DEVELOPMENT OF A MODEL OF THE CONTACT STABILIZATION PROCESS

by

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LITERATURE REVIEW AND THEORETICAL CONSIDERATIONS

Contact stabilization and a variation of the contact process, two stage aeration, are modifications of the activated sludge process of waste treatment. In a conventional activated sludge system (Figure 1) wastewater is aerated with a mixed, heterogenous microbial population for a period of 6-8 hours. The sludge is then allowed to settle under quiescent conditions and a portion is returned to the head of the aeration tank where it is again mixed with the incoming wastewater. The clarified supernatant is discharged to the receiving stream.

The contact stabilization modification (Figure 2) generally employs two aeration basins separated by a clarifier. Wastewater and activated sludge are mixed together or "contacted" in the first basin and aerated for 0.5 to 1.5 hours. The sludge is then settled out in the clarifier and aerated or "stabilized" for 4-8 hours in the second basin. This stabilized sludge is then returned to the contact tank to continue the waste treatment process.

Ullrich and Smith (24) were the first to utilize the contact stabilization system. They applied the "biosorption" process to an overloaded conventional activated sludge plant in Austin, Texas. The plant was originally designed to treat 6.0 MGD but by 1950 the average flow rate was 10.7 MGD. The effluent BOD and SS concentrations had increased significantly and operational problems and plant upset were nearly continuous. Plant modification to the biosorption process reduced operational problems, increased capacity to 16 MGD and produced average effluent BOD_5 and SS concentrations of 22 mg/l and 24 mg/l respectively (9). At about this same time, Eckenfelder (5) developed a similar modification of the conventional process to treat high strength organic wastes from a cannery. During the first two years of operation, the process averaged 80-90% BOD removal

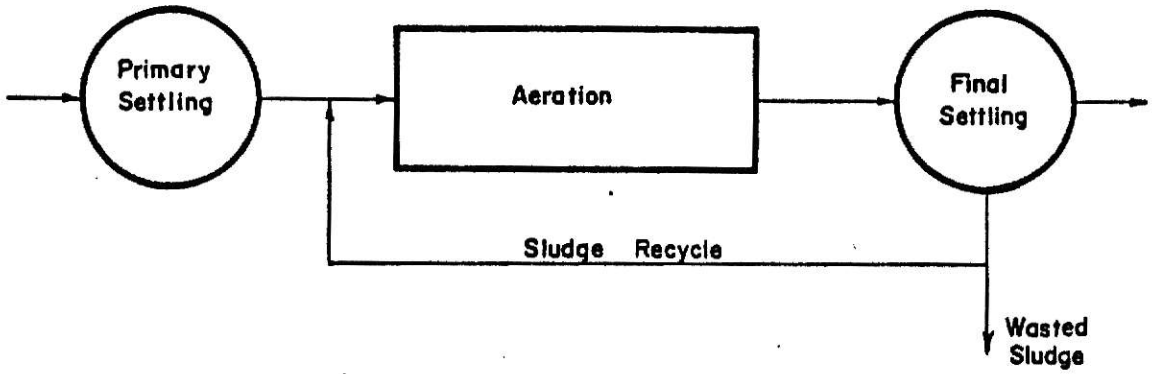

Figure 1—Conventional Activated Sludge System

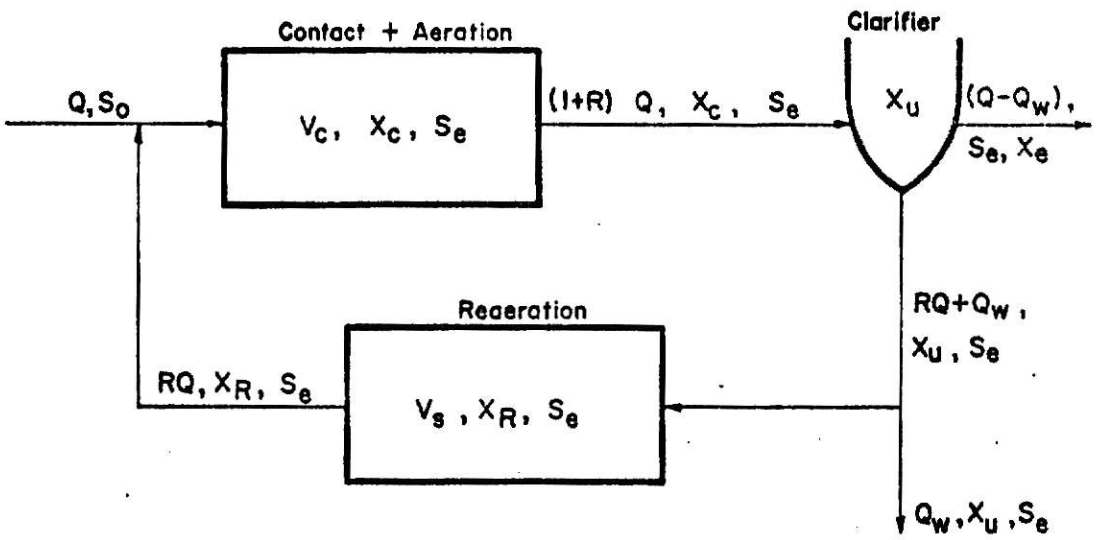

Figure 2—Contact Stabilization System

despite frequent shock loads.

These investigators based the development of their new processes on an observed phenomenon of activated sludge. They established that activated sludge mixed with raw wastewater for short (10-30 minutes) periods of time produced BOD reductions of 85-95% in the supernatant of the clarified mixed liquor. They also determined that aeration of the settled sludge for 1.5-3 hours was necessary to recondition or stabilize the sludge so that it could be again mixed with raw wastewater for effective BOD removal. If the sludge was stabilized for too long a period it became "over-oxidized" and lost its ability to remove BOD. Sludge that was stabilized for too short a period was termed "under-oxidized" and settled poorly although its BOD removal efficiency remained high.

The contact stabilization process takes advantage of the rapid removal of biodegradable substrate (BOD) that is characteristic of a properly stabilized sludge. This rapid removal is shown in Figure 3. It is generally attributed to a combination of biochemical and physicochemical mechanisms which are: 1) physical adsorption, enmeshment and entrapment of particulate BOD within the flocculent biomass and 2) biochemical absorption and storage of soluble BOD by the biomass. The subsequent rise in the supernatant BOD is attributed to extra-cellular enzymatic hydrolysis and release of the trapped particulate substrate that had been removed during the contact phase. The BOD is then gradually reduced by absorption.

The mechanisms of the contact stabilization process have been studied by several investigators. Smallwood (22) used radioactively labeled particulate algae and soluble nutrient broth and glucose. He found that the algae were adsorbed but the soluble compounds were not. Banerji, et. al. (1) showed that rapid removal of colloidal size starch molecules was due to adsorption. Jones (13) studied the mechanism of phyical entrapment of

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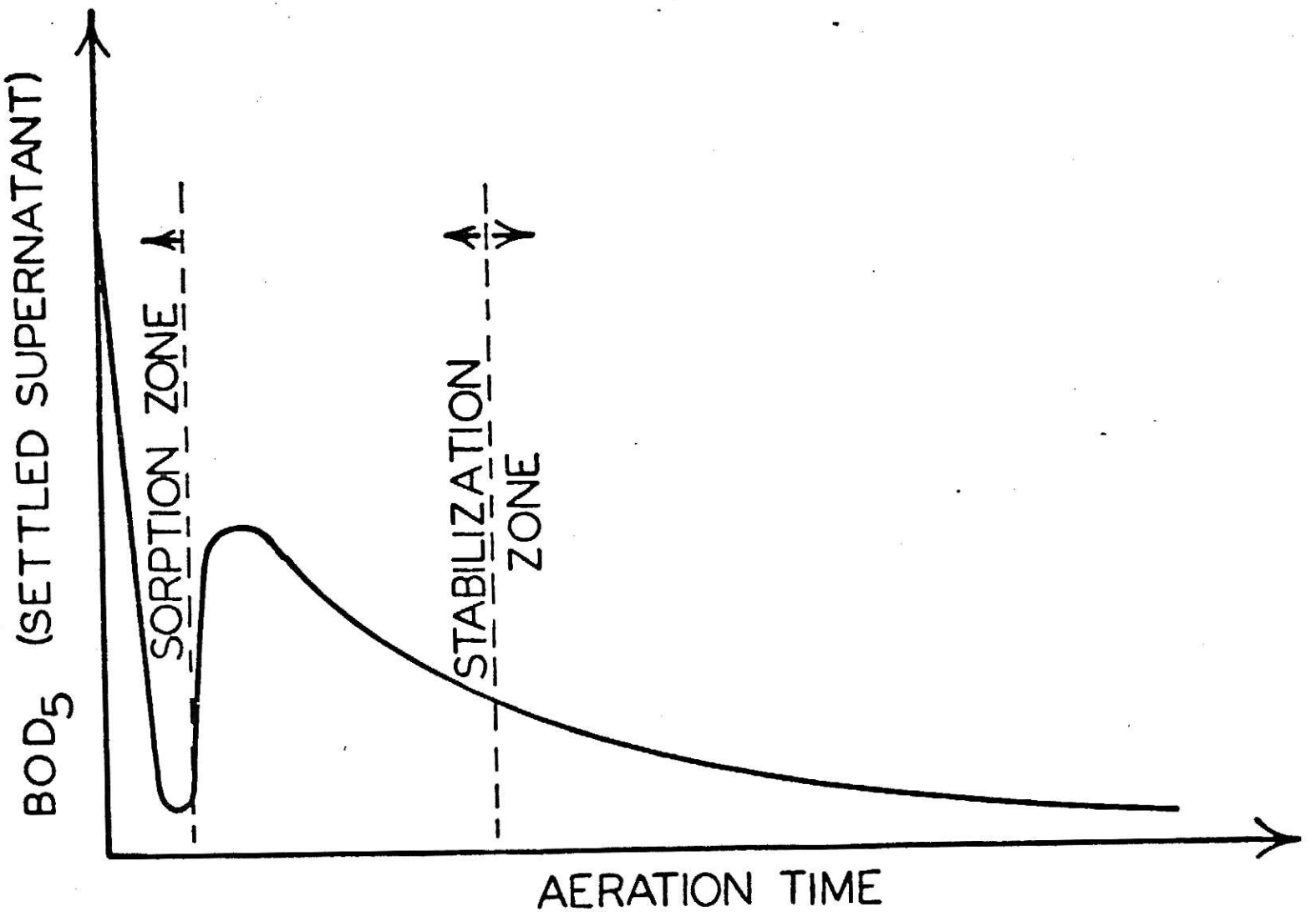

Figure 3. BOD Removal in Contact Stabilization (7).

particulate substrates. He concluded that the rapid removal reaction was a function of the nature of the particulate material, the mixing velocity, and the concentration of biological solids in the flocculant form. Although the removal rate was not determined, it appeared to increase with the percentage of particulate BOD in the wastewater.

Absorption was proposed as a mechanism of rapid removal only after it was demonstrated that soluble materials were not adsorbed. The reasoning was that cells would store large quantities of organics when they were first fed and later metabolize this stored material. Subsequent studies by Siddiqi, et. al. (21) failed to establish absorption and storage as a definitive mechanism for soluble substrate removal. Their data indicated that less than 10% of the glucose in the feed synthetic wastewater was removed by absorption. This led McCarty (15) to conclude that the only reasonable explanation for the observed rapid removal of soluble substrate was a rapid metabolism of the material rather than absorption and storage followed by a "delayed" metabolism.

Because the initial BOD removal is rapid, the contact period can be as short as 15-30 minutes. The short contact time, however, makes the process susceptible to upset when the influent volumetric flow rate has large variations. For the wastewater volumes commonly served by package plants, peak influent flow rates can often exceed the mean flow rate by a factor of 2-4 (6). Thus, if a conventional contact stabilization system was used, the contact time at average dry weather flow (ADWF) would be reduced to 5-10 minutes during peak flow. Because of this sensitivity to fluctuations in the influent flow rate, the contact stabilization process is not well adapted to package plants. A variation of the contact process called two stage aeration was therefore developed to fill the need for a package plant that would utilize the rapid removal ability of stabilized

activated sludge. In the two stage aeration system, the contact period at ADWF is extended to 90-150 minutes and the stabilization period is increased to 7.5 to 8 hours. The flow scheme is identical to that of a contact stabilization system. Figure 4 indicates that the first stage aeration is long enough to avoid the secondary increase in BOD and also allows for some of the substrate to be metabolized before clarification. The longer second stage allows sufficient time for endogenous respiration which helps minimize the solids accumulation. This results in a smaller waste sludge volume than in a conventional contact stabilization system.

Current design criteria for the contact stabilization process are significantly different from the concept originally proposed by Ullrich and Smith. The recommended design standards specified by many regulatory agencies require a three hour rather than 30 minute detention time in the contact zone. The Ten State [GLUMRB] Standards (20), for example, recommend contact times of 2-3 hours depending on the plant design flow. Therefore, what has been commonly referred to as a contact stabilization process, is, in many cases, a two stage aeration system with its added stability for package plant design.

Both of these reaeration processes have several advantages over the conventional activated sludge process. They are:

1) reduced aeration tankage requirements.
2) greater capability for damping and recovering from toxic shock loads since only a portion rather than the entire mixed liquor is adversely affected.
3) elimination of the need for primary sedimentation and its raw sludge disposal requirements.

Until recently, there have been few attempts to mathematically describe the mechanisms and kinetics of the contact stabilization process.

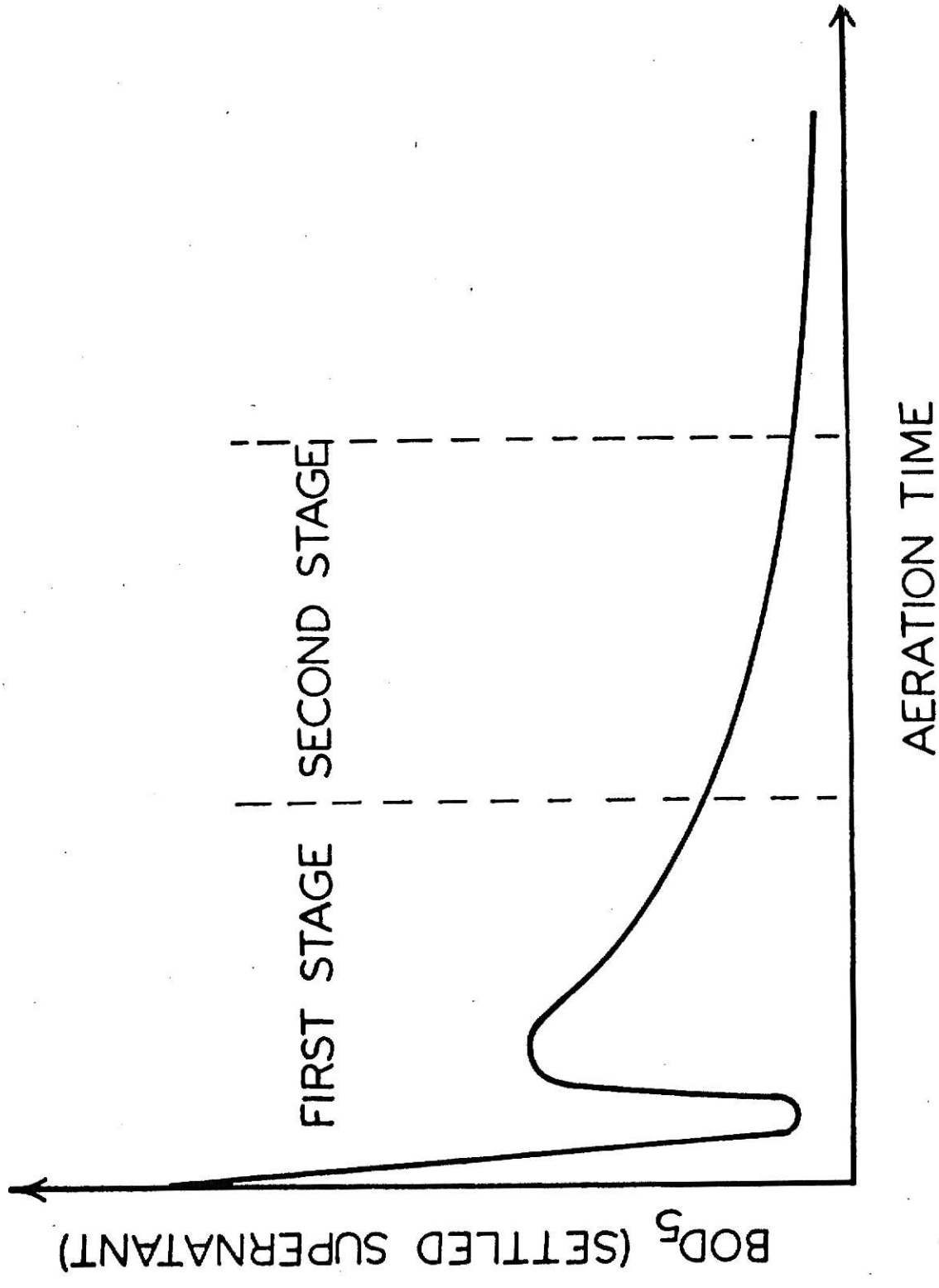

Figure 4. BOD Removal in Two Stage Aeration (7).

Jenkins and Orhon (12) developed a design nomograph relating the sludge specific growth rate (reciprocal sludge age) to the specific removal rate (F/M). They determined that for the contact stabilization process there was not a unique sludge age associated with each specific organic removal rate as is typical of conventional activated sludge systems (11). Instead, the sludge age-removal rate relationship was a family of curves dependent upon the fractional distribution of microbial mass between the contact and stabilization tanks. By manipulation of design and operating conditions, sludge production or the growth rate could be controlled independent of the specific organic removal rate. Thus, the degree of nitrification, a parameter which is directly proportional to the net growth rate, could be varied at a given substrate removal rate. Important results of these observations were that sludge production could be reduced and the loading at which nitrification occurred could be increased. These results were obtained without any deterioration in effluent quality compared to a conventional treatment process.

Subsequent investigations, however, did not support these findings. Gujer and Jenkins (8) found that the contact stabilization and conventional activated sludge processes produce approximately the same amount of sludge and the fractional distribution of sludge between the contact and stabilization basins did not significantly influence sludge production. They also reestablished the single linear relationship between the specific growth rate and removal rate independent of the fractional distribution of the sludge mass between the two reactors. They recognized that both the sludge recycle ratio and the distribution of sludge mass influenced the removal kinetics of soluble and particulate substrates. They did not present however, any operational or design criteria to quantify these relationships. It has

been well established (13) that substrate removal efficiency during a brief contact period increases with the fraction of particulate substrate in the influent. A mathematical model of the contact stabilization process should therefore reflect these observed phenomena.

Benefield and Randall (2) developed a series of design equations based on a combination of the activated sludge models of McKinney (16) and Lawrence and McCarty (14). Application of the equations, however, required knowledge of several kinetic constants that were either undefined or could not be evaluated by typical laboratory activated sludge procedures.

The Unified Model (7) of McKinney and Eckenfelder has also been applied to contact stabilization. It provides a comprehensive analysis of sludge active mass and endogenous mass, effluent soluble and particulate substrate concentrations and oxygen requirements. Many of its kinetic and mechanistic concepts are the same as those in the proposed model. It is applicable, however, only if reliable values for the many necessary constants are available.

EXPLANATION OF TERMS

In order to develop and utilize the theoretical design equations for the contact stabilization process, certain terms and concepts must be clearly defined. The term "substrate" is used in this paper to refer to the biodegradable organic material in a wastewater. The substrate is used by the microbial population for energy and for synthesis of new cell material. In wastewater, substrate may be in the soluble or particulate form. "Soluble substrate" is any organic compound that is dissolved in the wastewater. "Particulate substrate" is suspended and/or colloidal size material that is not dissolved in the wastewater. The "total substrate" in a wastewater is the sum of the soluble and particulate substrate fractions.

Total substrate removal in the contact stabilization process is accomplished in two distinct and sequential steps, that is, uptake or removal, followed by utilization. Substrate is termed "removed" by one of the previously described mechanisms when it is no longer measurable in the supernatant of the mixed liquor. Utilization of the removed substrate is complete when the microbial population has metabolized the substrate for energy and synthesis at the expense of dissolved oxygen. Substrate cannot therefore, be metabolized without first being removed, but it can be removed and not immediately metabolized.

Three other terms, "exogenous substrate," "sorbed substrate," and "endogenous substrate," need to be clarified. Exogenous substrate is any substrate, soluble or particulate, that is not removed from the wastewater. It is external to the microbial cells and has not yet been entrapped or enmeshed by the biological floc. Sorbed substrate is particulate substrate that is also external to the microbial cell but has been trapped within the biological floc either by adsorption or physical entrapment. Endogenous substrate is stored within the microbial cell awaiting metabolism. It may be in the form of the microbial storage compounds glycogen and PHB, or it may be an expendable portion of cellular material that is readily metabolized during starvation. Unlike exogenous substrate, sorbed and endogenous substrates cannot be measured in the supernatant of the mixed liquor.

OBJECTIVE

The objective of this paper was to develop design equations for the contact stabilization process, based on the kinetic models of Lawrence and McCarty (14) and McKinney and Eckenfelder (7). A procedure for evaluation of the necessary kinetic constants and a design example are also presented.

BASIC ASSUMPTIONS

In order to develop a rational mathematical model of the contact stabilization process, the following assumptions were made:

1. The mass of microbial solids in the raw wastewater stream entering the contact tank is negligible compared to the mass of solids under aeration.
2. No microbial activity takes place in the secondary clarifier. The sludge growth rate is therefore dependent only on microbial activity in the aeration tanks, and thus sludge age calculations include only the mass of solids under aeration.
3. Complete mix conditions exist in both the contact and reaeration tanks.
4. Steady state conditions exist throughout the system.
5. The rates of removal of total substrate and soluble substrate follow first order reaction kinetics.
6. The soluble substrate in the influent wastewater that is removed in the contact tank is also metabolized in that tank. All of the particulate substrate in the influent wastewater is removed in the contact tank but is metabolized in the reaeration tank. There is, therefore, no particulate substrate of wastewater origin in the effluent from the final clarifier.
7. Exogenous substrate that enters the reaeration tank is not metabolized and thus passes through unchanged. Removal of substrate is therefore accomplished exclusively in the contact tank.
8. The cell yield and endogenous decay coefficients for soluble substrate and particulate substrate are equal.

DEVELOPMENT OF DESIGN EQUATIONS

A schematic of a typical contact stabilization process is shown in Figure 2. The nomenclature used to develop the design equations is presented in this figure and is defined as it appears in the text.

Writing a materials balance for substrate around the contact tank yields the following expression:

Rate of change in substrate mass around the contact tank	=	Substrate in influent waste stream + substrate in recycle from the reaeration tank - substrate removed in contact - substrate lost in effluent.

or

$$\left(\frac{dS}{dt}\right) V_C = QS_o + QRS_e - \left(\frac{dS}{dt}\right)_T V_C - (1 + R)QS_e \qquad (1)$$

where

$\left(\frac{dS}{dt}\right)$ = net rate of change in the substrate concentration in the contact tank, $ML^{-3}T^{-1}$

V_C = volume of the contact tank, L^3

Q = volumetric flow rate of the raw wastewater, L^3T^{-1}

S_o = total substrate concentration of the raw wastewater, ML^{-3}

R = volumetric recycle ratio

$\left(\frac{dS}{dt}\right)_T$ = rate of total substrate removal per unit volume per unit time, $ML^{-3}T^{-1}$

S_e = total wastewater substrate concentration remaining in the effluent from the final clarifier, ML^{-3}

Since all of the influent particulate substrate is removed in the contact tank (Assumption 6), S_e represents the concentration of soluble substrate in the clarifier effluent:

$$S_e = (S_e)_p + (S_e)_s \quad (2)$$

where

$(S_e)_s$ = soluble substrate concentration remaining in the clarifier effluent, ML^{-3}

$(S_e)_p$ = particulate substrate concentration remaining in the clarifier effluent, ML^{-3}

but

$$(S_e)_p = 0$$

Therefore,

$$S_e = (S_e)_s$$

Any additional oxygen demand exerted by the carryover of microbial solids must be added to the demand exerted by the substrate concentration, S_e. Term S_e also represents the soluble substrate concentration in the effluent from the reaeration tank since there is no exogenous substrate removed during reaeration (Assumption 7).

At steady state conditions, $\left(\frac{dS}{dt}\right) = 0$ and equation (1) can be rewritten for first order substrate removal kinetics as follows:

$$0 = QS_o + QRS_e - K_T X_C S_e V_C - QS_e - QRS_e \quad (3)$$

where

K_T = first order rate constant for total substrate removal per unit of microbial mass per unit time, $L^3M^{-1}T^{-1}$

and

X_C = concentration of microbial mass in the contact tank, ML^{-3}

Rearranging equation (3) yields

$$Q(S_o - S_e) = K_T X_C S_e V_C \quad (4)$$

Since

$$\frac{V_C}{Q} = (t_d)_C \quad (5)$$

where

$(t_d)_C$ = hydraulic detention time in the contact tank, T

then

$$S_o - S_e = K_T X_C S_e (t_d)_C \tag{6}$$

Solving for S_e,

$$S_e = \frac{S_o}{K_T X_C (t_d)_C + 1} \tag{7}$$

A materials balance for the net rate of change in microbial mass around the reaeration tank yields:

Rate of change in microbial mass in the reaeration tank = Microbial mass in the influent to the reaeration tank + microbial growth from metabolism of particulate substrate - microbial mass lost in the effluent from the reaeration tank.

or

$$\left(\frac{dX}{dt}\right) V_R = RQX_U + \left[Y\left(\frac{dS}{dt}\right)_p - k_d X_R\right] V_R - RQX_R \tag{8}$$

where

$\left(\frac{dX}{dt}\right)$ = net rate of change in the microbial mass in the reaeration tank, $ML^{-3}T^{-1}$

V_R = volume of the reaeration tank, L^3

X_U = concentration of microbial mass in the underflow from the secondary clarifier, ML^{-3}

X_R = concentration of microbial mass in the reaeration tank, ML^{-3}

Y = gross cell yield coefficient

$\left(\frac{dS}{dt}\right)_p$ = rate of particulate substrate metabolism per unit volume per unit time, $ML^{-3}T^{-1}$

k_d = endogenous decay coefficient, T^{-1}

Equation (8) can be rewritten for a finite time period as

$$\left(\frac{\Delta X}{\Delta t}\right) V_R = RQX_U + \left[Y\left(\frac{\Delta S}{\Delta t}\right)_p - k_d X_R\right] V_R - RQX_R \qquad (9)$$

The term $\left(\frac{\Delta S}{\Delta t}\right)_p$ represents the rate of metabolism of the particulate substrate removed during the contact period.

Since

$$\left(\frac{\Delta S}{\Delta t}\right)_p = \frac{(S_o)_p - (S_e)_p}{(t_d)_R}$$

and

$$(S_e)_p = 0$$

where

$(t_d)_R$ = hydraulic detention time in the sludge reaeration tank, T

and

$(S_o)_p$ = particulate substrate concentration in the raw wastewater, ML^{-3}

then

$$\left(\frac{\Delta S}{\Delta t}\right)_p = \frac{(S_o)_p}{(t_d)_R}$$

and equation (9) can be rewritten for steady state conditions as follows:

$$\left(\frac{\Delta X}{\Delta t}\right) V_R = 0 = RQX_U + Y\left[\frac{(S_o)_p}{(t_d)_R}\right] V_R - k_d X_R V_R - RQX_R \qquad (10)$$

Since $(t_d)_R = \frac{V_R}{RQ}$, Equation (10) can be reexpressed as

$$0 = RQX_U + Y(S_o)_p RQ - k_d X_R V_R - RQX_R \qquad (11)$$

Solving for V_R, the reaeration tank volume, yields:

$$V_R = \frac{RQ[X_U - X_R + Y(S_o)_p]}{k_d X_R} \qquad (12)$$

Alternatively, equation (12) can be rearranged, after appropriate substitution to determine the mixed liquor solids concentration in the reaeration tank, X_R.

$$X_R = \frac{X_U + Y(S_o)_p}{k_d(t_d)_R + 1} \tag{13}$$

A materials balance for the net rate of change in microbial mass around the contact tank yields:

Rate of change in microbial mass in the contact tank	=	Microbial mass in recycle + microbial growth from metabolism of soluble substrate - microbial solids lost in the effluent from the contact tank.

or

$$\left(\frac{dX}{dt}\right) V_C = RQX_R + \left[Y\left(\frac{dS}{dt}\right)_S - k_d X_C\right] V_C - (1 + R)QX_C \tag{14}$$

where

$\left(\frac{dX}{dt}\right)$ = net rate of change in the microbial mass concentration in the contact tank, $ML^{-3}T^{-1}$

$\left(\frac{dS}{dt}\right)_S$ = rate of soluble substrate metabolism per unit volume per unit time, $ML^{-3}T^{-1}$

Equation (14) can be rewritten for first order substrate removal as follows:

$$\left(\frac{dX}{dt}\right) V_C = RQX_R + YK_S X_C (S_e)_s V_C - k_d X_C V_C - QX_C - RQX_C \tag{15}$$

where

K_S = first order rate constant for soluble substrate removal per unit of microbial mass per unit time, $L^3M^{-1}T^{-1}$

At steady state, $\left(\frac{dX}{dt}\right) = 0$ and the expression becomes

$$R = \frac{(k_d V_C + Q - YK_S(S_e)_s V_C)X_C}{Q(X_R - X_C)} \tag{16}$$

Since $(S_e)_s = S_e$, equation (16) can be written as

$$R = \frac{(k_d V_C + Q - YK_S S_e V_C)X_C}{Q(X_R - X_C)} \tag{17}$$

McCarty (14) defined the mean cell residance time (MCRT) or sludge age, θ_c, as follows:

$$\theta_c = \frac{\text{total microbial mass under aeration}}{\text{total microbial mass withdrawn daily}}$$

The daily loss of microbial solids includes solids lost intentionally through sludge wasting and those lost unintentionally in the effluent from the secondary clarifier. Therefore,

$$\theta_c = \frac{X_C V_C + X_R V_R}{(Q - Q_w)X_e + Q_w X_U} \tag{18}$$

where

θ_c = sludge age or MCRT, T

Q_w = volumetric rate of sludge wasting, L^3T^{-1}

and

X_e = concentration of microbial mass in the effluent from the secondary clarifier, ML^{-3}

The oxygen uptake rate in each tank is a function of the substrate removed and the cell mass synthesized. If all the substrate was removed and completely oxidized to the metabolic end products of CO_2, H_2O, and energy, the oxygen requirement would be the ultimate oxygen demand of the substrate removed. However, some of the substrate is not oxidized completely, but is instead utilized for the production of new cell material. The total oxygen requirement is, therefore, the amount required to satisfy the ultimate oxygen demand of the substrate removed minus the oxygen equivalent of the cell mass synthesized. The general expression for the oxygen utilization rate is, therefore,

$$O_{O_2} = \left(\frac{dS}{dt}\right)_L - 1.4\left(\frac{dX}{dt}\right) \tag{19}$$

where

$\left(\frac{dS}{dt}\right)_L$ = the rate of change in the substrate concentration, expressed as the ultimate oxygen demand, $ML^{-3}T^{-1}$

1.4 = the oxygen equivalent of the cell mass

$\left(\frac{dX}{dt}\right)$ = cell mass synthesized from the substrate removed, per unit time, $ML^{-3}T^{-1}$

For the contact tank, the oxygen utilization rate, $(O_{O_2})_C$, is

$$(O_{O_2})_C = \frac{(S_o)_s - (S_e)_s}{(t_d)_C} - 1.4\left[\frac{Y[(S_o)_s - (S_e)_s]}{(t_d)_C} - k_d X_C\right] \tag{20}$$

The oxygen utilization rate for the reaeration tank, $(O_{O_2})_R$, is

$$(O_{O_2})_R = \frac{(S_o)_s}{(t_d)_R} - 1.4\left[\frac{Y(S_o)_p}{(t_d)_R} - k_d X_R\right] \tag{21}$$

$(S_o)_p$, $(S_o)_s$, and $(S_e)_s$ are the substrate concentrations expressed as ultimate oxygen demand, ML^{-3}.

TABLE 1. - SUMMARY OF DESIGN EQUATIONS

Design Characteristic	Equation	Equation No.
Effluent Substrate Concentration (neglecting oxygen demand of microbial solids)	$S_e = \frac{S_o}{K_T X_C (t_d)_C + 1}$	(7)
Volume of Sludge Reaeration Tank	$V_R = \frac{RQ[\, X_U - X_R + Y(S_o)_p \,]}{k_d X_R}$	(12)
Microbial Solids Concentration in Sludge Reaeration Tank	$X_R = \frac{X_U + Y(S_o)_p}{k_d (t_d)_R + 1}$	(13)
Return Sludge Recycle Ratio	$R = \frac{(k_d V_C + Q - Y K_S S_e V_C) X_C}{Q(X_R - X_C)}$	(17)
Sludge Age or MCRT	$\theta_c = \frac{X_C V_C + X_R V_R}{(Q - Q_w) X_e + Q_w X_U}$	(18)
Sludge Wasting Rate	$Q_w = \frac{V_C X_C + X_R V_R - Q X_e \theta_c}{\theta_c (X_U - X_e)}$	(22)
Oxygen Utilization Rate in Contact Tank	$(O_{0_2})_C = \frac{(S_o)_s - (S_e)_s}{(t_d)_C} - 1.4\left[\frac{Y[(S_o)_s - (S_e)_s]}{(t_d)_C} - k_d X_C\right]$	(20)
Oxygen Utilization Rate in Reaeration Tank	$(O_{0_2})_R = \frac{(S_o)_p}{(t_d)_R} - 1.4\left[\frac{Y(S_o)_p}{(t_d)_R} - k_d X_R\right]$	(21)

DESIGN PARAMETERS

Table 2 lists the design parameters that must be specified for application of the preceeding equations.

TABLE 2. - INITIALLY REQUIRED DESIGN PARAMETERS

Design Parameter	Comment
S_e(mg/ℓ)	Specified by discharge permit.
X_C(mg/ℓ)	Typical concentration range of 1,000 - 3,000 mg/ℓ (17).
$(t_d)_R$ (hrs.)	Typical reaeration periods of 7.5 - 8.0 hrs. (6).
θ_c (days)	Typical MCRT of 5 - 15 days (17).

Table 3 lists those design coefficients that must be assumed or evaluated by established laboratory techniques. The procedures for these evaluations are presented in Appendix II.

TABLE 3. - REQUIRED DESIGN COEFFICIENTS

Design Coefficient	Comment
K_T(ℓ/mg-hr)	Typical range 0.02 - 0.08 on COD basis (3, 18).
K_S(ℓ/mg-hr)	0.05 on COD basis (18).
k_d (day^{-1})	Assumed constant for soluble and particulate substrate. Typical range is 0.04 - 0.07 day^{-1} on COD basis (11).
Y(mg/mg)	Assumed constant for soluble and particulate substrate. Typical range is 0.33 - 0.67 mgVSS/mg COD (11).

If the required design parameters and coefficients have been evaluated, the design procedure is as follows:

1) Determine the contact tank hydraulic detention time after rearranging equation 7.

$$(t_d)_C = \frac{S_o - S_e}{S_e K_T X_C} \tag{7}$$

2) Determine the contact tank volume for the specified flow rate from equation 5.

3) Determine the recycle ratio from equation 17.

4) Determine the mixed liquor solids concentration in the reaeration tank.

$$X_R = \frac{X_U + Y(S_o)_p}{k_d(t_d)_R + 1} \tag{13}$$

Equation 13 indicates that X_R is a function of the clarifier underflow solids concentration, X_U, and the reaeration period, $(t_d)_R$. Since the underflow concentration is dependent upon the settling characteristics of the mixed liquor, several values of X_R at a specific $(t_d)_R$ should be determined over a reasonable working range of X_U. Similar relationships between X_R and X_U can then be developed for each $(t_d)_R$ over a working range of reaeration periods. Plots of X_R vs. $(SVI)_C$ for various values of $(t_d)_R$ are shown in Figure 5. The graphs indicate that X_R is a strong function of $(SVI)_C$ but nearly independent of $(t_d)_R$. The final choice of X_R, therefore, depends upon the operation of the final clarifier.

5) Determine the volume of the reaeration tank for the design value of X_R from equation 12.

6) Determine the sludge wasting rate from equation 22 for an assumed concentration of the effluent microbial solids, X_e.

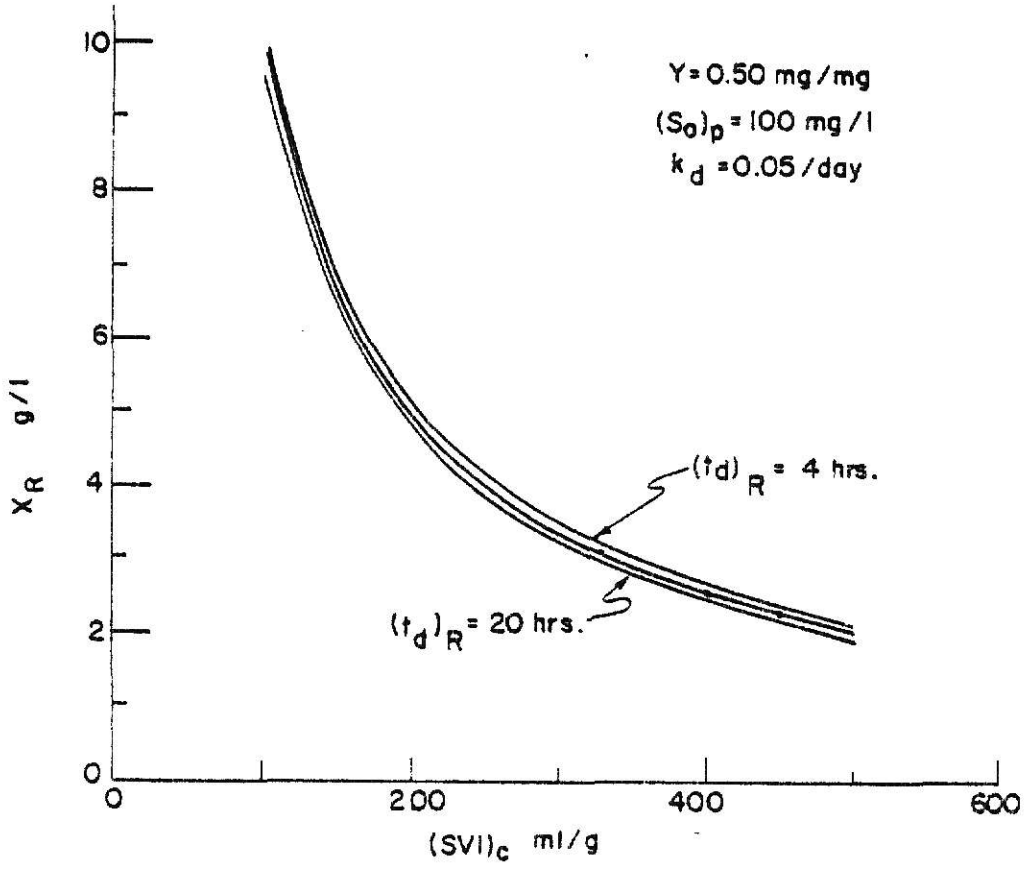

Figure 5. Variation in Reaeration Tank MLSS Concentration as a Function of Contact Tank SVI and Reaeration Tank Detention Time

7) Determine the oxygen uptake rate in the contact tank from equation 20 and the total oxygen required (for organic substrate removal) from equation 23.

$$W_C = (O_{0_2})_C (t_d)_C Q \tag{23}$$

where

W_C = total oxygen required in the contact tank, MT^{-1}

Similarly, in the reaeration tank, the uptake rate is determined from equation 21 and the total oxygen requirement from equation 24.

$$W_R = (O_{0_2})_R (t_d)_R (RQ) \tag{24}$$

where

W_R = total oxygen required in the reaeration tank, MT^{-1}

DISCUSSION

A mathematical model and its basic assumptions can be verified in two ways. The first method utilizes pilot plant operation, extensive data collection and analysis, and calculation of actual operating parameters. The operational data is then compared to the parameters predicted by the model. The second method utilizes available operational data to determine the necessary kinetic coefficients and uses these data to design a treatment process. This system is then analyzed to determine if the design characteristics (tankage requirements, sludge production, air requirements) are reasonable for an actual treatment plant. The latter method was used to evaluate this model. Values of the kinetic coefficients K_T, K_S, Y, and k_d were determined from extensive data on the activated sludge treatment of the soluble and particulate COD fractions of a typical domestic sewage (18).

In the development of this model, certain basic assumptions were made.

Certain ones need verification by means of currently available data and information. Assumption 5 states that the rate of removal of total substrate and soluble substrate follow first order reaction kinetics. In a study using single soluble organic compounds and mixtures of those compounds, Tischler and Eckenfelder (23) concluded that although the actual COD removal rate was the summation of a number of zero order removals, the overall removal rate could be approximated by first order kinetics. They noted that this offered an explanation for the apparent first order removal kinetics frequently observed during the biological treatment of complex wastes. An analysis of Miller's data for total and soluble COD removal showed that the total and the soluble removal rates could be reliably approximated by a first order reaction.

Assumption 6 states that the soluble substrate in the influent wastewater that is removed in the contact tank is also metabolized in that tank. All of the particulate substrate in the influent wastewater is removed in the contact tank but is metabolized in the reaeration tank. There is, therefore, no particulate substrate of wastewater origin in the effluent from the final clarifier. McCarty (15) stated that removal of soluble substrate must be effected by rapid metabolism. He based this on data presented by Siddiqi, et. al. (21). Because the soluble substrate is in the contact tank for 30-150 minutes, this rapid metabolism would occur in the contact tank where soluble substrate removal takes place. In a discussion of a paper by Jacquart, et. al. (10), Orhon (19) stated that particulate removal and its metabolism involved a more complex mechanism than did soluble removal and metabolism. The particulate material was subjected to physical and enzymatic processes before metabolism took place. Hence metabolism of particulate matter would occur during stabilization since there was not sufficient time in the contact tank for enzymatic hydrolysis and subsequent oxidation and synthesis.

Assumption 6 also states that there is no particulate substrate of wastewater origin in the effluent from the final clarifier. An analysis of data from Miller (18) showed that at values of MCRT and MLVSS that permitted a stable system with good sludge settling properties, effluent particle substrate was, indeed, zero.

Assumption 8 states that the cell yield and endogenous decay coefficients for soluble substrate and particulate substrate are equal. This assumption was also used by Jacquart, et. al. (10) in the development of a mathematical model of activated sludge behavior. Analysis of data from Miller (18) showed that the yield coefficient and endogenous decay coeficient for soluble and particulate substrate were not significantly different at the 0.05 level.

DESIGN EXAMPLE

The design example will use a wastewater with a high particulate substrate fraction. The contact stabilization process is ideally suited to such a wastewater. As the ratio of particulate substrate to soluble substrate increases, the detention time in the contact tank, $(t_d)_C$, decreases. The total substrate removal rate constant used for such a wastewater cannot be used to assess detention times in a wastewater with a low proportion of particulate to soluble substrate due to the fact that the amount of particulate substrate to be removed affects the value of the overall removal rate.

The design coefficients Y, k_d, K_T, K_S are taken from an analysis of data of Miller (18). A complete presentation of these data is given in Appendix III.

From Miller (18): $Y = 0.39$ mg/mg (COD basis)

$k_d = 0.036$/day

$K_S = 0.06$ ℓ/mg-day (COD basis)

$K_T = 0.076$ ℓ/mg-day (COD basis)

Assume: S_o = 500 mg COD/ℓ

$(S_o)_s$ = 200 mg COD/ℓ and $(S_o)_p$ = 300 mg COD/ℓ

Q = 0.5 MGD (1900 m^3/day)

$(SVI)_C$ = 250 mℓ/g

$(t_d)_R$ = 8 hr. = 0.333 day

$(S_e)_s$ = 70 mg COD/ℓ

X_C = 2000 mg VSS/ℓ

X_e = 25 mg/ℓ as TSS

θ_c = 10 day

1. $(t_d)_C = (S_o - S_e)/(S_e K_T X_C) = (500 - 70)/[(70)(0.076)(2000)]$

$= 0.0404$ day $= 0.97$ hr. $= 58$ min.

2. $X_U = \frac{10^6}{(SVI)_C} = \frac{10^6}{250} = 4000$ mg VSS/ℓ

$$X_R = \frac{X_U + Y(S_o)_p}{[k_d(t_d)_R + 1]}$$

$$= \frac{4000 + (0.39)(300)}{(0.036)(0.33) + 1}$$

$= 4068$ mg VSS/ℓ

3. $V_C = (t_d)_C Q$

$= (0.0404)(0.5)$

$= 0.0202$ MG $= 2700$ ft^3 (76.41 m^3)

4. $$R = \frac{(k_d V_C + Q - Y K_S S_e V_C) X_C}{Q(X_R - X_C)}$$

$$R = \frac{[(0.036)(0.0202) + 0.5 - (0.39)(0.06)(70)(0.0202)]2000}{0.5(4068-2000)}$$

$R = 0.90$

$Q_R = RQ = (0.90)(0.5) = 0.45$ MGD (1710 m^3/day)

5. $$V_R = \frac{RQ[X_U - X_R + Y(S_o)_p]}{k_d X_R}$$

$$= \frac{(0.90)(0.5)[4000 - 4068 + (.39)(300)]}{(0.036)(4068)}$$

$$= 0.1506 \text{ MG} = 20{,}134 \text{ ft}^3 \ (569.8 \text{ m}^3)$$

6. a) Volumetric loading rate

$$\frac{\text{lb COD/day}}{1000 \text{ ft}^3} = \frac{(500)(0.5)(8.34)}{20.13 + 2.70}$$

$$= \frac{91 \text{ lb COD/day}}{1000 \text{ ft}^3} \ (1458 \text{ kg COD/day/m}^3)$$

if the ratio of BOD_5:COD is approximately 1:2, then

$$\frac{\text{lb } BOD_5}{1000 \text{ ft}^3} = 45.7 \ (732 \text{ kg } BOD_5\text{/day/m}^3)$$

This value is consistent with the recommendations of the Ten State Standards (20), which suggest an aerator loading rate of 30 to 50 lb. BOD_5 per 1000 ft^3 of aeration tank, depending on the plant design flow and the total pounds of BOD_5 per day.

b) Organic loading rate assuming that VSS:TSS is approximately 0.8:1, (17),

$$\frac{\text{lb}BOD_5}{\text{lbMLSS}} = \frac{(500)(0.5)(0.5)}{2000(\frac{1}{0.8})(0.0202) + (4068)(\frac{1}{0.8})(0.1506)}$$

$$= 0.152$$

This organic loading rate is close to the recommended loading rate of 0.2 - 0.5 (20).

7. $$Q_w = \frac{(V_C X_C + V_R X_R - Q X_e \theta_c)}{\theta_c (X_U - X_e)}$$

$$= \frac{[(0.0202)(2000) + (0.1506)(4068) - (0.5)(25)(0.8)(10)]}{10[4000 - (25)(0.8)]}$$

$$= 0.014 \text{ MGD} = 14{,}000 \text{ gal/day} \ (53.2 \text{ m}^3\text{/day})$$

8. O_2 required in

a) contact tank

$$(O_{O_2})_C = \frac{(S_o)_s - (S_e)_s}{(t_d)_C} - 1.4\left[\frac{Y[(S_o)_s - (S_e)_s]}{(t_d)_C} - k_d X_C\right]$$

$$= \frac{200 - 70}{0.0404} - 1.4\left[\frac{0.39[(200 - 70)]}{0.0404} - (0.036)(2000)\right]$$

$$= 1562 \text{ mg } O_2/\ell/\text{day}$$

$$= 65.1 \text{ mg } O_2/\ell/\text{hr or } 32.5 \frac{\text{mg } O_2/\ell/\text{hr}}{\text{g MLVSS}}$$

$$W_C = (O_{O_2})_C (t_d)_C Q$$

$$= (1562)(0.0404)(0.5)(8.34)$$

$$= 263 \text{ lb } O_2/\text{day } (119.3 \text{ kg } O_2/\text{day})$$

b) reaeration tank

$$(O_{O_2})_R = \frac{(S_o)_P}{(t_d)_R} - 1.4\left[\frac{Y(S_o)_P}{(t_d)_R} - k_d X_R\right]$$

$$= \frac{300}{0.333} - 1.4\left[\frac{(0.39)(300)}{0.333} - [(0.036)(4068)]\right]$$

$$= 613.6 \text{ mg } O_2/\ell/\text{day}$$

$$= 25.6 \text{ mg } O_2/\ell/\text{hr or } 6.3 \frac{\text{mg } O_2/\ell/\text{hr}}{\text{g MLVSS}}$$

$$W_R = (O_{O_2})_R (t_d)_R RQ$$

$$= (613.6)(0.333)(0.90)(0.5)(8.34)$$

$$= 767.6 \text{ lb } O_2/\text{day } (348.1 \text{ kg } O_2/\text{day})$$

9. $\frac{263}{263 + 767.6} \times 100\% = 25.5\%$ of the air goes to the contact tank and 74.5% of the air goes to the reaeration tank. The ratio of the volumes of the two tanks, $V_R:V_C$ is 0.1506:0.0202 or approximately 7.5:1.

CONCLUSION

Design equations based on the kinetic models of Lawrence and McCarty and McKinney and Eckenfelder have been developed. The necessary kinetic coeficcients have been evaluated from laboratory data and applied to the design for contact stabilization treatment of a typical domestic wastewater. The system designed was reasonably sized and was in agreement with recommended standards for the contact stabilization process. Since the assumptions upon which these equations were based have been justified by experimental work and since the system that was designed was a reasonable one, it can be concluded that the proposed equations are valid and can be utilized for design of the contact stabilization process.

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APPENDIX II - Notation

Nomenclature

k_d = endogenous decay coefficient, T^{-1}

K_T = first order rate constant for total substrate removal per unit microbial mass per unit time, $L^3M^{-1}T^{-1}$

K_S = first order rate constant for soluble substrate removal per unit microbial mass per unit time, $L^3M^{-1}T^{-1}$

Q = volumetric flow rate of raw wastewater, L^3T^{-1}

Q_w = volumetric sludge wasting rate, L^3T^{-1}

R = volumetric recycle ratio

$(O_{O_2})_C$ = oxygen utilization rate in the contact tank, $ML^{-3}T^{-1}$

$(O_{O_2})_R$ = oxygen utilization rate in the reaeration tank, $ML^{-3}T^{-1}$

S_e = total substrate concentration in the effluent from the final clarifier, contact and reaeration tanks, ML^{-3}

$(S_e)_p$ = particulate substrate concentration in the effluent from the contact tank, ML^{-3}

$(S_e)_s$ = soluble substrate concentration in the effluent from the contact tank, ML^{-3}

S_o = total substrate concentration in the raw wastewater, ML^{-3}

$(S_o)_p$ = particulate substrate concentration in the raw wastewater, ML^{-3}

$(S_o)_s$ = soluble substrate concentration in the raw wastewater, ML^{-3}

$(t_d)_C$ = hydraulic detention time in the contact tank, T

$(t_d)_R$ = hydraulic detention time in the reaeration tank, T

V_C = volume of the contact tank, L^3

V_R = volume of the reaeration tank, L^3

X_C = microbial mass concentration in the contact tank, ML^{-3}

X_e = microbial mass concentration in the effluent from the secondary clarifier, ML^{-3}

X_R = microbial mass concentration in the reaeration tank, ML^{-3}

X_U = microbial mass concentration in the underflow from the secondary clarifier, ML^{-3}

W_C = total oxygen requirement in the contact tank, MT^{-1}

W_R = total oxygen requirement in the reaeration tank, MT^{-1}

Y = cell yield coefficient

θ_c = sludge age or mean cell residence time (MCRT), T

APPENDIX III - Laboratory Methods and Data

Application of the mathematical model described requires knowledge of the following wastewater characteristics: K_T, K_S, Y, k_d, S_o, $(S_o)_s$, $(S_o)_p$, $(S_e)_s$. These terms can be evaluated in bench scale, complete mix continuous flow reactors (4).

All substrate concentrations are evaluated on a COD rather than BOD basis because BOD analysis is time consuming and difficult to reproduce due to varying degrees of oxidation during the test. BOD values also vary because of nitrification during the incubation period. The influent total COD concentration, S_o, is comprised of two fractions, soluble and particulate. The soluble material is that which passes through a 0.45μ pore size filter. The COD of the filtrate is designated $(S_o)_s$. The particulate COD, $(S_o)_p$, is determined by the difference between S_o and $(S_o)_s$. The total effluent COD concentration can be similarly divided into soluble and particulate. Because all wastewater particulate material is assumed to be removed during the contact phase, S_e will contain only soluble substrate and thus $S_e = (S_e)_s$. Any particulate material in the effluent would be due to microbial solids from the mixed liquor being carried out with the final clarifier supernatant.

The substrate removal rate constants, K_T and K_S, the sludge yield coefficient Y, and the endogenous decay coefficient k_d, are evaluated using acclimated sludge operated over a working range of sludge age values (or F/M values). Operation at any specific sludge age is continued until a steady state exists for the mixed liquor volatile solids and effluent COD concentrations. The removal rate constants are determined from the reactor performance data as indicated in Figures 6a and 6b. The linear relationships are established from a least squares analysis and the goodness of fit evaluated

from the correlation coefficient. It should be noted that both rate constants can be evaluated if the influent soluble COD is measured in addition to the conventional operating parameters. It is not necessary, therefore, to operate a separate series of reactors using only the soluble portion of the wastewater as a feed source.

The cell yield coefficient, Y, and the endogenous decay coefficient, k_d, are determined from an empirical relationship between microbial growth and substrate removal in a biological system (7).

$$\frac{dX}{dt} = Y\left(\frac{dS}{dt}\right) - k_d X \tag{25}$$

On a finite basis, this equation can be redefined as a linear relationship.

$$\frac{\Delta X}{\Delta t} = Y\left(\frac{\Delta S}{\Delta t}\right) - k_d X$$

or

$$\frac{\Delta X}{X} = Y\left(\frac{\Delta S/\Delta t}{X}\right) - k_d = \frac{1}{\theta_c} \tag{26}$$

Utilizing this linear relationship, the cell yield and endogenous decay coefficients are determined from reactor performance data as shown in Figure 7.

This relationship assumes that the yield coefficients for the particulate COD and soluble COD are equal. This has been assumed by previous investigators (10), and an analysis of available data (18) indicates that this is a reasonable assumption.

Miller's data (18), as presented in Tables 4a and 4b, were used to evaluate the required design coefficients, K_S, K_T, Y, and k_d. Plots of these data points are shown in Figures 8, 9, and 10.

Miller used two different types of feed which he called soluble-colloidal and supra-colloidal. The soluble-colloidal feed consisted of wastewater from a nearby domestic treatment plant which was passed through

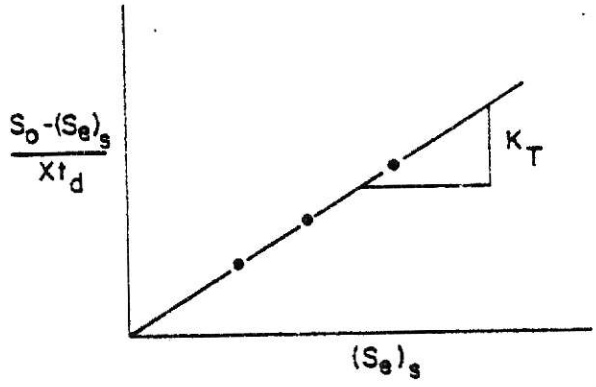

Figure 6a. Determination of total substrate removal rate.

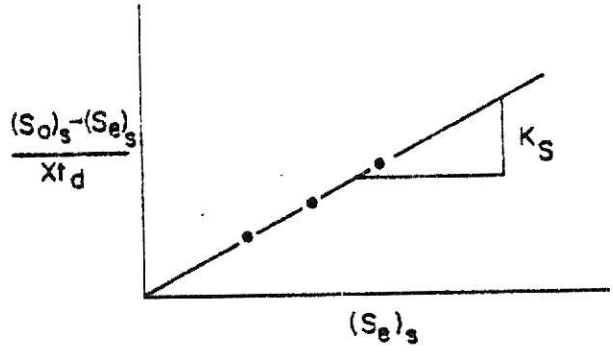

Figure 6b. Determination of soluble substrate removal rate.

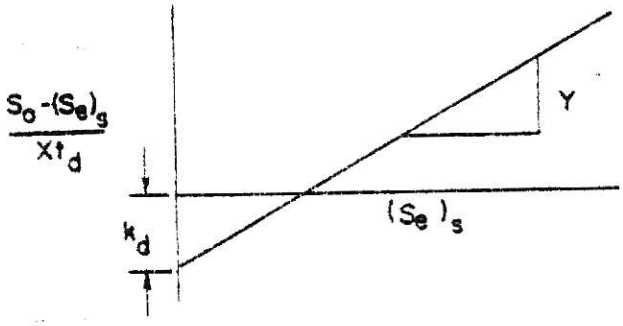

Figure 7. Determination of cell yield and endogenous decay coefficients.

a 100 mesh soil screen filter, then through cartridge filters and a diatomaceous earth filter. This filtrate was then passed through a 0.22μ membrane filter which served to sterilize the feed. The supra-colloidal feed was made from two equal parts of filtered wastewater. The first part was filtered through a soil screen and through a glass wool pad and finally autoclaved. The second part was soluble-colloidal feed which was added to insure the presence of those heat labile compounds that might have been destroyed by autoclaving.

A linear regression analysis was performed on each of the plots. Analysis of the data for the soluble-colloidal, continuous flow system yielded a K_S value of 0.06 ℓ/mg-day on a COD basis. The correlation coefficient for this group of data was 0.46. Linear regression analysis on the supra-colloidal batch feed yielded a K_T value of 0.076 ℓ/mg-day on a COD basis. The correlation coefficient for these data was 0.84. All available data in both tables were used for an evaluation of Y and k_d. This evaluation yielded a Y value of 0.39 mg/mg and a k_d value of 0.036 day^{-1}. The correlation coefficient was 0.97. These coefficients were then used in the design problem calculations.

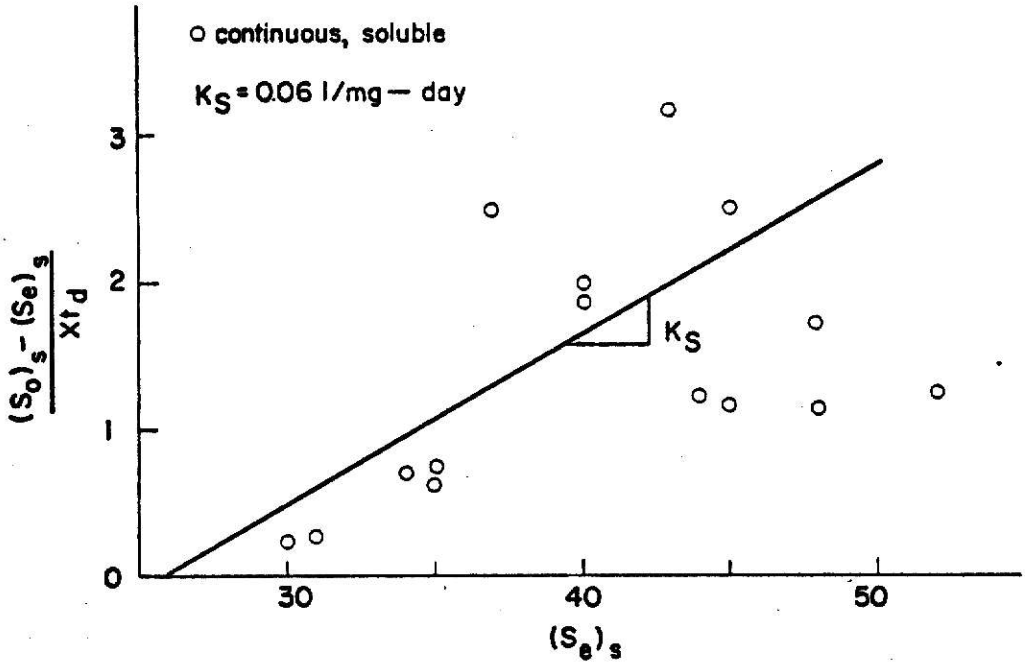

Figure 8. Evaluation of K_S. Data from Miller (18).

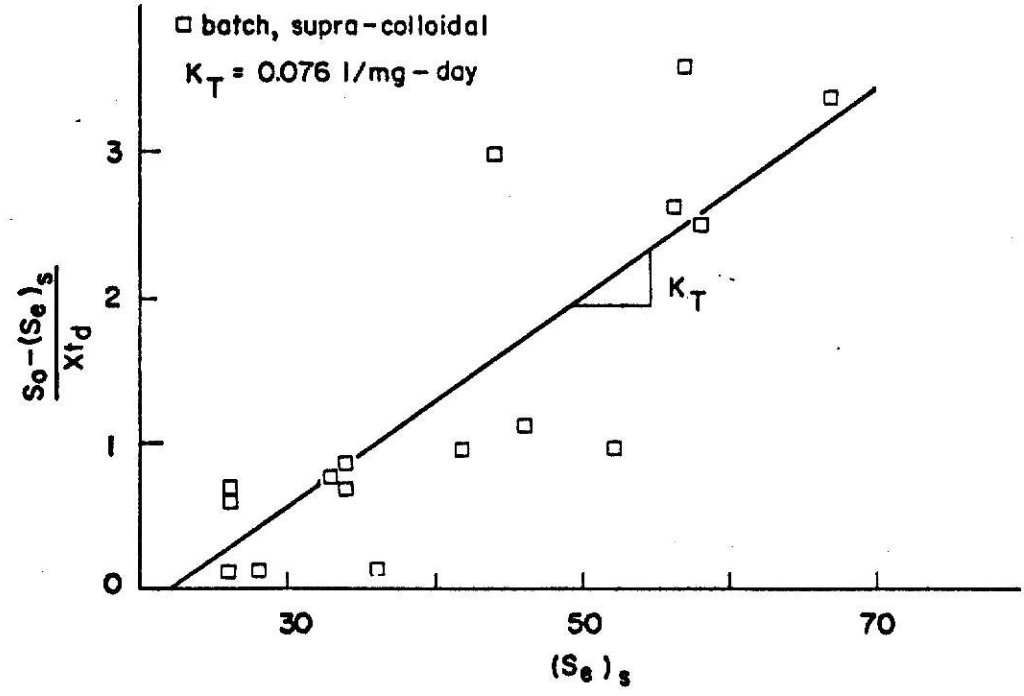

Figure 9. Evaluation of K_T. Data from Miller (18).

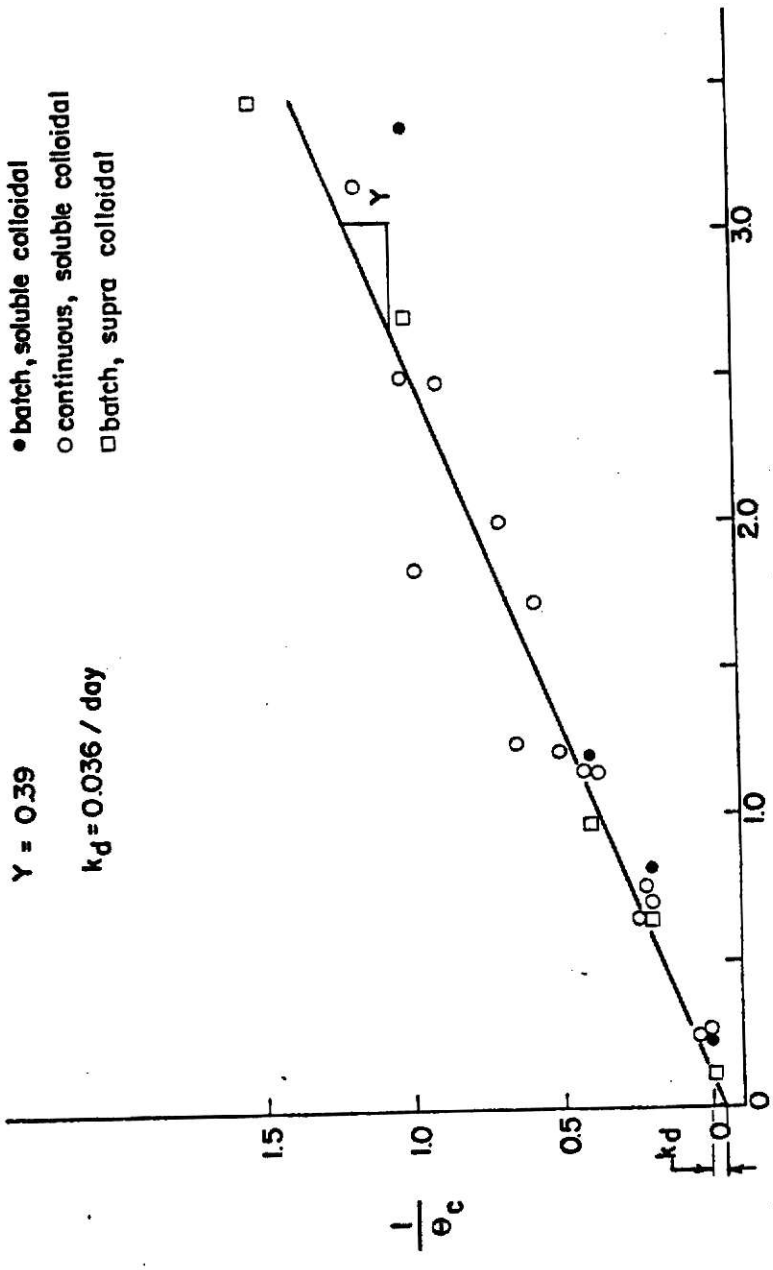

Figure 10. Evaluation of Y and k_d. Data from Miller (18).

TABLE 4a. - DATA FROM MILLER FOR EVALUATION OF K_S, K_T, Y, k_d

	$\dot{q}$[1] COD/(VSS)(day)	$1/\theta c$ (day^{-1})	$(S_e)_s$ (mg/l)
Soluble-colloidal continuous flow	2.03	0.77	40
	1.25	0.65	52
	3.17	1.14	43
	0.26	0.04	30
	0.75	0.23	35
	0.70	0.18	34
	1.14	0.38	45
	2.48	0.90	37
	1.84	0.87	40
	1.15	0.42	48
	2.50	1.02	45
	0.27	0.03	31
	0.64	0.27	35
	1.73	0.57	48
	1.22	0.51	44
Supra-colloidal batch feed	0.12	0.0	26
	0.13	0.0	36
	0.13	0.0	28
	0.59	0.2	26
	0.68	0.2	26
	0.65	0.2	34
	0.75	0.2	33
	0.96	0.4	42
	0.88	0.4	34
	0.94	0.4	52
	1.12	0.4	46
	2.98	1.0	44
	2.49	1.0	58
	2.63	1.0	56
	3.35	1.5	67
	3.58	1.5	57

[1] For soluble-colloidal feed, $q = \dfrac{(S_o)_s - (S_e)_s}{Xt_d}$

For supra-colloidal feed, $q = \dfrac{S_o - (S_e)_s}{Xt_d}$

TABLE 4b. - DATA FROM MILLER FOR EVALUATION OF Y, k_d

	Ave. q^1 for $1/\theta_c$ COD/(VSS)(day)	$1/\theta_c$ (day^{-1})
Soluble-colloidal batch feed	0.23	0.0
	0.82	0.2
	1.22	0.4
	3.35	1.0
Supra-colloidal batch feed	0.1233	0.0
	0.6291	0.2
	0.9594	0.4
	2.7063	1.0
	3.4506	1.5

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[1] For soluble-colloidal feed, $q = \frac{(S_o)_s - (S_e)_s}{Xt_d}$

For supra-colloidal feed, $q = \frac{S_o - (S_e)_s}{Xt_d}$

DEVELOPMENT OF A MODEL OF THE CONTACT STABILIZATION PROCESS

by

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B.S., Rensselaer Polytechnic Institute, 1971

AN ABSTRACT OF A MASTER'S REPORT

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ABSTRACT

Until the last decade, there have been few attempts to kinetically describe the contact stabilization process. This paper reviews developments in contact stabilization and presents design equations based on a combination of existing kinetic models. A procedure for evaluating the necessary kinetic constants and a design example are also included. Evaluation of the kinetic constants used in the design example is based on data collected from the literature on bench scale activated sludge research using domestic sewage as a feed source. Basic assumptions made in the development of the design equations are substantiated by research done by previous investigators. The design equations can be utilized for design of the contact stabilization process.