0.5\ (180 - 20)}{3500(1 + 0.06 \text{ x } 10)}$$

= 571.43 m^3

c) Calculate HRT

θ = V/Q = 571.43 * 24 / 4000 = 3.43 h (within 3 to 5 h)

d) Check for F/M

$F/M = \frac{Q.S_0}{V\ X}$ = 4000 * 180/(571.43 * 3500) = 0.36 kg BOD/kg VSS.d (within 0.2 – 0.6)

e) Check for volumetric loading

= Q . So /V = 4000 * 180 * 10^{-3}/571.43 = 1.26 kg BOD/m^3.d (within 0.8 to 2.0)

f) Quantity of sludge waste

Y_{obs} = Y/(1 + k_d . θ_c) = 0.5/(1 + 0.06*10) = 0.3125 mg/mg

Therefore, mass of volatile waste activated sludge

$P_x = Y_{obs}\, Q_0\, (S_0 - S)\, x\, 10^{-3}$ = 0.3125 * 4000 (180 – 20) * 10^{-3}

= 200 kg VSS/day

Therefore, mass of sludge based on total SS = 200/0.8 = 250 kg SS/d

g) Sludge waste volume based on mean cell residence time

$\theta_C = \frac{VX}{Q_W X_R}$ = 571.43 * 3500 / (Q_w * 10000*0.8) = 10 days

Hence, Q_w = 25.0 m^3/d (when wasting is done from the recycled line of SST)

h) Estimation of recirculation ratio

3500 (Q + Q_R) = 8000 Q_R

Therefore, Q_R/Q = 0.78

i) Estimation of air requirement

Total O_2 requirement (g/d) $= \frac{Q\ (S_0 - S)}{f}$ - 1.42 $Q_W X_R$

kg of oxygen required = [(4000(180 – 20) * 10^{-3})/0.68] - 1.42 * 25 * 8000 * 10^{-3}

= 657.17 Kg O_2/d

j) Volume of air required, considering air contain 23% oxygen by weight and density of air 1.201 kg/m^3

= 657.17/(1.201 * 0.23) = 2379.1 m^3/d

Considering oxygen transfer efficiency of 8%, the air required = 2379.1/0.08 = 29738.34 m^3/d

= 20.65 m^3/min

Considering safety factor of 2, the air requirement is = 2 x 20.65 = 41.30 m^3/min

k) Check for air volume

Air requirement per unit volume = 29738.34/ 4000 = 7.44 m^3/m^3

(Within the limit of 3.75 to 15 m^3/m^3)

Air requirement per kg of BOD_5 = 29738.34/ [(180-20) * 4000 * 10^{-3}] = 46.46 m^3/kg of BOD_5 (within the limit of 30 to 55 m^3/kg of BOD_5)