waste, pH being one of the more important factors. Optimum dosage is determined by conducting Jar Test in the laboratory.

12.6.1.1 Iron Salts

Ferric salts are better coagulants than ferrous salts because of their higher valency and their efficiency over a wider pH range. Ferric salts are effective at approximate pH values above 3, the efficiency increasing with increase in pH, while the useful pH range of ferrous salts is above 10. But when waste waters are highly alkaline due to presence of trade wastes, it may be cheaper to use larger dosage of ferrous salts as they are relatively cheaper. Chlorinated copperas which is an equimolar mixture of ferric sulphate and ferric chloride formed by the addition of chlorine to ferrous sulphate is also used in place of ferric salts.

12.6.1.2 Aluminium Salts

Aluminium chloride and sulphate of alumina (filter alum) are the commonly used aluminium salts. Where alum is used, the sludge produced is greater in volume and also bulky than with iron salts making it less easily settleable.

12.6.1.3 Lime and Sodium Carbonate

These are used for pH adjustment to favourable ranges of coagulants especially when sewage is highly acidic. Lime is sometimes used independently as precipitant, particularly when iron pickling liquors are present in sewage. The action may be due to formation of calcium carbonate floc or reactions with small amounts of aluminium or iron salts present in sewage. Lime incidentally helps in grit separation, oil and grease removal and is perhaps the cheapest chemical used in chemical precipitation.

12.6.2 Unit Operations

The process consists of the three unit operations viz., proportioning and mixing of chemicals, flocculation and sedimentation.

12.6.2.1 Mixing

The required dose of chemical is weighed and fed to sewage by means of proportioning and feeding devices, ahead of the mixing unit. Mixing is accomplished in a rapid or flash mixing unit provided with paddles, propellers or by diffused air and having detention period of 0.5 to 3 minutes. The paddles of propellers are mounted on a vertical shaft and driven by a constant speed motor through reduction gears. The size and speed of the propeller is so selected as to give a propeller capacity of twice the maximum flow through the tank. The shaft speed is generally of the order of 100 - 120 rpm and power requirement is about 0.1 kw/mLd.

12.6.2.2 Flocculation

The principle of flocculation in sewage is similar to flocculation in water purification. The floccules that are formed after flash mixing with chemicals are made to coalesce into bigger sizes by either air flocculation or mechanical flocculation. Both diffused air and mechanical vertical draft tube are used for air flocculation. Revolving paddle type is the most common of the mechanical flocculators. The tanks are usually in duplicate with a detention period of 30-90 minutes depending upon results required and the type of sewage treated. However, the dose of chemical required as well as the flocculation period are best determined by laboratory test followed by pilot plant studies for optimum results. The paddles are mounted either on a horizontal or vertical shaft. The peripheral speed of the paddles is kept in the range of 0.3 to 0.45 mps. The flow-through velocity through the flocculator should be in the range of 15 to 25 cm/sec to prevent sedimentation.

In case of domestic sewage and certain industrial wastes, mechanical flocculation without addition of chemicals will induce self-flocculation of the finely divided suspended solids and hence increase the efficiency of sedimentation

12.6.2.3 SEDIMENTATION

The flocculated sewage solids are settled out in a subsequent sedimentation tank. The design features of these tanks are similar to secondary settling tanks as discussed in 12.3.2. Usually detention period of 2 hrs and an overflow rate of not more than 50 $m^3/d.m^2$ for average flows is adopted in the design of these sedimentation tanks.

CHAPTER 13

AEROBIC SUSPENDED GROWTH SYSTEMS

13.1 INTRODUCTION

Aerobic suspended growth systems are of two basic types, those which employ sludge.recirculation, viz., conventional activated sludge process and its modifications and those which do not have sludge recycle, viz., aerated lagoons. In both cases sewage containing waste organic matter is aerated in an aeration basin in which micro-organisms metabolize the soluble and suspended organic matter. Part of the organic matter is synthesized into new cells and part is oxidized to carbon dioxide and water to derive energy. In activated sludge systems the new cells formed in the reaction are removed from the liquid stream in the form of a flocculant sludge in settling tanks. A part of this activated sludge is recycled to the aeration basin and the remaining forms waste or excess sludge. In aerated lagoons the microbial mass leaves with the effluent stream or may settle down in areas of the aeration basin where mixing is not sufficient.

The suspended solids concentration in the aeration tank liquor, also called mixed liquor suspended solids (MLSS), is generally taken as an index of the mass of active micro-organisms in the aeration tank. However, the MLSS will contain not only active micro-organisms but also dead cells as well as inert organic and inorganic matter derived from the influent sewage. The mixed liquor volatile suspended solids (MLVSS) value is also used and is preferable to MLSS as it eliminates the effect of inorganic matter.

Aerobic and facultative bacteria are the predominant micro-organisms which carry out the above reactions of organic matter i.e. oxidation and synthesis. Their cellular mass contains about 12% Nitrogen and 2% Phosphorous. These nutrients should be present in sufficient quantity in the waste or they may be added, required, for the reactions to proceed satisfactorily. A generally recommended ratio of BOD_5:N:P is 100:5:1. Domestic wastewater is generally balanced with respect to these nutrients.

13.2 ACTIVATED SLUDGE PROCESS VARIABLES

An activated sludge plant essentially consists of the following: (i) Aeration tank containing microorganisms in suspension in which the reaction takes place, (ii) Activated sludge recirculation system, (iii) Excess sludge wasting and disposal facilities, (iv) Aeration systems to transfer oxygen and (v) Secondary sedimentation tank to separate and thicken activated sludge. These are schematically illustrated in Fig.13.1 (a) to (e).

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

13.2.1 Loading Rate

The loading rate expresses the rate at which the sewage is applied in the aeration tank. A loading parameter that has been developed empirically over the years is the hydraulic retention time (HRT), Θ, d

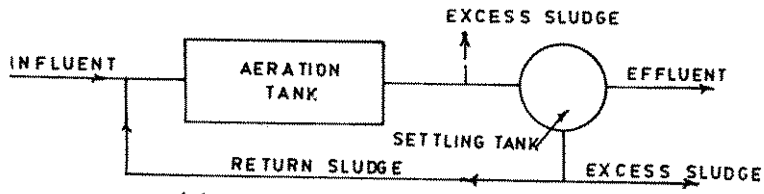


FIG.(a): CONVENTIONAL ACTIVATED SLUDGE

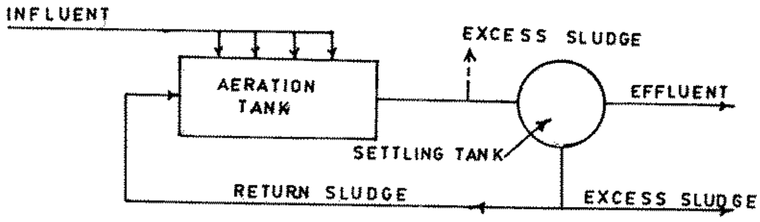


FIG.(b): STEP AERATION

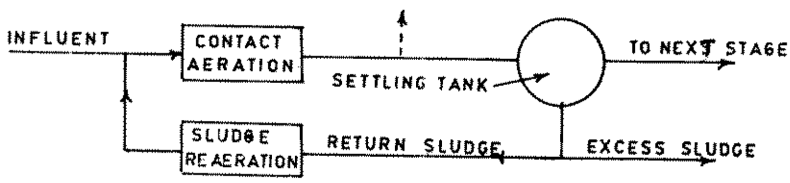


FIG.(c): CONTACT STABILISATION

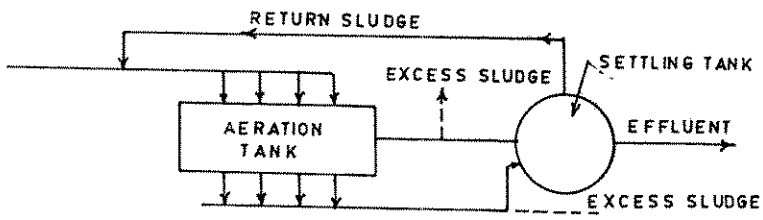


FIG.(d): COMPLETE MIX PLANT

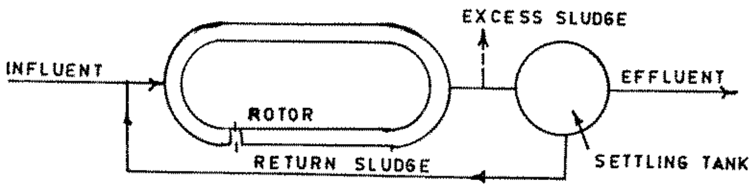


FIG.(e): OXIDATION DITCH

FIG.13.1: SCHEMATIC DIAGRAMS OF ACTIVATED SLUDGE TREATMENT WITH DIFFERENT MODIFICATIONS.

$$\theta = \frac{V}{Q} \quad (13.1)$$

Where

V = volume of aeration tank, m^3, and

Q = sewage inflow, m^3/d

Another empirical loading parameter is volumetric organic loading which is defined as the BOD applied per unit volume of aeration tank, per day.

A rational loading parameter which has found wider acceptance and is preferred, is specific substrate utilization rate, U, per day which is defined as:

$$U = \frac{Q\ (S_o - S)}{V\ X} \quad (13.2)$$

A similar loading parameter is mean call residence time or sludge retention time (SRT), Θ_c, day:

$$\Theta_c = \frac{VX}{Q_w X_s} \quad (13.3)$$

Where S_o and S are influent and effluent organic matter concentrations respectively, conventionally measured as BOD_5, (g/m^3) X and X_s are MLSS concentration in aeration tank and waste activated sludge from secondary settling tank under flow, respectively, (g/m^3) and Q_w = waste activated sludge rate, (m^3/d).

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

$$Q_w X_s = YQ\ (S_o - S) - k_d X V \quad (13.4)$$

Where Y = maximum yield coefficient (microbial mass synthesized/mass of substrate utilised) and k_d = endogenous respiration rate constant, (d^{-1}).

From the above equations it is seen that

$$1 / \Theta_c = YU - k_d \quad (13.5)$$

Since both Y and k_d are constants for a given waste, it is, therefore, necessary to define either Θ_c or U Eq. (13.5) is plotted in Fig.13.2 for typical values of Y = 0.5

and $k_d = 0.06/d$ for municipal wastewaters.

If the value of S is small compared to S_o, which is often the case for activated sludge systems treating municipal wastewater, U may also be expressed as Food applied to Microorganism ratio, F/M:

$$F / M = QS_o / XV \tag{13.6}$$

The Θ_c value adopted for design controls the effluent quality, and settleability and drainability of biomass. Other operational parameters which are affected by the choice of Θ_c values are oxygen requirement and quantity of waste activated sludge. Figure 13.3 gives Θ_c value as a function of temperature for 90-95% reduction of BOD of municipal wastewaters. Typical values of loading parameters for various activated sludge modifications commonly used in India are furnished in Table 13.1.

13.2.2 Mixing Regime

The mixing regime employed in the aeration tank may be plug flow or completely mixed flow. Plug-flow implies that the sewage moves down progressively along the aeration tank essentially unmixed with the rest of the tank contents. Completely mixed flow involves rapid dispersal of the incoming sewage throughout the tank. In the plug flow system, the F/M and the oxygen demand will be highest at the inlet end of the aeration tank and will then progressively decrease. In the completely mixed system, the F/M and the oxygen demand will be uniform throughout the tank.

13.2.3 Flow Scheme

The flow scheme involves the pattern of sewage addition and sludge return to the aeration tank and also the pattern of aeration. Sewage addition may be 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 may be directly from the settling tank to the aeration tank or through a sludge reaeration tank. Aeration may be at a uniform rate or it may be varied from the head of the aeration tank to its end.

13.3 CONVENTIONAL SYSTEM AND MODIFICATIONS

The conventional system represents the early development of the activated sludge process. Over the years, several modifications to the conventional system nave been developed to meet specific treatment objectives by modifying the process variables discussed in 13.2.

In step aeration, settled sewage is introduced at several points along the tank length which produces a more uniform oxygen demand throughout. Tapered aeration attempts to supply air to match oxygen demand along the length of the tank. Contact stabilization provides for reaeration of return activated sludge from the final clarifier, which allows a smaller aeration or contact tank. While conventional system maintains a plug flow hydraulic regime, completely mixed process aims at instantaneous mixing of the influent waste and return sludge with the entire contents of the aeration tank. Extended aeration process operates at a low organic load producing lesser quantity of well stabilized sludge. The conventional system and the last two modifications named above have found wider acceptance. These are described below in greater detail.

13.3.1 Conventional System

The Conventional system is always preceeded by primary settling. The plant itself consists of an aeration tank, a secondary settling tank, a sludge return line and an excess sludge waste line leading to a digester.

The BOD removal in the process is 85-92 percent. The plant employs a plug flow regime which is achieved by a long and narrow configuration of the aeration tank with length equal to 5 or more times the width. The sewage and mixed liquor are let in at the head of the tank and withdrawn at its end. Because of the plug flow regime, the oxygen demand at the head of the aeration tank is high and then tapers down. However, air is supplied in the process at a uniform rate along the length of the tank. This leads to either oxygen deficiency in the initial zone or wasteful application of air in the subsequent reaches. Another limitation of the plug flow regime is that there is a lack of operational stability at times of excessive variation in rate of inflow and in influent strength. For historical reasons, the conventional system is the most widely used type of the activated sludge process. Plants upto 300 mld capacity have been built in India.

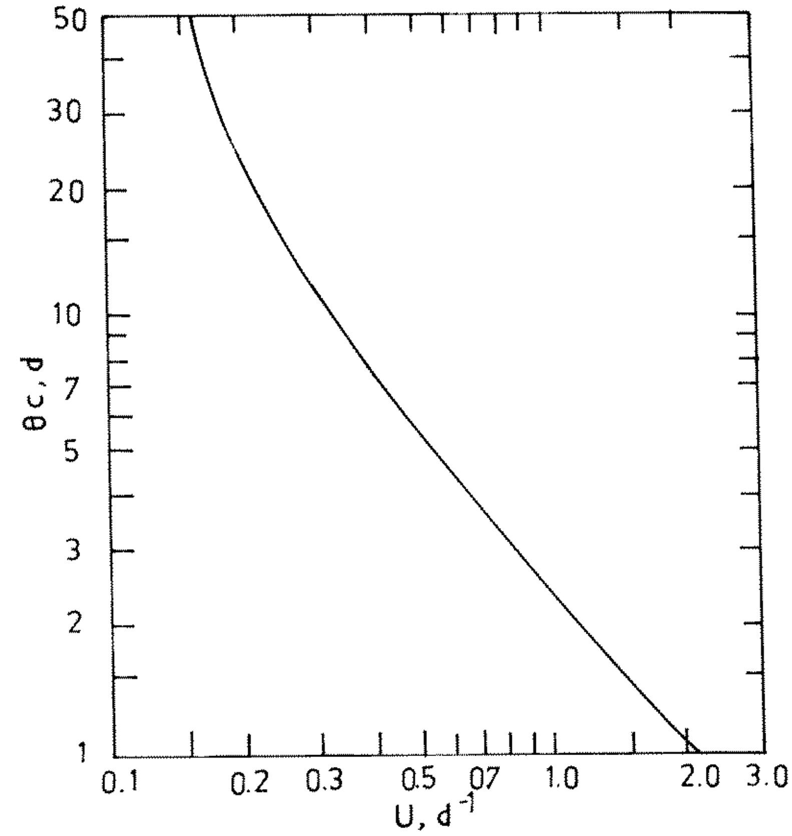


FIG.13.2: RELATIONSHIP BETWEEN SRT(θc) AND SPECIFIC SUBSTRATE UTILIZATION RATE (U) FOR Y = 0.5 AND k = 0.06 d^{-1}

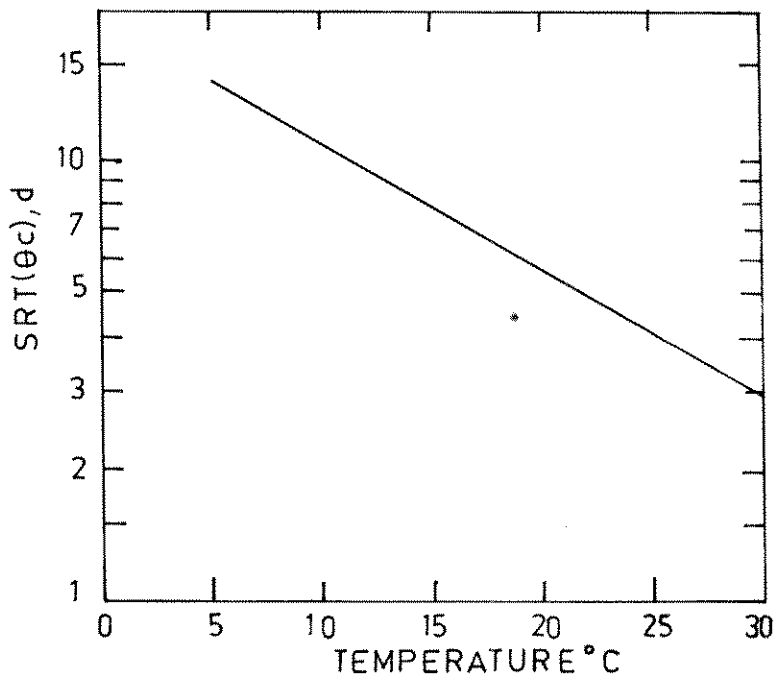


FIG.13.3: SRT AS A FUNCTION OF AERATION BASIN TEMPERATURE FOR 90-95% BOD REMOVAL

13.3.2 Completely Mixed

The complete mix activated sludge plant employs a completely mixed flow regime. In a rectangular tank, complete mixing is achieved by distributing the sewage and the return sludge uniformly along one side of the tank and withdrawing the aerated sewage uniformly along the opposite side. In a circular or square tank complete mixing is achieved by mechanical aerators with adequate mixing capacity installed at the centre of the tank.

The completely mixed plant has the capacity to hold a high MLSS level in the aeration tank enabling the aeration tank volume to be reduced. The plant has increased operational stability at shock loadings and also increased capacity to treat toxic biodegradable wastes like phenols.

13.3.3 Extended Aeration

The flow scheme of the extended aeration process and its mixing regime are similar to that of the completely mixed process except that primary settling is omitted. The process employs low organic loading, long aeration time, high MLSS concentration and low F/M. The BOD removal efficiency is high. Because of long detention in the aeration tank, the mixed liquor solids undergo considerable endogenous respiration and get well stabilised. The excess sludge does not require separate digestion and can be directly dried on sand beds. Also the excess sludge production is a minimum.

The oxygen requirements for the process is higher and the running costs are also therefore high. However, operation is rendered simple due to the elimination of primary settling and separate sludge digestion. The method is, therefore, well suited specially for small and medium size communities and zones of a larger city.

In small plants intermittent operation of extended aeration systems may be adopted. Intermittent aeration cycles are: (i) closing of inlet and aerating the sewage, (ii) stopping aeration and letting the contents settle and (iii) letting in fresh sewage which displaces an equal quantity of clarified effluent. Sludge is wasted from the mixed liquor. To handle continuous flows a number of units may be operated in parallel.

The oxidation ditch is one form of an extended aeration system having certain special features like an endless ditch for the aeration tank and a rotor for the aeration mechanism. The ditch consists of a long continuous channel usually oval in plan. The channel may be earthen with lined sloping sides and lined floor or it may be built in concrete or brick with vertical walls. The sewage is aerated by a surface rotor placed across the channel. The rotor not only aerates the sewage but also imparts a horizontal velocity to the mixed liquor preventing the biological sludge from settling out.

13.4 DESIGN CONSIDERATION

The items for consideration in the design of activated sludge plant are aeration tank capacity and dimensions, aeration facilities, secondary sludge settling and recycle and excess sludge wasting.

13.4.1 Aeration Tank

Equations 13.2 to 13.4 can be combined to yield

$$VX = \frac{YQ(S_o - S)}{1 + k_d \theta_c} \quad (13.7)$$

The volume of the aeration tank is calculated for the selected, value of Θ_c by assuming a suitable value of MLSS concentration, X, in Eq. (13.7).

Alternatively the tank capacity may be designed from F/M and MLSS concentration according to Equation 13.6. The F/M and MLSS levels generally employed in different types of commonly used activated sludge systems are given in Table 13.1 along with their corresponding BOD removal efficiencies.

It is seen that economy in reactor volume can be achieved by assuming a large value for X. However, it is seldom taken to be more than 5000 g/m^3. A common range is between 1000 and 4000 g/m^3. Considerations which govern the upper limit are: initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitations of oxygen transfer equipment to supply oxygen at required rate in a small reactor volume, increased solids loading on secondary clarifier which may necessitate a larger surface area to meet limiting solid flux, design criteria for the tank and minimum HRT for the aeration tank for stable operation under hydraulic surges.

Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. This configuration is achieved by the provision of round-the-end baffles in small plants when only one or two tank units are proposed and by construction as long and narrow rectangular tanks with common intermediate walls in large plants when several units are proposed. In extended aeration plants other than oxidation ditches and in complete mix plants the tank shape may be circular or square when the plant capacity is small or rectangular with several side inlets and equal number of side outlets, when the plant capacity is large.

The width and depth of the aeration channel depends on the type of aeration equipment employed. The depth controls the aeration efficiency and usually ranges from 3 to 4.5 m, the latter depth being found to be more economical for installations treating more than 50 mld. Beyond 70 mld duplicate units are preferred. The width controls the mixing and is usually kept between 5 and 10 m. Width-depth ratio should be adjusted to be between 1.2 to 2.2. The length should not be less than 30 or not ordinarily longer than 100 m in a single section length before doubling back. The horizontal velocity should be around 1.5 m/min. Excessive width may lead to settlement of solids in the tank. Triangular baffles and fillets are used to eliminate dead spots and induce spiral flow in the tanks. Tank free-board is generally kept between 0.3 and 0.5 m.

Due consideration must be given in the design of aeration tanks to the need for emptying them for maintenance and repair of the aeration equipment. Intermediate walls should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction.

The inlet and outlet channels of the aeration tank should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction.

The inlet and outlet channels of the aeration tanks should be designed to maintain a minimum velocity of 0.2 mps to avoid deposition of solids. The channels or conduits and their appurtenances should be sized to carry the maximum hydraulic load to the remaining aeration tank units when any one unit is out of operation.

The inlet should provide for free fall into aeration tank when more than one tank unit or more than one inlet is proposed. The free fall will enable positive control of the flows through the different inlets. Outlets usually consist of free fall weirs. The weir length should be sufficient to maintain a reasonably constant water level in the tank. When multiple inlets or multiple tanks are involved, the inlets should be provided with valves, gates or stop planks to enable regulation of flow through each inlet.

13.4.2 Oxygen Requirements

Oxygen is required in the activated sludge process for the oxidation of a part of the influent organic matter and also for the endogenous respiration of the micro-organisms in the system.

The total oxygen requirement of the process may be tormulated as follows:

$$O_2 required \frac{g}{d} = \frac{Q(S_o - S)}{f} - 1.42 Q_w X_s \qquad (13.8)$$

Where

f = ratio of BOD_5 to ultimate BOD and 1.42 = oxygen demand of biomass, g/g.

The formula does not allow for nitrification but allows only for carbonaceous BOD removal. The extra theoretical oxygen requirement for nitrification is 4.56 Kg O_2/per kg NH_3 -N oxidised to NO_3 - N

The total oxygen requirements per Kg BOD_5 removed for different activated sludge processes are given in Table 13.1. The amount of oxygen required for a particular process will increase within the range shown in the table as the F/M value decreases.

13.4.3 Aeration Facilities

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the waste water against a specific level of dissolved oxygen in the waste water. The aeration devices apart from supplying the required oxygen demand shall also provide adequate mixing or agitation in order that the entire mixed liquor suspended solids present in the aeration tank will be available for the biological activity. The recommended dissolved oxygen concentration in the aeration tank is in the range 0.5 to 1 mg/l for conventional activated sludge plants and in the range 1 to 2 mg/l for extended aeration type activated sludge plants and above 2 mg/l when nitrification is required in the activated sludge plant.

Aerators are rated based on the amount of oxygen they can transfer to tap water under standard conditions of 20° C, 760 mm Hg barometric pressure and zero DO.

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

$$N = \frac{N_s(C_s - C_L) \times 1.024^{T-20} \alpha}{9.17} \qquad (13.9)$$

Where

N = oxygen transferred under field conditions, Kg O_2/hr

N_s = oxygen transfer capacity under standard conditions, kg O_2/hr

C_s = dissolved oxygen saturation value for sewage at operating temperature

C_L = operation DO level in aeration tank usually 1 to 2 mg/1

T = Temperature, degree C

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

Oxygen may be supplied either by surface aerators or diffused air aeration systems employing fine or coarse diffusers. In India surface aerators are the method of choice because of easier maintenance. The oxygen transfer capacities of surface, fine and coarse diffused air systems under standard conditions lie between 1.2-2.4, 1.2-2 and 0.6-1.2 kg O_2/kw.h., respectively.

13.4.3.1 DIFFUSED AIR AERATION

Diffused air aeration involves the introduction of compressed air into the sewage through submerged diffusers or nozzles. The aerators may be of the fine bubble or coarse bubble type. In the former, compressed air is released at or near the bottom of the aeration tank through porous tubes or plates made of aluminium oxide or silicon oxide grains cemented together in a ceramic matrix.

Air supplied to porous diffusers should contain less than 0.02 mg of dust per m^3. Troubles due to clogging from the inside can be reduced by providing air filters and those due to clogging from outside can be avoided by providing adequate air pressure below the diffusers at all times. In spite of such precautions, fine bubble diffusers will require periodical cleaning.

Coarse bubble aerators have slightly lower aeration efficiency than fine bubble aerators but are cheaper in first cost and are less liable to clogging and do not require filtration of air. Air diffusers are generally placed along one side of the aeration tank, helping to set up a spiral flow in the tank which improves mixing and prevents the solids from settling. They are located 0.3m to 0.6m above tank floor to aid in tank cleaning and reduce clogging during shutdown. The agitator-sparjer is a special mechanical aerator system involving the release of compressed air at the bottom of the aeration tank in large bubbles and the breaking up of the bubbles into fine bubbles by submerged turbine rotors located above the air outlets. The turbine rotors also provide mixing.

13.4.3.2 SURFACE AERATORS

Surface aerators were linked to small installations in the past but with recent improvements in their design, they are being increasingly used for large plants in preference to diffused air aeration systems. Some of their advantages are higher oxygen transfer capacity, absence of air piping and air filter and simplicity of operation and maintenance.

Surface aerators generally consist of large diameter impeller plates revolving on vertical shaft at the surface of the liquid with or without draft tubes. A hydraulic jump is created by the impellers at the surface causing air entrainment in the sewage. The impellers also induce mixing. The speed of rotation of the impellers is usually 70-100 rpm for geared motor systems.

The aeration rotors for small oxidation ditches are generally of cage type but may also be of the angle iron type. Particular attention must be paid to the design of shaft length, bearings and alignment. Vertical shaft aerators are easier to maintain and are used with deeper ditches.

13.4.3.3 MIXING REQUIREMENTS

The aeration equipment have also to provide adequate mixing in the aeration tank to keep the solids in suspension. Mixing considerations require that the minimum power input in activated sludge aeration tanks where MLSS is of the arder of 4000-5000 mg/l, should not be less than 15-26 w/m^3 of tank volume. The power input of aerators derived from oxygenation considerations should be checked to satisfy the mixing requirements and increased where required.

13.4.4 Measuring Devices

Devices should be installed for indicating flow rates of raw sewage or primary effluent, return sludge and air to each aeration tank. For plants designed for sewage flow of 10 mld or more, integrating flow recorders should be used.

13.4.5 Secondary Settling

Secondary settling assumes considerable importance in the activated sludge process as the efficient separation of the biological sludge is necessary not only for ensuring final effluent quality but also for return of adequate sludge to maintain the MLSS level in the aeration tank. The secondary settling tank of the activated sludge process is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. The high concentration of suspended solids in the effluent require that the solids loading rate should also be considered.

The recommended overflow rates and solids loading rates for secondary settling tanks of activated sludge have been given in Table 12.1.

13.4.6 Sludge Recycle

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

$$\frac{Q_R}{Q} = \frac{X}{X_s - X} \qquad (13.10)$$

Where

Q_R = Sludge recirculation rate, m^3/d.

The sludge settleability is determined by sludge volume index (SV1) defined as volume occupied in ml by one gram of solids in the mixed liquor after settling for 30 min and is determined experimentally. If it is assumed that sedimentation of suspended solids in the laboratory is similar to that in sedimentation tank, then $X_s = 10^6/SVI$. Values of SVI between 100 and 150 ml/g indicate good settling of suspended solids and can be achieved for values suggested in Fig.13.3.

The X_s value may not be taken more than 10,000 g/m^3 unless separate thickeners are provided to concentrate the settled solids or secondary sedimentation tank is designed to yield a higher value. Using the above value for X_s and 5000 mg/l for X in Eq.(13.10), the sludge recirculation ratio comes out to be 1.0. The return sludge is always to be pumped and the recirculation ratio should be limited to the values suggested in Table 13.1.

13.4.7 Excess Sludge Wasting

The sludge generated in the aeration tank has to be wasted to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with increasing F/M and decrease with increasing temperature.

The excess sludge generated under steady state operation may be estimated from Eq.(13.3) or (13.4).

In the case of domestic sewage, the excess sludge to be wasted will be about 0.35-0.5kg/kg BOD_5 removed for the conventional system and about 0.25-0.35 Kg/Kg BOD_5 removed in the case of extended aeration plants having no primary settling. The volume of sludge to be wasted will depend on the suspended solids concentration in the waste stream.

Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter procedure is to be preferred as the concentration of suspended solids will then be fairly steady in the waste stream providing better control on biomass wasted. The waste sludge is either discharged into the primary settling tank or thickened in a sludge thickening unit and digested directly. In extended aeration plants the excess sludge is taken to sludge drying beds directly and the sludge filtrate discharged into the effluent stream.

13.4.8 Nitrification

Activated sludge plants are ordinarily designed for the removal of only carbonaceous BOD. However, there may be incidental nitrification in the process. Nitrification will consume part of the oxygen supplied to the system and reduce the DO level in the aeration tank. Nitrification will also lead to subsequent denitrification in the secondary settling tank causing a rising sludge problem also called blanket rising '. Nitrification is aided by low F/M and long aeration time. It may be pronounced in extended aeration plants especially in hot weather. At the other extreme in the contact stabilisation process and in the modified aeration plant, there may be little or no nitrification.

Nitrification though generally not desired may be required in specific cases, e.g. when ammonia has to be eliminated from the effluent in the interest of pisciculture or when nitrification cum denitrification is proposed for elimination of nitrogenous matter from the effluent for control of eutrophication. In such cases, plug flow systems have been developed for efficient removal of both carbon and nitrogen. Alternatively a two stage system may be designed with carbonaceous BOD removal in the first stage and nitrification in the second stage.

13.4.9 Operation

The most important aspect in the operation of an activated sludge plant is the maintenance of proper F/M which is achieved by increasing or decreasing the MLSS levels in the aeration tank to suit the influent BOD_5 loads. The MLSS in the aeration tank can be regulated by controlling the rate of sludge return based on SVI determined experimentally. Excess sludge wasting is generally controlled based on experience.

The quick settleability of sludge is an important factor in the efficient performance of the activated sludge plant. The SVI serves also as an index of sludge settleability. SVI values of 80-150 are considered satisfactory.

Sludge with poor settling characteristics is termed bulking sludge. Sludge bulking results in poor effluent due to the presence of excessive suspended solids and also in rapid loss of MLSS from aeration tank. Sludge bulking is generally due to inadequate air supply resulting in low pH or septicity and growth of filamentous organisms.

Sludge bulking is controlled by eliminating the causes and by application of chlorine either to the sewage or to the return sludge to control filamentous growths. Chlorine requirements are 0.2 to 1.0 percent of dry solids weight in return sludge.

Occasionally, the secondary settling tank may function poorly even when the sludge volume index is satisfactory and sludge may rise up in the tank and escape with the effluent. Rising sludge may be due to denitrification in the settling tank releasing nitrogen bubbles which buoy up the sludge. The problem can be overcome by increasing the return sludge rate, increasing the speed of the sludge scraper mechanism and increasing the sludge wasting rate.

TABLE 13.1
CHARACTERISTICS AND DESIGN PARAMETERS OF ACTIVATED SLUDGE SYSTEMS FOR MUNICIPAL WASTEWATERS

Process Type	Flow regime	MLSS mg/l	MLVSS / MLSS	F/M $KgBOD_5$ / Kg MLSS Day	HRT, h	Θ_c, d	Q_R / Q	Eff %	Kg O_2/ KG BOD_5 removed
Conventional	Plug Flow	1500 - 3000	0.8	0.3 - 0.4	4 - 6	5 - 8	0.25 - 0.5	85 - 92	0.8 - 1.0
Completely mixed	Completely mixed	3000 - 4000	0.8	0.3 - 0.5	4 - 5	5 - 8	0.25 - 0.8	85 - 92	0.8 - 1.0
Extended aeration	Completely mixed	3000 - 5000	0.6	0.1 - 0.18	12 - 24	10 - 25	0.5 - 1.0	95 - 98	1.0 - 1.2

13.5 AERATED LAGOONS

Aerated lagoons are generally provided in the form of simple earthen basins with inlet at one end and outlet at the other to enable the wastewater to flow through while aeration is usually provided by mechanical means to stabilise the organic matter. The major difference between activated sludge systems and aerated lagoons is that in the latter settling tanks and sludge recirculation are absent.

Aerated lagoons are of two principal types depending on how the microbial mass of solids in the system is handled: Facultative Aerated Lagoons are those in which some solids may leave with the effluent stream and some settle down in the lagoon since aeration power input is just enough for oxygenation and not for keeping all solids in suspension. As the lower part of such lagoons may be anoxic or anaerobic while the upper layers are aerobic, the term facultative is used.

Aerobic Lagoons, on the other hand, are fully aerobic from top to bottom as the aeration power input is sufficiently high to keep all the solids in suspension besides meeting the oxygenation needs of the system. No settlement occurs in such lagoons and under equilibrium conditions the new (microbial) solids produced in the system equal the solids leaving the system. Thus, the solids concentration in the effluent is relatively high and some further treatment is generally provided after such lagoons. If the effluent is settled and the sludge recycled, the aerobic lagoon, in fact, becomes an activated sludge or extended aeration type lagoon.

A few typical characteristics of the above types of lagoons are given in Table (13.2) for ready reference.

Facultative type aerated lagoons have been more commonly used the world over because of their simplicity in operation and minimum need of machinery. They are often referred to simply as 'aerated lagoons'. Their original use came as a means of upgrading overloaded oxidation ponds in some countries without adding to the land requirement. In fact, much less land is required compared to oxidation ponds.

TABLE 13.2

SOME CHARACTERISTICS OF AERATED LAGOONS

Sl.No.	Characteristics	Facultative Aerated Lagoons	Fully Aerobic	Extended Aeration System (for comparsion)
1.	Detention time, days.	3 - 5	2 - 3	0.5 - 1.0
2.	Depth, m	2.5 - 5.0	2.5 - 4.0	2.5 - 4.0
3.	Land required, sq.m/person	0.15 - 0.30	0.10 - 0.20	-
4.	BOD removal efficiency %	80 - 90	50 - 60	95 - 98
5.	Overall BOD removal rate, K, per day 20° C (soluble only)	0.6 - 0.8	1 - 1.5	20 - 30
6.	Suspended solids (SS) in unit, mg/l	40 - 150	150 - 350	3000 - 5000
7.	VSS/SS	0.6	0.8	0.6
8.	Desirable power level watts/Cu.m. of lagoon volume.	0.75	2.75 - 6.0	15 - 18
9.	Power requirement, kWh/person/year	12 - 15	12 - 14	16 - 20

In earlier times the design of aerated lagoons was often done using simple thumb-rules of detention time and power per capita. But, over the years it has come to be recognised that lagoons being large bodies of water are subject to seasonal temperature effects and flow mixing conditions. Flow conditions in aerated lagoons are neither ideal complete-mixing nor ideal plug-flow in nature. They are dependent on lagoon geometry and are better described by dispersed flow models of the type given by Wehner and Wilhem for first-order kinetics and hence the design procedure given below takes treatability of the waste, temperature and mixing conditions into account.

Fully aerobic lagoons always have a complete-mixing regime and a slightly different mode of design is followed. However, as aerobic lagoons have not yet been built in India (except one case) further discussion is limited to facultative aerated lagoons only.

13.6 DESIGN VARIABLES

For facultative aerated lagoons, the dispersed flow model just referred to gives the relation between influent and effluent substrate concentrations. So and S, respectively and other variables such as the nature of the waste, the detention time and the mixing conditions, as shown in Equation.

$$\frac{S}{So} = \frac{4\, a.e^{1/2d}}{(1+a)^2 \; e^{a/2d} - (1-a)^2 \, e^{-a/2d}} \qquad (13.11)$$

in which the term $a = \sqrt{1+4K\theta.d}$

d = dispersion number (dimensionless)

$= D/UL = D.\Theta/L^2$

in which, D = Axial dispension coefficient (length^2/time)

L = Length of axial travel path

Θ = theoretical detention time. (Volume/Flow rate)

U = velocity of flow through lagoon (length/time)

K = Substrate removal rate in lagoon (time^{-1})

So & S = Initial and final substrate concentrations (mass/volume)

A graphical solution of the above equation is shown in Fig. 13.4 from which it is seen that prior knowledge of the substrate removal rate K as well as of the mixing condition likely to prevail in a lagoon is necessary to determine the efficiency of BOD removal at selected detention time. This is discussed further below.

13.6.1 Mixing Conditions

The mixing conditions in a lagoon are reflected by the term 'd' which is known as the "Dispersion Number" and equals (D / UL) or ($D\Theta$ / L^2). It is affected by various factors. Observed results have shown the (D / UL) values to be in the approximate range given in Table (13.3) for different length-width ratios of lagoons.

By suitable choice of a lagoon's geometry one can promote either more plug flow or more complete mixing type of conditions. Fig. 13.5 gives some examples of different types of arrangements using baffles or cells in series. In case of cells in series, each cell may be well mixed with value of D/UL approaching 3.0 or 4.0 but overall the arrangements would give a relatively plug-flow type arrangement. Values of D/UL can be determined by conducting dye (tracer) tests on existing units using well-known methods, but where D/UL values are required for design purposes prior to construction, they can be estimated either from lab-scale models or by using empirical equations available. Low values of D/UL signify plug flow conditions and generally give higher efficiencies of substrate removal whereas the converse is the case with higher values of D/UL. However, process efficiency is not the only consideration; process stability under fluctuating inflow quality and quantity conditions, has also to be kept in view. For municipal or domestic sewage, relatively plug flow type conditions (i.e.low values of D/UL) are preferred. In case of industrial wastes, relatively well mixed condition may be preferred (i.e.higher values D/UL) depending upon the nature of the industrial waste; the greater the fluctuations in quality and quantity of industrial wastes, the greater the advantage in adopting well-mixed conditions.

TABLE 13.3

LIKELY VALUES OF DISPERSION NUMBERS D/UL
AT DIFFERENT LENGTH-WIDTH RATIOS

Aerated Lagoon	Approximate range of D/UL values.	Typical mixing condition
Length-width ratio 1 : 1 to 4 : 1	3.0 to 4.0 and over	Well mixed
Length-width ratio 8 : 1 or more	0.2 - 0.6	Approaching plug flow
Two or Three cells in series	0.2 - 0.6 (overall)	-- do --

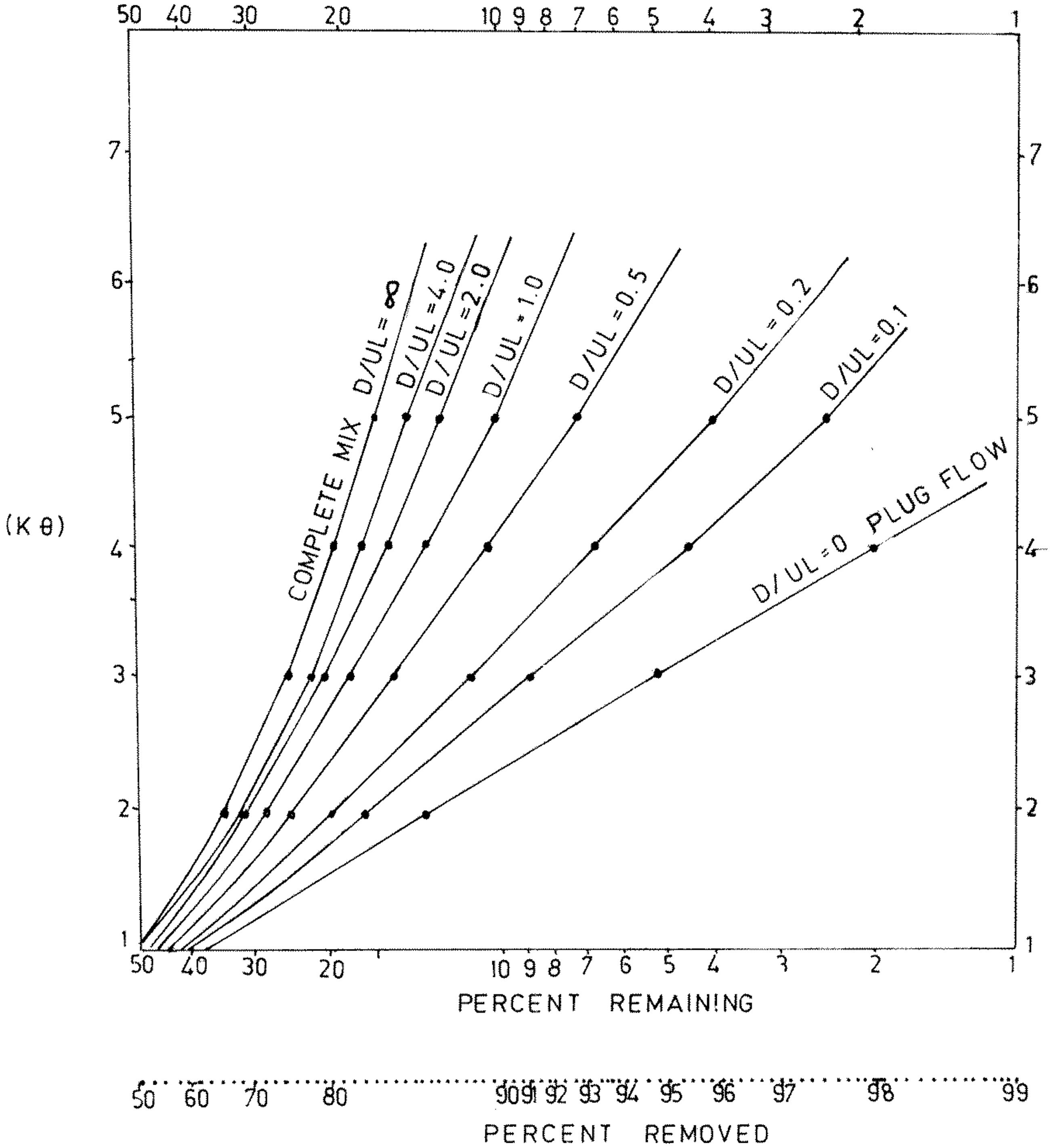
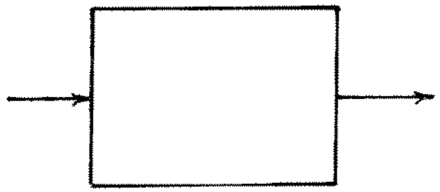
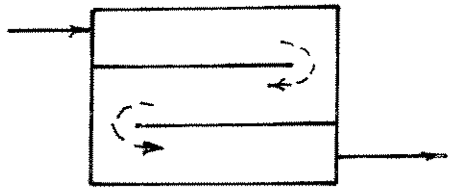


FIG.13.4 : SUBSTRATE REMOVAL EFFICIENCY USING THE DISPERSED FLOW MODEL (WEHNER-WILHEM EQUATION)



SQUARISH OR RECTANGULAR LAGOONS ARE LIKELY TO GIVE D/UL = 3.0 TO 4.0 OR MORE



BAFFLES COULD BE INTRODUCED IN THE LAGOONS TO GIVE A RELATIVELY LONG AND NARROW CHANNEL OF FLOW BRINGING DOWN D/UL = 0.2 TO 0.6

PROVIDING TWO OR THREE CELLS IN SERIES COULD ALSO GIVE AN OVERALL D/UL = 0.2 TO 0.6

FIG.13.5 :ESTIMATED EFFECT OF LAGOON GEOMETRY ON VALUE OF DISPERSION NUMBER D/UL