1 Characterisation and Degree of Treatment of Waste Water

1.0 INTRODUCTION

The ultimate disposal of waster water can only be onto the land or into the water. But whenever the water courses are used, for the ultimate disposal, the waste water is given a treatment to prevent any injury to the aquatic life in the receiving water. Normally the treatment consists of the removal of suspended and dissolved solids through different units of the treatment plant. Engineers can design a treatment plant to accomplish any degree of treatment. But a complete treatment or 100% removal of the pollution load is uneconomical and never aimed at in any waste treatment plant. The water courses can assimilate certain portion of the pollution load without affecting seriously the water quality and the environment. In fact, the treatment plant accomplishes a major part of the cleaning work, and then the nature, after the disposal of the effluent into the water courses, completes the work and thus rids the water of pollution.

Therefore, like any other Civil Engineering design, before proceeding with the design of the treatment plant itself, it is essential to determine:

(*i*) the characteristics of the raw waste water,

and (*ii*) the required characteristics of the treatment plant effluent, which will not pollute the receiving water course beyound certain acceptable limits. In most of the cases, however, the required characteristics of the effluents are established by the law of the local authority, instead of that by vigorous engineering analysis.

1.1 CHARACTERISTICS OF WASTE WATER

As mentioned above, the characterization of the raw waste is essential in the planning for effective and economical methods of water pollution control. Due to the varying nature of the industrial wastes, many of the recent installations have designed their treatment units with due consideration to the raw waste characteristics, and the effluent characteristics, as established by the Indian Standards Institution (ISI), State Pollution Control Boards, or by the local administrative authorities. But the characterization of the municipal waste water prior to a treatment plant design have not received the attention it deserves, probably because of its lower pollution potential compared to that of Industrial Wastes.

The characteristics of the municipal waste water vary from place to place, and depend on various factors like ecoomic status and food habits of the community, water supply position, and the weather condition of the locality. The characteristics of the waste from an Indian ciry may not be similar to that from a city in the USA.

Suspended Solids (SS) and 5 day 20°C BOD (BOD_5) are the usual parameters of pollution, particularly in domestic waste waters. As the pollution load varies according to the dilution offered by the water used, it is a convenient and common practice to express the SS and BOD_5 in terms of per capita contribution. However, with a greater amount of water use, more amount of solids per capita are expected to join the waste water flow, but it remains fairly constant beyond a waste flow rate of about 450 litres/capita/day.

The per capita contributions of SS, Total Solids, and BOD_5 in different Indian cities are shown in table 1.1. The composition of domestic waste water in some Indian cities are given in Table 1.2. The composition of different industrial wastes will be presented afterwards in the chapters dealing with the types of wastes under question.

In the absence of other information per capita SS and BOD_5 may be taken as 90 to 95 gms and 40 to 45 gms/day respectively in Indian cities. BOD associated with the suspended solids, in normal domestic waste water, is usually at a rate of 0.25 kg of BOD per kg of SS. The 1st order BOD removal rate constant may be much higher than the usually assumed value of 0.23 per day at 20°C.

TABLE 1.1. Per Capita Contributions to the Pollution in Different Indian Cities

Parameters Cities		BOD_5 gms/day	Total Solids gms/day	SS gms/day	BOD removal rate constant, per day	Flow, l/day
Housing Estate near Kolcutta		38-44	246-270	88-100	0.232-0.251	327
Bombay		32-64	—	—	0.281	122
Kanpur	*(i)*	70	—	64	—	580
	(ii)	32-39	—	—	—	—
	(iii)	58	—	198	0.345	490
Madras		27	—	—	—	—
Nagpur		18	—	—	—	—
Jamshedpur		27	—	—	—	—
Ahmedabad	*(i)*	27-45	—	—	—	—
	(ii)	63	—	70	—	340
Durgapur		45	—	—	—	—
Jaipur		35	—	35	—	81
UK & Germany average		54	—	—	— —	—
USA average		54	—	90	—	300

TABLE 1.2. Composition of Domestic Waste Water in Different Indian Cities

Parameters	Kolkatta	Delhi	Madras	Kanpur	Nagpur	Ahmedabad	USA Average
Temperature, °C	24	—	—	27	30	30	—
pH	7.0	7.4	7.4	7.0	7.0	—	—
Total solids, mg/l	785	—	—	—	—	—	—
Suspended solids, mg/l	287	354	530	560	292	206	295
Chlorides, mg/l	—	147	259	114	40	352	—
BOD_5 mg/l	125	203	352	255	334	186	180
COD, mg/l	290	377	—	532	—	—	—

TABLE 1.3. ISI Tolerance Limits for the Sewage and Industrial Effluents and that of Inland Surface Water.

Characteristics	Tolerance limits for sewage effluents discharged into inland sufrace water. IS : 4764-1973	Tolerance limits for industrial effluents discharged into		Tolerance limits for Inland surface water, when used as raw water for public water supplies and bathing ghats IS : 2296-1974
		Inland surface water IS: 2490-1974	Public sewers IS: 3306-1974	
1	2	3	4	5
BOD (5 day 20°C), mg/1	20	30	500	3
COD, mg/1	—	250	—	—
pH	—	5.5-9.0	5.5-9.0	6.0-9.0
Total suspended solids, mg/1	30	100	600	—
Temperature, °C	—	40	45	—
Oil and Grease, mg/1	—	10	100	0.1
Phenolic compounds, mg/1	—	1.0	5	0.005
Cyanides (as CN), mg/1	—	0.2	2.0	0.01
Sulphides (as S), mg/1	—	2.0	—	—
Fluorides (as F), mg/1	—	2.0	—	1.5

(Contd.)

TABLE 1.3. (*Contd.*)

1	2	3	4	5
Total residual cholorine, mg/1	—	1.0	—	—
Insecticides, mg/1	—	zero	—	zero
Arsenic (as AS), mg/1	—	0.2	—	0.2
Cadmium (as Cd), mg/1	—	2.0	—	—
Chromium, hexavalent (as Cr), mg/1	—	0.1	2.0	0.05 (Total chromium)
Copper, mg/1	—	3.0	3.0	—
Lead, mg/1	—	0.1	1.0	0.1
Mercury, mg/1	—	0.01	—	—
Nickel, mg/1	—	3.0	2	—
Selenium, mg/1	—	0.05	—	0.05
Zinc, mg/1	—	5.0	15.0	—
Chloride (as Cl), mg/1	—	—	600	600
Sulphates, mg/1	—	—	—	1000
% Sodium	—	—	60	—
Ammoniacal Nitrogen, mg/1	—	50	50	—

(*Contd.*)

TABLE 1.3. (*Contd.*)

1	2	3	4	5
Nitrates (as No_3), mg/l	—	—	—	50
Radioactive materials:				
α-emitters, μc/ml—	10^{-7}	—	10^{-9}	
β-emitters, μc/ml—	10^{-6}	—	10–8	
Dissolved Oxygen, mg/l	—	—	—	40% of the saturation value, or 3 mg/l, which ever is higher.
Coliform organism—(monthly average)—MPN per 100 ml	—	—	—	Should not exceed 5000. (should not exceed 20000 with less than 5% samples, and 5000 with less than 20% samples).

Note: Tables 1.3 and 16.1 of this book have been reproduced with the permission of ISI, from Indian Standard nos. 2296-1974, 2490 (pt 1)-1974, 3306-1974, 4764-1973, and 4903-1968, to which reference is invited for further details. These standards are available for sale from Indian Standards Institution, New Delhi and its Regional and Branch Offices at Ahmedabad, Bangalore, Bhubaneswar, Bombay, Kolkatta, Chandigarh, Hyderabad, Jaipur, Kanpur, Madras, Patna, and Trivandrum.

1.2 CHARACTERISTICS OF THE TREATMENT PLANT EFLUENTS

The required quality of the treatment plant effluents is solely dictated by the quality requirements of the receiving water. As stated earlier the quality of the receiving water may be established either by law or by a vigorous engineering analysis giving due consideration to the natural purification or self-purification that occurs in the receiving water. Effluent standards as well as river water qualities, as recommended by I.S.I. are given in Table 1.3. Once the quality required to be maintained in the receiving water at a particular cross-section is established (either at the point of discharge, or at a point downstream to the point of discharge), the quality required of the effluent being discharged can be computed.

The natural purification is a slow process, and depends on various factors. In the following sections, some of the significant forces of natural purification will be discussed; it will be then demonstrated how the knowledge of the same can be utilized in establishing the required quality of the effluent, and thus the degree of treatment of the treatment plants.

1.3 PATTERN OF POLLUTION AND SELF-PURIFICATION IN A STREAM

When pollutants are discharged into a stream, a succession of changes in water quality takes place, in the down stream side of the point of pollution. The resulting pattern of change along the stream establishes a well defined profile of pollution and self purification, which again changes with seasons and hydrography.

Whenever a single, heavy charge of putrescible organic matter is added into a clean stream, depending on the hydrography of the stream, the suspended matter is either settled at the bed near the point of discharge, or is carried along with the water to the downstream side. If the wetted surface of the river bed is sufficiently large, a major portion of the organic load is also removed from the main stream by adsorption. At the same time, the aerobic micro-organisms, which utilize the organic pollutants as the source of their food and energy, grow till the food supply is adequate for them, and thus the organic matter is stabilized under aerobic condition. The removal of organics are accomplished by (*i*) settling and adsorption, and by (*ii*) microbiological activities.

The intensity of the life activities of the micro-organisms is reflected by the biochemical oxygen-demand (BOD). Due to the microbial activities, the oxygen resources of the water are heavily drawn upon; in an overloaded stream, the dissolved oxygen (DO) may be completely exhausted due to these activities.

In course of time and flow, the food supply gets exhausted. The life activities of the microbial population come to an end; and as such the BOD is decreased. The rate of reaeration, or the absorption of oxygen from the atmosphere, which at first has lagged behind the rate of oxygen consumption by the micro-organisms, assumes a momentum and very soon takes the lead. The water becomes clear and the stream returns to its original condition. The self purification is thus complete.

As stated earlier, the self-purification is a very slow process. A heavily polluted stream may have to traverse quite a long distance for many days for the attainment of a significant degree of purification. It may also be noted that, beside the factor noted earlier, many other natural forces play an important role in the natural purification either in favour or against the process. These will be discussed briefly in the subsequent sections.

It should be recognized that, the natural forces of purification enter in one way or the other into the artificial treatment processes, where they are purposely intensified in order to accomplish the desired degree of treatment in a verty short time and small space.

1.3.1 Parameters of Pollution and Self-purification

The extent or degree of pollution and that of self-purification can be measured in various ways. The method of measurement depends on the nature of the pollution and the subsequent uses of the receiving water. When the nuisance potential of the discharged effluent is of great concern, the BOD and DO of the receiving water, taken together, are measured to trace the profile of pollution and self-purification in the stream. The BOD represents the intensity of biodegradable matter remaining in the stream at any time, and the DO shows the ability of the stream to purify itself through the biochemical processes. Other parameters of pollution include pH, Suspended solids, and Toxic substances, and will be dealt with, briefly afterwards. It may however be noted that the BOD and DO taken together, and the other parameters mentioned above, provide no information on the state of eutrophication and the possibility of algal bloom. Conditions sometimes demand the removal of inorganic nutrients in addtion to organic materials. As such, the nitrogenous nutrients may also be considered as the parameter of pollution.

1.4 DISSOLVED OXYGEN

Organic waste normally undergoes aerobic decomposition in the stream. Only when the rate of supply of oxygen (mainly from atmosphere) cannot keep pace with the rate of oxygen demand, the condition within the stream becomes anaerobic. The anaerobic condition is however not desirable, as while the aerobic receiving water look reasonably clean and are free from odour, the anaerobic condition makes it black, unsightly and malodourous. The biochemical reactions within the stream exert BOD, resulting in the deoxygenation of the stream.

Apart from the above, a certain portion of the biodegradable organics get deposited at the bed of the stream. They undergo anaerobic and benthic decomposition. The products of such decomposition are organic acids and reduced gases. These are further stabilized by the aerobic microorganisms in the upper layer, thereby increasing the BOD of the stream. A small amount of oxygen is also utilized by the higher animals for their respiratios.

But, as stated earlier, the simultaneous replenishment of oxygen in the stream occurs due to the absorption of oxygen from the atmosphere and also due to the release of oxygen by the green plants during photosynthesis. This is known as reaeration or reoxygenation of the stream. The interplay between the deoxygenation and reaeration produces a well defined profile of the dissolved oxygen in the stream as shown in Fig. 1.1.

The simplest way of estimating the required effluent quality, however, merely takes into account the available DO of the stream and ignore the amount of oxygen available through reaeration. The

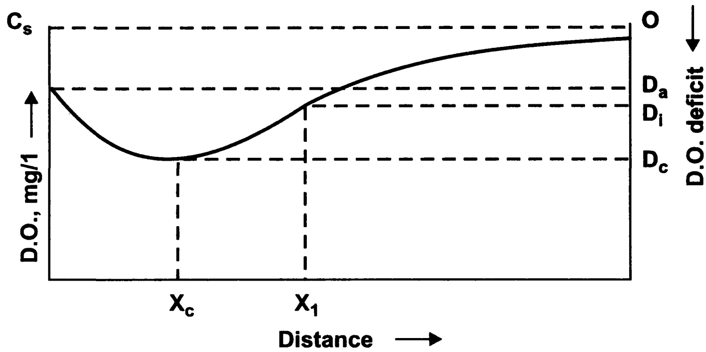

Figure 1.1: Oxygen Sag Curve.

permissible effluent BOD is calculated using the following simple relationship.

$$Q.\ DO_r - L.q = DO_m\ (Q + q) \qquad \ldots (1.1)$$

in which Q = rate of flow of the stream,

q = rate of waste flow,

DO_r = DO of the stream, upstream to the point of discharge,

L = Permissible BOD_L of the waste water,

and DO_m = Permissible minimum DO of the stream, downstream to the point of discharge.

The more accurate and rational method of estimating the maximum permissible BOD load requires the knowledge of the DO profile. The forecasting of a DO profile downstream to a point of discharge again requires thorough knowledge of different natural forces in deoxygenation and reaeration.

1.4.1 Deoxygenation Through Exertion of BOD

The instantaneous rate of deoxygenation at any time t is given by the following 1st order equation.

$$\frac{dC}{dt} = K_2\ (C_3 - C) = K_2 D \qquad \ldots (1.2)$$

in which C = concentration of DO at any time t, mg/1,
L = ultimate BOD of the polluted water, remaining after time t, mg/1,
and K_1 = deoxygenation constant, per day.

The Eqn. 1.2 is identical to the 1st order BOD removal rate equation

$$\frac{dL}{dt} = -K_1L \quad \text{... (1.3)}$$

which is valid when the BOD removal is mediated only through the microbiological activities.

When the BOD removal is also accomplished by physical forces like sedimentation and adsorption, the Eqn. 1.3 is modified to the following form:

$$\frac{dL}{dt} = -K_1L - K_3L \quad \text{... (1.4)}$$

in which K_3 = rate constant for BOD removal by sedimentation and adsorption, per day.

The deoxygenation constant, K_1, varies with temperature as per the following relationship

$$K_1 = (K_1)_{20} \cdot \left(\theta_{K_1}\right)^{T-20} \quad \text{... (1.5)}$$

in which $(K_1)_{20}$ = deoxygenation constant at a temperature of 20°C,
K_1 = deoxygenation constant of a temperture of T°C,
and θ_{K_1} = a temperature coefficient = usually 1.047 for K_1 in the temperature range of 15° to 30°C.

The value of K_1 may vary from 0.12 per day for treated waste effluent to 0.39 per day for a strong waste.

The ultimate BOD, L, was previously assumed to vary with temperature, and equations similar to the Eqn. 1.5 were also derived for the temperature variation of the ultimate BOD of the wastes.

Later work suggests that no such variation should occur with temperature.

A small fraction of the BOD is removed by settling and adsorption; if the condtions favour, the settled particulate wastes under-go anaerobic digestion. As stated earlier, the products of the anaerobic digestion, *viz.*, organic acids, gases etc. are utilised as the source of food and energy by the aerobic microorganisms. So the products of digestion of settled particles become part of the BOD of the stream. Apart from this, certain amount of BOD is also added to the stream by the local run off. The rate at which the BOD is added to the stream may be given by the following relationship:

$$\frac{dL}{dt} = L_b \qquad \text{... (1.6)}$$

in which L_b = rate of addition of BOD by local run off and by the products of anaerobic digestion of bottom sludge deposits.

1.4.2 Reaeration of the Polluted Streams

The rate at which water absorbs oxygen from the atmosphere can be given by the following equation:

$$\frac{dC}{dt} = K_2 (C_3 - C) = K_2 D \qquad \text{... (1.7)}$$

in which D = DO deficit after time t, mg/1,
K_2 = reaeration coefficient, per day,
and C_3 = DO saturation concentration.

The magnitude of K_2 is dependent on various factors like the surface exposure, volume of water flow and the turbulance of the stream. Isaac and Gaudy have given the following dimensionally homogeneus equation for the theoretical value of K_2:

$$K_2 = 2.3\ k_2 = 2.3\ c\ \frac{D_m^{\frac{1}{2}}}{V^{1/6} g^{1/6}} \cdot \frac{v}{H^{3/2}} \qquad \text{... (1.8)}$$

in which K_2 = reaeration coefficient (expressed in $\log_e$),
k_2 = reaeration coefficient (expressed in $\log_{10}$),
c = proportionality constant (dimensonless),
D_m = molecular diffusion coefficient for oxygen in water equal to 2.037×10^{-9} m^2/sec. at 20°C (it varies with temperature and the temperature coefficient is usually taken as 1.037),
v = kinematic viscosity of water,
g = gravitational constant,
V = mean velocity of flow,
and H = average depth of flow.

In a simulated stream the value of c was found to be equal to 0.06339. Natural stream data in a particular case however indicates that c may be taken as 0.07762, which takes the irregular channel geometry into account.

By substituting the known values of D_m, v and g, and the assumed value of c (equal to 0.07762) in Eqn. 1.8, the expression for K_2 at 20°C may be given by the following expression:

$$(K_2)_{20} = 4.75 \ V/(H)^{\frac{3}{2}} \qquad \ldots (1.9)$$

in which $(K_2)_{20}$ = reaeration coefficient at 20°C, per day,
V = mean velocity in m/sec.,
and H = average depth of the stream, m.

The Eqn. 1.8 or 1.9 should only be used in the absence of any actual test result. The value of K_2 not only varies from stream to stream, but also varies along the length of a particular stream. The variation of K_2 with temperature can be given by the following relationship:

$$K_2 = (K_2)_{20} \ \theta \ K_2 \ ^{(T-20)} \qquad \ldots (1.10)$$

in which K_2 = reaeration coefficient at T°C,
$(K_2)_{20}$ = reaeration coefficient at 20°C,
and θ_{K_2} = reaeration temperature coefficient, usually 1.0241.

Howe, however, proposed the following modified relationship for the temperature variation of K_2:

$$K_2 = \frac{(K_2)_{20}}{(\theta_{K_2})^{T-20}} \qquad \text{... (1.11)}$$

in which θ_{K_2} is 1.0125.

In addition to the reareation by absorption of atmospheric oxygen, a small quantity of oxygen is introduced to the system by phyotosynthesis, and another portion is lost due to the respiration of aquatic plants and animals. It is convenient to combine the photosynthesis, respiration, and any other source or sink of oxygen in the stream, and express the rate of change of oxygen by the following relationship:

$$\frac{dC}{dt} = \pm S_r \qquad \text{... (1.12)}$$

in which S_r = rate of change of dissolved oxygen concentration as a result of physical and biological forces, not included in Eqns. 1.2 and 1.7.

1.4.3 BOD and DO Profile

The rate of change of DO concentration and BOD along a stretch of a polluted stream, can be given by the following two one-dimensional DO-BOD equations, obtained by writing the material balance across an element of the stream water:

$$\frac{\partial C}{\partial t} + V\frac{\partial C}{\partial x} = \frac{1}{A}\frac{\partial}{\partial x}\left(AE\frac{\partial C}{\partial x}\right) - K_1L + K_2D \pm S_r \qquad \text{... (1.13)}$$

and,

$$\frac{\partial L}{\partial t} + V\frac{\partial L}{\partial x} = \frac{1}{A}\frac{\partial}{\partial x}\left(AE\frac{\partial L}{\partial x}\right) - (K_1 + K_3)L \pm L_b \qquad \text{... (1.14)}$$

in which, E = dispersion coefficient, usually m^2/day ($L^2\, t^{-1}$).

It may be noted that, the second term in the L.H.S. of Eqns. 1.13 and 1.14, $V(\partial L/\partial x)$, represents the advective component of the flux across the cross section. The first term of the R.H.S. of the Eqns. 1.13 and 1.14 represents the longitudinal dispersion component of the flux. The remaining terms in the R.H.S. represent the sources and sinks, as discussed earlier in Sections 1.4.1 and 1.4.2.

However, in a stream the dispersion may be neglected without any serious error. And also it is convenient to express DO concentrations in terms of "DO saturation deficit", D. Again in a steady state condition, $\partial C/\partial t = 0$. Therefore, in a steady state condtion, and neglecting all sorts of longitudinal dispersion, the Eqns. 1.13 and 1.14 reduce to:

$$V\frac{dD}{dx} = -K_1L + K_2D \pm S_r \qquad \text{... (1.15)}$$

and

$$V\frac{dL}{dx} = -(K_1 + K_3)L + L_b \qquad \text{... (1.16)}$$

The solution of the Eqns. 1.15 and 1.16 yields the following relationship for the DO deficit at any distance x from the point of discharge:

$$D(x) = D_aF_2 + \frac{K_1}{K_2 - (K_1 + K_3)}\left[L_a - \frac{L_b}{K_1 + K_3}\right](F_1 + F_2)$$

$$+\left[\frac{S_R}{K_2} + \frac{K_1L_b}{K_2(K_1 + K_3)}\right](1 - F_2) \qquad \text{... (1.17)}$$

in which, D_a = initial DO deficit, or D at $x = 0$,

$$F_2 = \exp\left(-K_2\frac{x}{V}\right), \qquad \text{... (1.18)}$$

$$F_1 = \exp\left[-(K_1 + K_3)\frac{x}{V}\right], \qquad \text{... (1.19)}$$

L_a = initial BOD_L, or L at $x = 0$,

and V = Velocity of flow = Q/A.

A typical plot of the Eqn. 1.16 is shown in Fig. 1.1. The spoon shaped DO profile thus obtained is known as Oxygen Sag curve.

The sag curve possesses two characteristic points:

(*i*) the critical point at a distance x_c, where the maximum DO deficit, D_c, occurs,

(*ii*) the point of inflection at a distance x_i,

where the rate of recovery is maximum.

By differentiation of Eqn. 1.17, and solving for x after equating the derivative to zero, the critical distance x_c may be obtained as follows:

$$x_c = \frac{V}{K_2 - (K_1 + K_3)} \ln \left[\frac{K_2}{K_1 + K_3} + \frac{K_2 - (K_1 + K_3)}{(K_1 + K_3) L_a - L_b} \cdot \left\{ \frac{L_b}{K_1 + K_3} - \frac{K_2 D_a - S_r}{K_1} \right\} \right] \quad \ldots (1.20)$$

To facilitate the analysis, as will be demonstrated in a following section, the Eqn. 1.17 is rearranged in the following form to give the initial 1st stage BOD, L_a:

$$L_a = \frac{D - F_3(1 - F_2) - D_a F_2}{F_4 \, (F_1 - F_2)} + \frac{L_b}{K_1 + K_3}, \quad \ldots (1.21)$$

in which $$F_3 = \frac{S_r}{K_2} + \frac{K_1 L_b}{K_2 \, (K_1 + K_2)} \quad \ldots (1.22)$$

and $$F_4 = \frac{K_1}{K_2 - (K_1 + K_3)} \quad \ldots (1.23)$$

It may be noted that the values of F_3 and F_4 remain constant for a particular stretch of the stream.

It may however be noted that in many cases, K_3, L_b and S_r are negligible, and if neglected in the analysis, the Eqn. 1.17 reduces to the following form of well known Streeter and Phelps formula:

$$D = \frac{K_1 L_a}{K_2 - K_1}\left[\exp\left(-K_1 \frac{x}{V}\right) - \exp\left(-K_2 \frac{x}{V}\right)\right]$$

$$+ D_a \exp\left(-K_2 \frac{x}{V}\right) \qquad \text{... (1.24)}$$

In such cases, x_c and D_c are given by the following two relationships

$$x_c \frac{V}{K_1 (f - 1)} \ln\left[f\left\{1 - (f - 1)\frac{D_a}{L_a}\right\}\right] \qquad \text{... (1.25a)}$$

and $$D_c = \frac{L_a e^{-K_1 t_c}}{f}, \qquad \text{... (1.26)}$$

in which, $$f = \frac{K_2}{K_1} = \text{self-purification ratio} \qquad \text{... (1.27)}$$

Rearrangement of Eqn. 1.25a yields:

$$\left(\frac{L_a}{D_c f}\right)^{f - 1} = f\left[1 - (f - 1)\frac{D_a}{L_a}\right] \qquad \text{... (1.25)}$$

Eqn. 1.25 may be solved for L_a by trial and error methods.

1.4.4. Determination of K_1, K_2, K_3, L_b and S_r

In fact, K_1 is a reaeration rate constant of a biological process, and is to be determined in laboratory, testing samples from the stretch of the river under similar environmental conditions.

Once the value of K_1 is determined, the values of K_3 and L_b can be determined using the following relationship for BOD_L at any time x/V.

$$L_x = L_a \exp\left[-K_1\left(\frac{x}{V}\right)\right] \qquad \text{.... (1.28)}$$

in which, L_x = estimated theoretical value of BOD_L at a distance x from the point of discharge.

Now if L_x is greater than the actual value of L, it may be assumed that $L_b = 0$, and K_3 may be estimated by using the following relationship, which is obtained through integration of Eqn. 1.16, and substituting $L_b = 0$.

$$L = L_a \exp\left[-(K_1 + K_3)\frac{x}{V}\right] \qquad \text{.... (1.29)}$$

When L_x is less than the actual value of L, it is assumed that $K_3 = 0$, and the value of L_b is calculated using the following relationship, obtained similarly from Eqn. 1.16 but substituting $K_3 = 0$.

$$L = L_a \exp(-K_1 x/V) + (L_b/K_1)\,[1 - \exp(-K_1 x/V)] \qquad \text{... (1.30)}$$

If L_x is equal to the actual value of L, both K_3 and L_b may be assumed to be equal to zero.

K_2 can best be determined by the Eqns. 1.8, 1.9 and 1.10. The remaining unknown S_r may now be calculated using Eqn. 1.17. S_r may be either positive or negative.

1.4.5 Allowable BOD Loading on Stream, L_a

To avoid anaerobic decomposition of the organic wastes, and also to provide adequate support to the aquatic life, the DO is not allowed to fall below about 3 mg/1. This information fixes the allowable maximum DO saturation deficit, D_{max}, in the stream.

For a given set of values of K_1, K_2, K_3, L_b, S_r, D_{max} and D_a, the maximum allowable BOD loading L_a can now be determined by an iterative process. The maximum allowable ultimate BOD of the treatment plant effluent may then be calculated from the stream flow and waste discharge data, assuming a complete mixing condition at the point of discharge.

The following procedure may be adopted for the estimation of allowable L_a:

(*i*) L_a is determined using the Eqn. 1.21 substituting $D = D_{max}$ and the maximum length of the stretch of the stream, x_{max}, as the value of x.

(*ii*) Using the value of L_a thus determined, the value of x_c is determined using the Eqn. 1.20.

The calculated value of x_c may be greater or less than the assumed value of $x = x_{max}$.

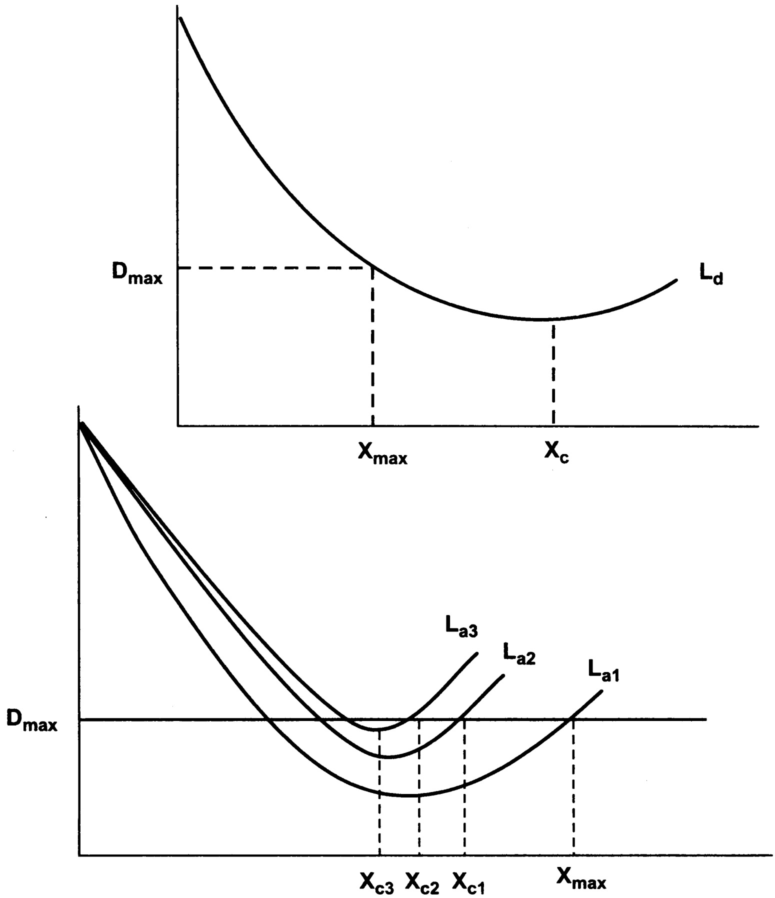

Figure 1.2: Definition Sketch of the Procedure Outlined in Section 1.4.5. (Loucks' Procedure).

(*iii*) if x_{max} is less than the value of x_c, calculated in step-*ii*, the estimated value of L_a in step-*i*, is not likely to result in a DO deficit greater than D_{max} within the stretch of the stream. And this L_a value may be accepted as a design value. [Fig. 1.2(*a*)].

(*iv*) But if x_{max} is greater than the value of x_c, calculated in step-*ii*, the calculated value of L_a in step-*i*, is likely to deplete the DO of the stream beyond the allowable limit. In such cases, following iterative procedure is to be adopted for the estimation of allowable L_a.

(*v*) Using the value of x_c determined in the step-*ii*, a fresh value of L_a is determined, using Eqn. 1.21.

(*vi*) A fresh value of x_c is then determined with the new value of L_a.

(*vii*) The procedure in steps *v* and *vi* are repeated until a reasonable agreement is achieved between the successive values of x_c [Fig. 1.2 (*b*)].

Once the maximum allowable BOD, L_a, of the stream is determined, the maximum allowable BOD of the waste, L_w, may be determined from the following BOD balance, assuming a complete mix condition at the point of discharge:

$$L_a (Q + q) L_s Q + L_w q \quad .... \quad (1.31)$$

In which Q = rate of flow in the stream,

q = rate of discharge of the waste,

L_s = BOD_L of the stream, upstream to the point of discharge.

L_w = BOD_L of the waste,

and L_a = BOD_L of the stream, at the point of discharge.

1.5 pH VALUES OF WASTES AND RECEIVING WATER

In domestic waste water, the pH values normally vary from 7.0 to 7.5, and therefore do not affect the receiving water. So in determining the degree of treatment of a domestic waste, the pH of the waste and that of the receiving water need not be considered. But a completely different picture may arise in the case of an industrial waste. Most of the industrial wastes are characteristically either acidic, or alkaline. Acidic wastes are corrosive to both metallic and concrete structures in the water courses, and are also toxic to the aquatic life. Moreover, when free acid reacts with the natural alkalinity of the water, it produces carbonate hardness, thus rendering it unfit for future laundry uses, or as boiler feed water.

Alkaline wastes, on the other hand, when discharged into the water courses, combine with the free carbon dioxide and further increase the alkalinity of the water. The degree of treatment of the acidic or alkaline waste is determined according to the pH of the receiving water after mixing, which can be estimated using the following approximate relationship.

For acidic wastes:

$$pH = 6.52 + \log\left(\frac{B-X}{50}\right) - \log\left(\frac{C}{22} - \frac{X}{50}\right) \qquad \text{.... (1.32)}$$

and for alkaline waste:

$$pH = 6.52 + \log\left(\frac{B+X}{50}\right) - \log\left(\frac{C}{22} - \frac{X}{50}\right) \qquad \text{.... (1.33)}$$

in which, B = alkalinity of water upstream to the point of discharge, in mg/1 as $CaCO_3$.

X = concentration of acidic or alkaline waste after dilution with the water, in mg/1 as $CaCO_3$.

and C = concentration of free CO_2 in mg/1.

The optimum range of pH for aquatic life is 6.8 to 9.0. Therefore, by substituting the limiting values of the pH (6.8 for acidic waste, and 9.0 for alkaline waste) in the Eqn. 1.32 or Eqn. 1.33, the maximum permissible value of X can be estimated. The permissible maximum concentration of the acidic or alkaline waste then can be determined, from an acid/alkali balance in the stream. It may however be noted that the Eqns. 1.32 and 1.33 are applicable for inorganic acids/alkalis only.

1.6 SUSPENDED SOLIDS

Organic suspended and dissolved solids undergo biodegradation and their pollution potentials are usually expressed in terms of BOD. The effects of these organic solids on the receiving water have been described earlier in Section 1.4. However, the fixed or inorganic solids and heavier organic solids settle quickly and form a sludge blanket near the point of discharge. The maximum concentration of the suspended solids (SS) in the receiving streams are often specified by the local authorities. The permissible concentration of SS in the waste effluent may be estimated as earlier, using the SS balance and neglecting the longitudinal dispersion; such a method assumes a complete mix condition at the point of discharge, which is far from the fact. For a more accurate method, the following relationship may be used:

$$S_w = (S_m - S_r)\left(a\frac{Q}{q} + 1\right) + S_r \qquad \text{.... (1.34)}$$

in which, S_w = permissible SS in the waste effluent, mg/1,
S_m = permissible SS in the stream, at a particular cross-section, downstream to the point of discharge, mg/1.
S_r = SS in the stream, upstream to the point of discharge, mg/1,
a = degree of mixing, or, mixing coefficient for the said particular cross-section,
Q = rate of flow of receiving water,
and q = rate of discharge of the waste water.

The degree of mixing, a, in the Eqn. 1.35, is a function of flow rates, bed tortuosity, eddy diffusion, distance of the point of complete mixing (where a = 1) from the point of discharge, etc. Theoretically the mixing is complete (and a = 1) at a distance of infinity from the point of discharge. In practice it is convenient to assume the mixing to be sufficiently complete at a point where a is equal to 0.90 to 0.95. The distance of the cross-section of the polluted stream, l, where the degree of mixing equals to a particular value of a, may be estimated using the following relationship:

$$l = \left[\frac{2.3}{a}\log\frac{aQ+q}{(1-a)q}\right]^3 \qquad \text{.... (1.35)}$$

If the permissible concentration of SS in the receiving stream, $S_{m,}$ is specified for a particular cross-section, at a distance of l

from the point of discharge, the degree of mixing, *a*, can be estimated by using the Eqn. 1.35; the substitution of this value of *a* in the Eqn. 1.34 will result in the permissible concentration of SS in the waste, S_w.

For all practical purposes, however, it may be assumed that the specified maximum permissible SS in the stream refers to that at the point of complete mixing. As such, the permissible SS in the waste effluent may be directly estimated using the Eqn. 1.34, assuming *a* equal to 0.90 or 0.95.

1.7 TOXIC SUBSTANCES

The toxic substances present in the industrial wastes not only disturb the ecological balance in the receiving water, but also may endanger the health of the animals and human beings who may have to use the polluted water for drinking purposes. The tolerance limits of the different toxic elements in the receiving water have been prescribed by different authorities. The permissible concentration of the toxic elements in the waste effluent may then be computed from the following relationship, obtained from the mass balance:

$$T_w = T_m\left(a\frac{Q}{q}+1\right) \qquad \text{.... (1.36)}$$

in which, T_w = allowable concentration of the toxic element in the waste effluent,

and T_m = permissible concentration of the toxic element in the receiving water, downstream to the point of discharge.

The permissible concentration of the toxic elements in the receiving water, T_m, refers to that at the point of complete mixing. In order to limit the concentration of toxic elements in the stream to the permissible value at any point in between the point of outfall and the point of complete mixing, the mixing coefficient, *a*, should be taken into considerations. In all other cases the value of *a* may be taken as unity.

When sufficient information about the individual toxicants, in regard to their concentration in the waste water and their allowable limits in the receiving water, are not available, the acute toxicity of the entire waste water is measured by means of bioassy tests, and are expressed in terms of "median tolerance limit", TL_m. The TL_m is defined as the concentration of waste in which just 50% of the test animals are able to survive for a specified period of exposure, usually 48–96 hours. Fish are usually used as test animals, and normally it is expected that a concentration of 1/10th of the 96-hr TL_m value will not cause any serious effect on the aquatic lives.

From the definitionz of TL_m, the water required for the dilution of the toxic waste, to reduce its toxicity to a value to cause 50% mortality among the test animals, is given by:

$$TL_m = \frac{q}{V+q} \times 100 \qquad \text{.... (1.37)}$$

in which, TL_m = median tolerance limit in percentage,
q = waste volume,
and V = dilution water volume.

The Eqn. 1.37 may be rewritten in the following form:

$$V = \frac{(100 - TL_m)}{TL_m} q \qquad \text{.... (1.38)}$$

In the stream pollution control, the stream flow necessary to dilute the waste sufficiently to a safe level is determined by multiplying the above dilution water volume by an application Factor, F_a, usually 10.

$$Q = V.F_a \qquad \text{.... (1.39)}$$

in which, Q = stream of flow rate,
F_a = application factor,
and V = dilution water flow rate.

1.8 ESTUARINE POLLUTION

When a certain amount of pollution is introduced continuously into a tidal stream, the tidal oscillation moves the polluted volume of water back and forth, and thereby causes a widely fluctuating water quality in the estuary. The prediction of water quality in terms of DO and BOD, or Nitrogenous nutrients, is based on the dynamic coupling between the hydrodynamic transport processes and the biochemical water quality change processes.

The biochemical processes which lead to the changes in the water quality in a stream has been discussed in the previous sections. The hydrodynamic transport processes include advection, mixing, and dispersion of the pollutants, as in a fresh water stream. But the problem becomes much more complicated in an estuary due to the longitudinal density gradient and the variation of the instantaneous longitudinal velocity of flow due to the tidal effect. Analysis of an estuary becomes further complicated, if the same is stratified due to the sea water intrusion near the bay. However, many dynamic estuaries, like Hooghly river, are vertically homogeneous. Where the estuary presents complex problems because of wide fluctuation of physico-chemical conditions within it, daily, seasonally and geographically, actual field studies are to be conducted, instead of depending on the mathematical models.

1.8.1 BOD and Do Profiles in An Estuary

The dynamic BOD-DO model in an estuary of varying and arbitrary cross-section consists of the following two equations, similar to Eqns. 1.13 and 1.14:

$$\frac{\partial C}{\partial t} + V_{xt}\frac{\partial C}{\partial x} = \frac{1}{A}\frac{\partial}{\partial x}\left(AE\frac{\partial C}{\partial x}\right) - K_1 L + K_2 D \pm S_r, \ldots \quad (1.40)$$

and,

$$\frac{\partial L}{\partial t} + V_{xt}\frac{\partial L}{\partial x} = \frac{1}{A}\frac{\partial x}{\partial x}\left(AE\frac{\partial L}{\partial x}\right) - (K_1 + K_3)L + L_b \quad \ldots . \ (1.41)$$

in which, V_{xt} = instantaneous longitudinal velocity of flow, averaged over the cross-section, including tidal and fresh water components.

Main difficulty in the simultaneous solution of the Eqns. 1.40 and 1.41 arises from the unsteady hydraulic regime of the estuary. Moreover, all the terms in the above equations may vary with distance and time. Analytical solutions to Eqns. 1.40 and 1.41, as such, cannot be obtained. The available analytical solutions of the above two equations are based on many simplifying assumptions.

However, by means of numerical analysis method, using digital computers, it is possible to predict approximately the BOD and DO at any distance and time in the estuary for a given set of specific input data.

If L_a is the value of L at $x = 0$, for all general cases, L_a is usually given by:

$$L_a = \frac{W}{A\sqrt{V_{xt} + 4K_1E}} \qquad \text{.... (1.42)}$$

in which, W = BOD or pollution load in kg/day.

The Eqn. 1.42 indicates that the concentration peak will be created in the estuary during each slack water conditions.

If tidal height variations are considered, the cross-sectional area A is function of x and t. Usually it is sufficient to consider only the longitudinal variation of A, at the mean-tide level, obtained from the hydrographic data.

The velocity V_{xt} is also a function of x and t, and its determination is a hydrodynamic problem, and it depends on the geometry of the estuary, bed roughness, and the characteristics of the tide at the ocean entrance to the estuary. Finite difference formulation for such determination of V_{xt} is also available in the literature (Harleman and Lee). However, in a simplified estuary, the following variation of the instantaneous velocity with time may be assumed:

$$V_{xt} = V_f + V_{max} \sin\left[\frac{48\pi T}{12.5}\right], \qquad \text{.... (1.43)}$$

in which, V_{max} = maximum tidal velocity at any time T,

T = time in days

V_f = fresh water velocity at the head of the tide,

$\frac{48\pi}{12.5}$ = frequency of the tide,

and 12.5 = tidal period

The dispersion coefficient, E, is usually assumed to vary only with x. The longitudinal variation of E is large in the region of salinity instrustion in the estuary. The variation of E may be estimated by writing mass balance equation for salinity, similar to the Eqns. 1.40 and 1.41, and from the actual field observations on salinity variation. However, in the upstream fresh water zone, the value of E is fairly small, and depends mainly on tidal velocity, and is given by the following equation:

$$E = 63\, nVR^{5/6} \qquad \text{.... (1.44)}$$

in which, n = Mannings coefficient of roughness of the estuary,
R = hydraulic radius in meters,
V = local tidal velocity in meters/sec,
and E = longitudinal dispersion coefficient in m^2/sec.

For the finite difference formulation of the numerical method of solution of the Eqns. 1.40 and 1.41, the readers may refer to the literature listed at the end of this chapter.

In a simplified estuary, where the terms S_r, L_b and K_3 in the Eqns. 1.40 and 1.41, are neglected, all other terms except the velocity are assumed to be constant, and assumed that the average velocity of flow in a tidal cycle is zero, the steady state solutions of the Eqns. 1.40 and 1.41, are given by the following two equations (8):

$$L(x) = L_a \exp\left[\pm x(K_1/E)^{\frac{1}{2}}\right], \qquad \text{.... (1.45)}$$

and,

$$C(x) = C_3 - \frac{K_1 L_a}{K_2 - K_1}\left[\exp\left\{\pm x(K_1/E)^{\frac{1}{2}}\right\} - (K_1/K_2)^{\frac{1}{2}} \exp\left\{\pm x(K_1/E)^{\frac{1}{2}}\right\}\right] \qquad \text{.... (1.46)}$$

in which, $L_a = \dfrac{W}{2A\sqrt{K_1 E}}$, the value of L at $x = 0$,

and W = input pollution load (BOD) in kg/day.

The BOD and DO values, averaged over a tidal cycle, obtained through a numerical analysis, assuming the velocity of flow to vary as given in the Eqn. 1.43 and all other terms in the Eqns. 1.40 and 1.41 to be constant, closely approximate to the values obtained for a steady state condition values, as given by the Eqns. 1.45 and 1.46.

Example 1.1: A city discharges 115 MLD of sewage into a stream whose minimum rate of flow is 8500 litres/sec. The velocity of the stream is about 3.2 Kmph. The temperature of the sewage is 20°C, while that of the stream is 15°C. The 20°C 5 day BOD of the sewage is 200 mg/1, while that of the stream is 1.0 mg/1. The sewage contains no DO, but the stream is 90% saturated at the upstream of the discharge. At 20°C, K_1 is estimated to be 0.3 per day, while K_2 is 0.7 per day. Determine the critical oxygen deficit and its location. Also estimate the 20°C 5 day BOD of a sample taken at the critical point. Use temperature coefficient of 1.047 per °C for K_1 and 1.024 per °C for K_2. Neglect K_3, L_b and S_r.

Solution :

1. Saturation concentration of Oxygen at 15°C = 10.2 mg/1.
   Do in the stream = 0.9 × 10.2 = 9.18 mg/1.
2. Discharge of sewage = 115 MLD = 1330 litres/sec.
3. Temperature of stream after mixing

$$= \frac{1330 \times 20 + 8500 \times 15}{1330 + 8500} = 15.7^\circ\text{C}$$

4. Do of the mixture

$$= \frac{1330 \times 0 + 8500 \times 9.18}{1330 + 8500} = 8.0 \text{ mg} / 1$$

5. 5 day 20°C BOD of the mixture

$$= \frac{1330 \times 200 + 8500 \times 1}{1330 + 8500} = 27.7 \text{ mg} / 1$$

6. Assuming BOD rate constant K = 0.3 per day at 20°C, initial first stage BOD, L_a

$$= \frac{27.7}{1 - e^{-0.3 \times 5}} = 35.7 \text{ mg/1}$$

7. Now, K_1 at 15.7°C
   $= 0.3 \times (1.047)^{15.7-20} = 0.247$ per day
   and K_2 at 15.7°C
   $= 0.7 \times (1.024)^{15.7-20} = 0.63$ per day

8. Saturation concentration of DO at 15.7°C = 10.1 mg/l
   Initial DO deficit, D_a = 10.1 – 8.0 = 2.1 mg/l
   Self-purification ratio, $f = K_2/K_1$
   $= 0.63/0.247$
   $= 2.55$
9. Therefore, from Eqn. 1.25,

$$x_c = \frac{2.3 \times 24 \times 3.2}{0.247(2.55-1)} . \log\left[2.55\left\{1-(2.55-1)\frac{2.1}{35.7}\right\}\right]$$

$$= 169 \text{ Km}$$

$$t_c = 169/(24 \times 3.2) = 2.2 \text{ days.}$$

10. Critical deficit, D_c

$$= \frac{35.7 \times e^{-0.247 \times 2.2}}{2.55}$$

$= 8.15$ mg/l

Minimum DO = 10.1 – 8.15 = 1.95 mg/l

11. Now first stage BOD at critical point, L_c

$= L_{a.}\, e^{-K_1 t_c} = 35.7 \times e^{-0.247 \times 2.2}$

$= 20.8$ mg/l

∴ 5 day 20°C BOD at 169 Km downstream to the point of discharge

$= 20.8\ (1 - e^{-0.3 \times 5})$

$= 16.2$ mg/l

QUESTIONS

1. Draw a typical oxygen sag curve in a stream subjected to pollution.
2. How do you determine the reaeration rate constant of a biological process?
3. Write down the equations that depict the dynamic BOD-DO Model in an estuary.