COOPERATIVE FACILITIES FOR RESEARCH IN WATER RESOURCES AT THE UNIVERSITY OF TEXAS

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Data Processing Center
- LGP-30 system
- 1620-D system
- Pace 221R analog computer

LABORATORY AND DESIGN PROCEDURES
FOR
WASTEWATER TREATMENT PROCESSES

by

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PREFACE

This volume presents a summary of laboratory and design procedures for many of the processes used in water pollution control facilities. The purpose of this compilation is to assist the engineer in establishing laboratory and pilot plant programs necessary for formulating necessary design criteria, to serve as a text for graduate courses in unit operations, and to provide a training manual for University and Government personnel. The theory and principles of the processes cited are amply described in other texts and included in the references.

The authors express their appreciation to Dr. Joseph Malina, Jr. and Dr. S. Balakrishnan and Mr. Guillermo Valencia who checked the example problems and assisted in the collation of this manuscript.

Austin, Texas
December, 1968

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Table 1.

LIST OF SYMBOLS

α = oxygen transfer coefficient

a = fraction of substrate removed, used for cell synthesis

a' = oxygen utilization coefficient for cell synthesis

A_v = specific surface area of filter media for trickling filters

b = cell auto-oxidation rate coefficient (day^{-1})

b' = oxygen utilization rate for endogenous respiration (day^{-1})

BOD_5 = biochemical oxygen demand (5 days at 20°C) mg/l

BOD_u = biochemical oxygen demand (ultimate at 20°C) mg/l

C_o = overflow concentration from sludge thickener, percent solids

C_u = underflow concentration of solids from thickener, percent solids

COD = chemical oxygen demand, mg/l

C_∞ = ultimate solids concentration at infinite time

D, X = depth

f = degradable fraction of volatile solids

F = food to micro-organism ratio expressed as lb BOD/lb MLVSS/day

F_o = overflow concentration for flotation process, percent solids

F_u = underflow concentration for flotation process, percent solids

G_s = air flow, scfm/aeration unit

K' = substrate removal rate coefficients for trickling filters

K, k = substrate removal rate coefficients and biological sludge oxidation rate

L = vacuum filter loading, lb per sq. ft per hour

mg/l = milligrams per liter

m, n = exponents, constant for specific type of sludge

Table 1. (cont'd)

ML = mass loading for sludge thickener–lbs per sq. ft. per day
N_o = amount of nitrogen in influent; amt. of carbon adsorptive
N = recycle ratio, R/Q
O_t = percent concentration of oxygen in air leaving aeration tank
P = vacuum pressure for vacuum filtration
P_o = amount of phosphorus in influent
Q = flow rate (MGD, gpm, etc.)
R = recycle rate (MGD, gpm, etc.)
R_o = specific resistance of the filter media sec^2/gm
s = coefficient of compressibility of filter cake
s_a = solubility of air in water, atmospheric measure
$S_o;S_e$ = influent and effluent substrate concentration
$S_a;S_r$ = substrate (expressed as BOD or COD) applied or removed
SS = suspended solids
t = time, retention time or cycle time in minutes, hrs or days
T = temperature in ^oC
TDS = total dissolved solids, inorganic
UA = unit area, ft^2/lbs solids per day
VSS = volatile suspended solids
V = carbon column flow rate; volume
$X_a;X_d$ X_v = average, degradable fraction, and volatile fraction of the mass of the biological cells - (in lbs)
ΔX_v = cell or sludge yield (lbs VSS per day)
θ = temperature coefficient for the rate constant k or K

Table 2.

Coefficients and Constants for Treatment of Wastes from the Chemical Industry

Activated Sludge	Range of Values (1 standard deviation)	Average
k (1/mg-hr)	0.00012 - 0.00076	0.00024
a	0.31 - 0.72	0.52
a'	0.31 - 0.76	0.53
b (day^{-1})	0.02 - 0.18	0.07
b' (day^{-1})	0.10 - 0.24	0.17
θ	1.0 - 1.04	1.03
F lb BOD/lb MLVSS/day	0.4 - 0.8	0.5
Aerated Lagoon		
K (day^{-1})	0.5 - 1.0	0.75
X_v (mg/l)	50 - 150 mg/l	100
θ	1.085 - 1.1	
Anaerobic Lagoons		
*K (day^{-1})	0.030 - 0.055	0.042
K_e (day^{-1})	0.07 - 0.13	0.10
θ	1.08 - 1.09	1.085
Trickling Filter		
$K' = kxA_v^m$	0.24 - 1.25	
n	0.4 - 0.5	
θ	1.035 - 1.08	
Gravity Thickeners		
B	4 - 16	
n	0.25 - 1.8	
ML	4 - 12 lb/sq ft/day	8
C_u	3 - 5 percent	

* K reaction rate for equation $\frac{S_e}{S_o} = \frac{1}{1 + Kt}$ (applicable to single ponds)

K_e reaction rate constant for equation $\frac{S_e}{S_o} = e^{-K_e \Sigma t}$ (applicable to series of ponds)

K' based on loading in MGAD

Table 2. (cont'd)

Vacuum Filtration	Range of Values	Average
m	0.5 - 0.71	
n	0.5 - 0.70	
s	0.72 - 0.88	
Cake Moisture	72 - 79 by wt.	
Loading Ranges	3 - 7 lb solids sq ft/hr	
Aerobic Digestion		
Retention time	10 - 24 days	15 days
Carbon Adsorption		
lbs COD/lb Carbon	0.3 - 0.6	0.5
Filtration		
gpm/ft^2	5 - 10	
Primary Sedimentation		
Overflow rate gal/day/ft^2	500 - 1000	750
Secondary Sedimentation		
Overflow rate gal/day/ft^2	500 - 700	600

SECTION 1

Laboratory Procedures for Wastewater Treatment Processes

EXPERIMENT 1

SEDIMENTATION OF DISCRETE AND FLOCCULENT SOLIDS AND THE PERFORMANCE OF A SETTLING TANK

Required Materials

8' Settling column

Gooch crucibles tared

Interval timer

Symbols

D = depth, ft.

OR = overflow rate, gpd/ft^2

SS = suspended solids, mg/l

Reference pp 2-7 (See attached Figure 1)

In a flocculent suspension the rate of sedimentation increases with time or depth in a settling tank. The rate of settling of a flocculent mixture cannot be theoretically calculated and is usually determined experimentally. The experiment herein described provides sufficient information to develop the overflow rate vs. removal curves for a particular water or waste. The design for primary sedimentation facilities can then be formulated. A settling column is shown in attached Figure 1.

Procedure

1. Fill the column with the waste. Bubble air through the column gently, but sufficient for mixing the contents to insure uniform initial composition. Be certain that the temperature of the mixture is in equilibrium with the atmosphere.
2. Sample at all taps at times 0, 5, 10, 20, 40, 60, 90, 120 minutes. Analyze the samples for suspended solids. Drain taps before sampling and withdraw sample so as not to disturb the suspension. Perform this test at the anticipated minimum, maximum and average suspended solids concentration in the water or waste.
3. If a sludge interface develops, the heights of the interface should be noted at each sampling time.
4. The final concentration of the sludge should be noted.

Calculations

1. Note the % solids removal at each sampling port for the various periods of time and plot the lines of equal removal as shown in Figure 4.
2. Calculate the total removal at each time interval and plot the % removal as a function of detention time for each initial solids concentration.
3. Plot the % solids removal as a function of detention time and as overflow rate for each initial solids concentration as shown in Figures 5 and 6.
4. Design a primary sedimentation tank for a desired solids removal for a design flow of 3 MGD, initial SS = 400 mg/l, effluent SS = 100 mg/l Apply the appropriate scale up factors to the overflow rate and detention time.

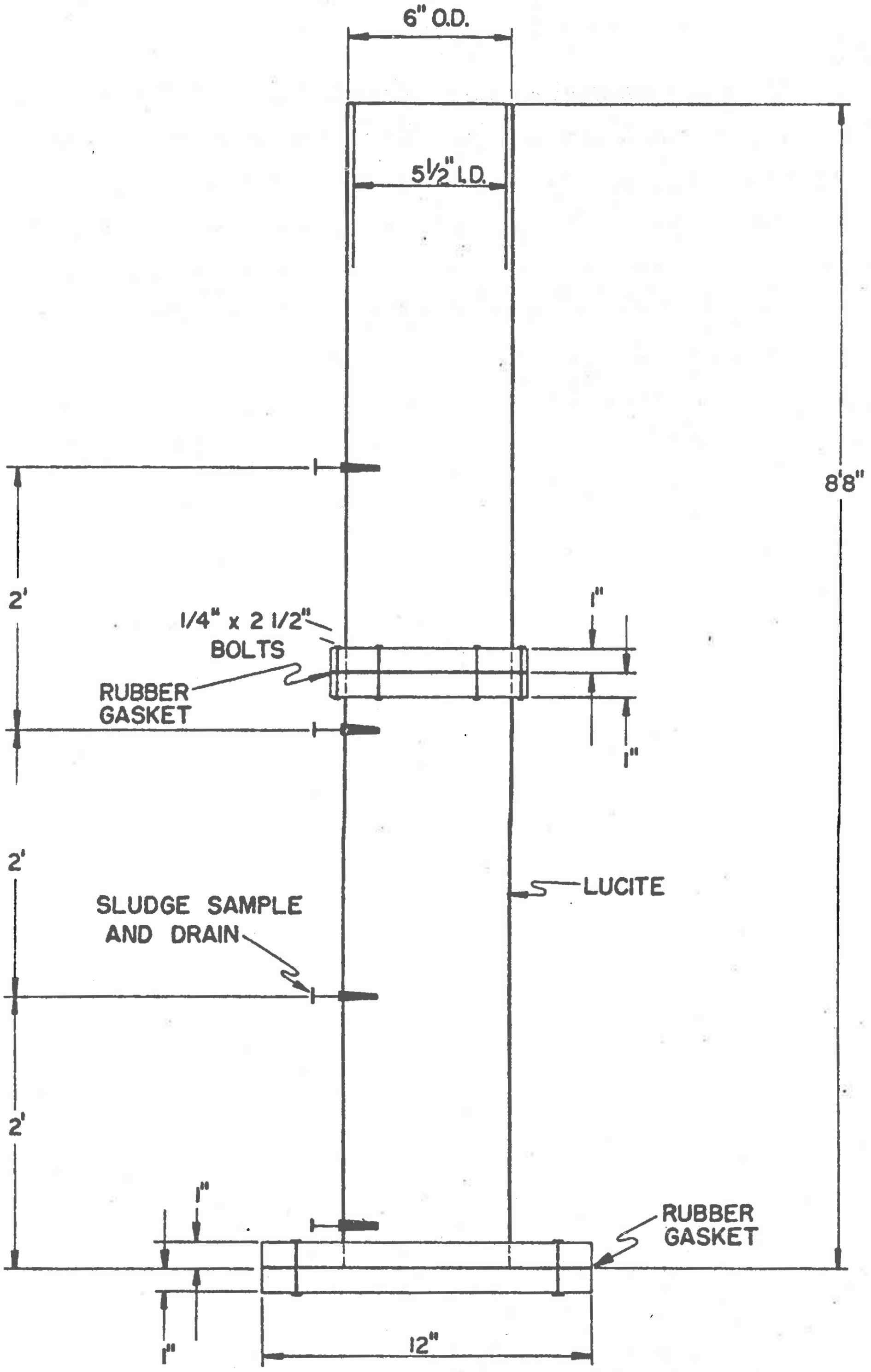


Figure 1 Settling Column

EXPERIMENT 2

FLOTATION

Required Materials

- Compressor
- Flotation Cell
- Stop watch
- Gooch crucibles

Symbols (lab test)

A/S = air/solids ratio, lbs air released/lb solids

s_a = air saturation, cm^3/liter

R = pressurized volume, liters

P = absolute pressure, atm.

Q = waste flow, liters

S_a = influent SS, mg/l

Reference pp 8-12 (See attached Figure 1)

The principle involved in flotation is the same as thickening except the direction of moving of solids. Activated sludge or low specific gravity materials which can be separated by thickening can be separated by flotation also. A laboratory flotation cell is shown in attached Figure 1.

Procedure

1. Partially fill the calibrated cylinder with clarified effluent or water.
2. Apply compressed air to the pressure chamber to attain the desired pressure.
3. Shake the air-liquid mixture in the pressure chamber for 1 min and allow to stand for 3 min to attain saturation. Maintain the pressure on the chamber for this period of time.
4. Release a volume of pressurized effluent to a cylinder and mix with the waste or sludge. The volume to be released is computed from the desired recycle ratio. The velocity of release through the nozzle should be of such a magnitude as not to shear the suspended solids in the feed mixture but to maintain adequate mixing.
5. Measure the rise of the sludge interface with time. Correction must be applied to scale up the height of rise in the test cylinder to the depth of the prototype unit.

6. After a detention time of 20 min the clarified effluent and the floated sludge are drawn off through a valve in the bottom of the cylinder.

7. Relate the effluent suspended solids and the float solids to the calculated air/solids ratio. When pressurized recycle is used, the air/solids ratio is computed:

$$\frac{A}{S} = \frac{1.3 s_a R (P - 1)}{Q S_a}$$

When pressurized recycle is not used,

$$\frac{A}{S} = \frac{1.3 s_a (P - 1)}{S_a}$$

Calculations

1. Develop a relationship between air/solids ratio and effluent suspended solids and float solids.
2. For an activated sludge of 8000 mg/l, design a floatation unit for thickening this sludge to its maximum concentration.

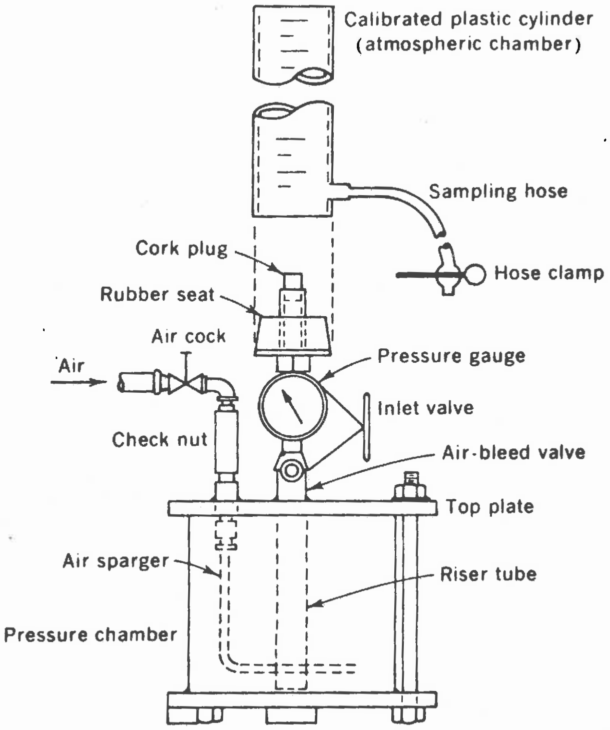


Figure 1 Laboratory Flotation Cell

EXPERIMENT 3

NEUTRALIZATION

Required Materials

1 - liter beaker
Mixer with variable speed adjustment
5% lime slurry
1 inch diameter glass column
Crushed limestone

Reference pp 13-24

Procedure

A. Lime Neutralization

1. Determine the titration curve for the acid waste with lime slurry.
2. Determine the relationship between agitation intensity or power level and contact time to neutralize the acid to pH 7.0 using the equilibrium data obtained in (1).
3. Develop the mathematical relationship which governs the results of (2).
4. Determine the lime requirement.

B. Limestone neutralization

1. Determine the relationship between rate of flow, pH and depth of bed for the neutralization of the acid waste. Vary the depth of bed from 6 inches to 3 feet and the flow rate from 50 gal/ft^2/hr to 1000 gal/ft^2/hr as an upflow velocity.

Calculations

Compare the economics of methods (A) and (B) for the neutralization of 100 gpm of the waste evaluated.

EXPERIMENT 4

WARBURG ANALYSES

Required Materials

Warburg constant volume respirometer

Warburg flasks

20% KOH solution

Krebs or Brodies fluid

Reference (See attached Figures 1 and 2)

Manometric Techniques, Umbreit, W. W., Burris, R. H., and Stauffer, J.F., Burgess Pub. Co., Minneapolis (1959).

Procedure

1. Thoroughly clean and dry all flasks and lubricate all stopcocks and ports with grease. Coat the rim of the center well in the flask to prevent migration of the KOH into the main flask chamber.
2. Check the level of the manometer fluid, adding or withdrawing fluid as required. Fill the water bath to the proper level & adjust the water to the desired test temperature.
3. Add the wastewater to the main flask compartment, add the KOH to the center well (about 1/4 full) and pour the required volume of biological seed through the side arm. Insert a strip of folded filter paper inside the center well to enhance alkali absorption of carbon dioxide. Set up a reference flask, or "thermobarometer" by adding only distilled water to the main flask compartment. The volume added should equal the total waste-seed volume added to the test flasks.
4. Attach the flasks to the support assembly with rubber bands or springs. Check the water level in the bath. The water should just approach the bottom of the ground glass fitting.
5. Turn on the shaking assembly with the gas/vent port open for approximately five minutes, allowing the system to equilibrate. Stop the shaking and check all fittings.

6. Adjust the manometer fluid to the reference point (150 mm) on the closed side of the manometer (right side) with the stopcock open.
7. Close the stopcock, turn on the shaking assembly and begin readings at the desired time intervals. Prior to taking the readings, turn the shaker off and adjust the closed arm to the reference position, and read the left, or open arm. Record each time the gas/vent stopcock must be opened in order to readjust the manometer fluid. A format for recording data is described below:

Time	Manometer Reading (Open Side)	Δ	Thermobarometer Correction*	Corrected Δ	Uptake μl	ΣUptake μl

*Thermobarometer corrects for atmospheric pressure changes during the test. If the fluid in the left arm of the T.B. manometer rises, there has been a decrease in pressure and the observed interval must be added to the test values. If the fluid falls, there has been an increase in pressure and the interval must be subtracted.

Applications

The Warburg Respirometer can be used to measure oxygen uptake and oxygen uptake rate manometrically. The COD or BOD of the flask contents can also be checked before and after the test to determine organic removal, relating this to the oxygen utilization. One of the most commonly used applications of the Warburg is to check the amenability of either different wastewaters or various concentrations of one wastewater to biological degradation. This is illustrated in Figure 2, where a 5% solution of a given wastewater was determined to be degradaded biologically, where a 10 and 15% solution appeared to be toxic or inhibatory.

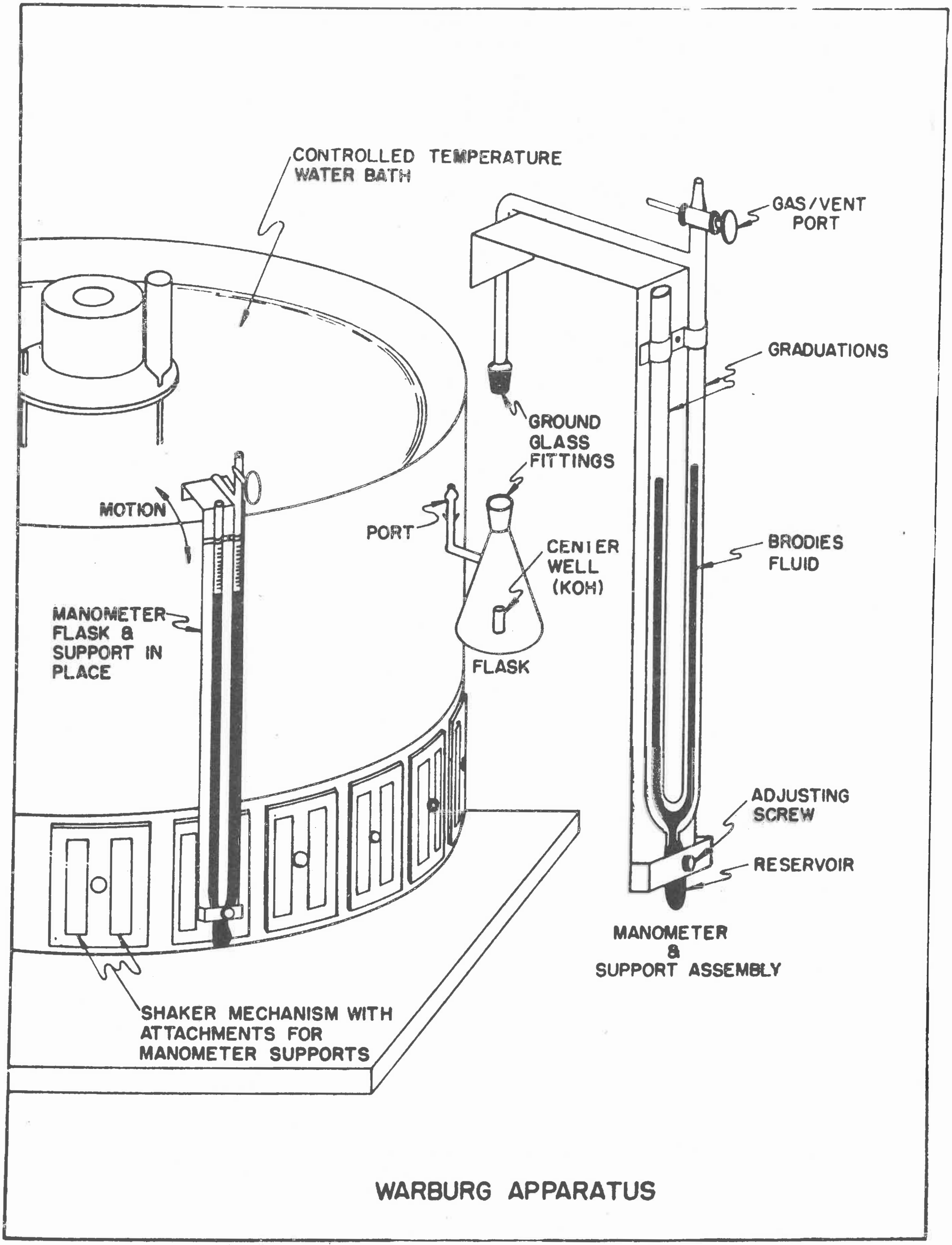


Figure 1 Warburg Respirometer

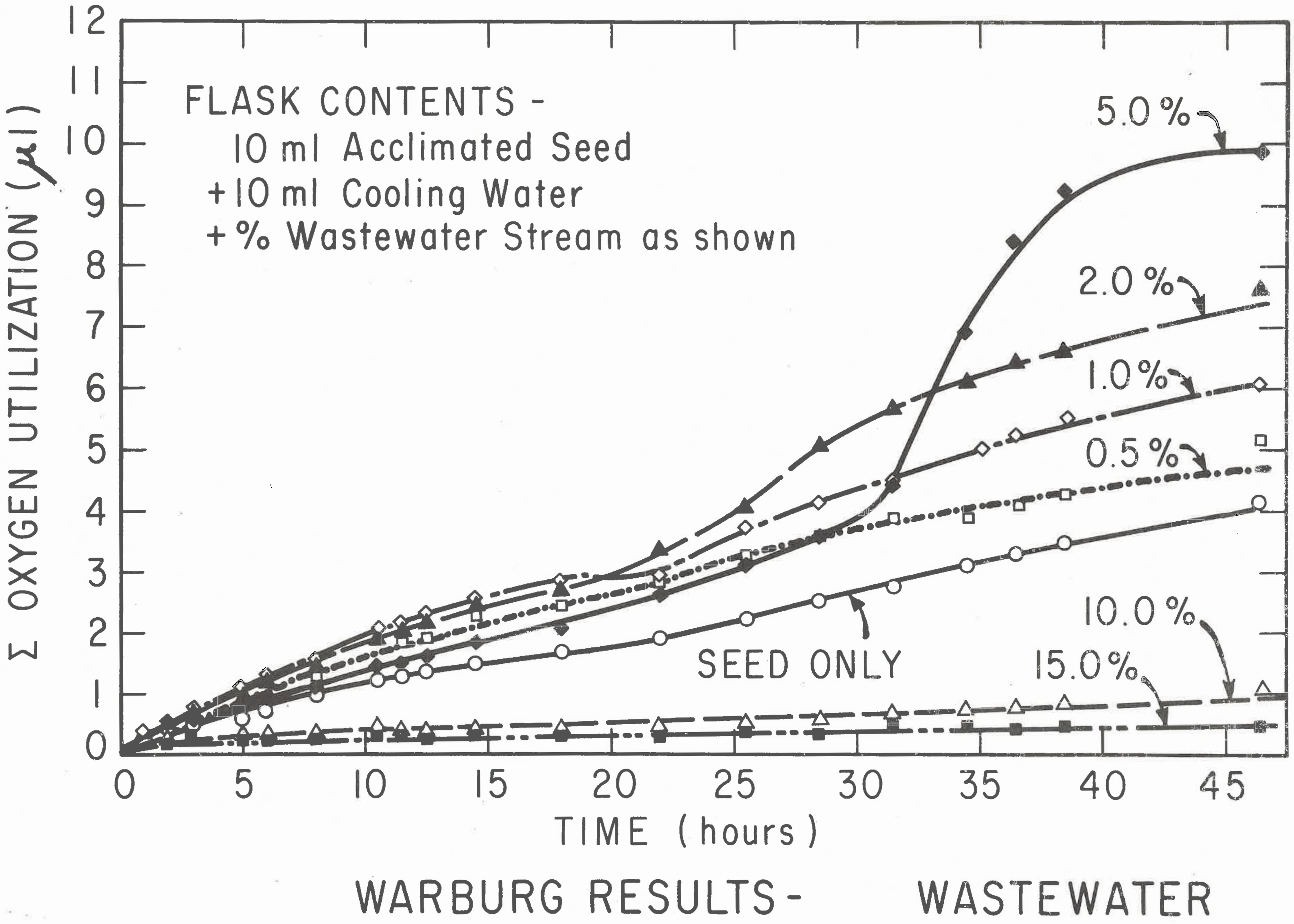


Figure 2 Warburg Applications

EXPERIMENT 5

DETERMINATION OF THE OXYGEN TRANSFER COEFFICIENT, ALPHA

Required Materials

Aeration tank
Sodium sulfite, cobalt catalyst
Oxygen probe, calibrated
Aeration device (mechanical or diffused)

Symbols

K_La = mass transfer coefficient, hr^{-1}

$\alpha = \frac{K_La \text{ wastewater}}{K_La \text{ water}}$

Reference p 35-37

Procedure

1. Fill the aeration tank with a defined volume of tap water.
2. Deoxygenate the tap water solution by purging with nitrogen gas or mixing with sodium sulfite. (Add approximately 8 mg/l of sodium sulfite per mg/l dissolved oxygen).
3. Reaerate the water using the aeration device. Record the dissolved oxygen concentration at various time intervals until at least 90% of saturation is reached. A calibrated oxygen probe or the Winkler Method can be used to determine the oxygen level.
4. Make a semi-log plot of the dissolved oxygen deficit (ordinate) versus the reaeration time (abscissa).
5. Evaluate K_La (hr^{-1}). This value is determined as follows:

$$K_La\ (hr^{-1}) = (\text{slope of plot, step 4})\ (2.31)\ \left(\frac{60 \text{ minutes}}{hr}\right)$$

(See Figure 17)

6. Repeat steps 1-5 using an equal volume of wastewater. Interferences of the wastewater with the Winkler Method of determining dissolved oxygen may necessitate the use of galvanic cell oxygen analyzer and probe.
7. If possible, repeat steps 1 through 6 for several mixing intensities. The mixing intensity can be established by varying the air flow or rotational speed of the aeration device.

Calculations

1. Show all calculations necessary for the determination of the tap water and wastewater $K_L a$.
2. Plot the oxygen transfer coefficient, alpha, as a function of mixing intensity.

EXPERIMENT 6

ACTIVATED SLUDGE, CONTINUOUS FEED
AERATED LAGOON, CONTINUOUS FEED

Required Materials

3 Lab-scale activated sludge units (see attached sketch, Figure 1)

Galvanic cell oxygen analyzer

Facilities for analyzing BOD, COD, TOC, suspended solids, organic and inorganic substances as required

pH meter

1 liter cylinder

Feed chemicals, reservoir containers, sigmamotor pumps

Reference pp 38-58 (See attached Figure 1)

Symbols

S_a = Substrate applied (COD, mg/l)
S_e = Effluent concentration (COD, mg/l)
X_a = Avg. VSS, mg/l
t = Detention Time, hrs, days
S_r = Substrate removed (COD, mg/l)
R_r = Oxygen Uptake rate, mg/l/day
K = Substrate removal rate, day^{-1}
a = Fraction of substrate removed used for cell synthesis
a' = Oxygen utilization coefficient for cell synthesis
b = Cell auto oxidation rate, day^{-1}
b' = Oxygen Utilization rate for auto-oxidation, day^{-1}

Instructions

Each group will be responsible for the operation of one activated sludge unit. A wastewater of known COD will be furnished. If the wastewater is deficient in nutrients (N and P) the required amounts should be calculated and added to the waste prior to starting the experiment. The activated sludge seed will be acclimated to the wastewater before the experiment begins. The organic loading schedule for each laboratory activated sludge unit is as follows.

Group I	≈ 0.1 lbs COD/day/lb VSS
Group II	≈ 0.6 lbs COD/day/lb VSS
Group III	≈ 1.2 lbs COD/day/lb VSS

Prepare the laboratory system in a manner similar to that illustrated in attached Figure 1.

Procedure

A. Start-up

1. Fill each unit with the seed sludge with the predetermined volume. Concentrate or dilute the sludge as required to have an initial VSS concentration of approximately 2400 mg/l.
2. Turn on the air and completely mix the tank contents. Adjust the sliding baffle to provide a 1/4" - 1/2" slot opening at the bottom. Subsequent baffle adjustment can be made as desired according to the sludge blanket height in the settling zone and the sludge interchange rate between the mixing and settling chambers.
3. Start the feed pump at a flow rate necessary to obtain the desired detention time. The applied wastewater should have a concentration of 250-300 mg/l COD except in the case of the highly loaded unit, where hydraulic considerations may necessitate using a concentration of approximately 500 mg/l.
4. Maintain the continuous flow-through systems until steady-state conditions prevail. This may be assumed once the oxygen uptake rate of the tank contents remains unchanged and the COD of the effluent is stabilized. Several detention times may be required before this occurs. Measure the VSS of the tank daily. (Plug the effluent tube, pull the baffle, and measure the VSS of the completely mixed contents. Once the sample has been withdrawn, reinsert the baffle, unplug the effluent tube and resume operations). Maintain a constant level of VSS by wasting solids daily equal to the daily buildup. It is suggested that solids be measured the first thing each morning, and the necessary volume wasted to bring the VSS concentration down to the predetermined level. It is possible that no solids will have to be wasted during the first few days of operation.

B. Steady-State Operation

Once steady-state operation has been assumed, the following sampling and analytical schedule is to be established:

Analysis	Frequency	Raw Waste[1]	Mixed Liquor[2]	Effluent[3]
COD, BOD, or organic carbon, mg/l (filtered and unfiltered composite samples)	3/week	x		x
pH	daily	x	x	x
SS, VSS, mg/l	3/week		x	x
Oxygen uptake, mg/l/day	3/week		x	
Dissolved oxygen, mg/l	daily		x	
Microscopic analysis (gram stain)	1/week		x	
Color, turbidity	3/week			x
Significant ions, cmpd., etc.	3/week	x		x

[1] Sample to be withdrawn from influent feed line or raw waste containers

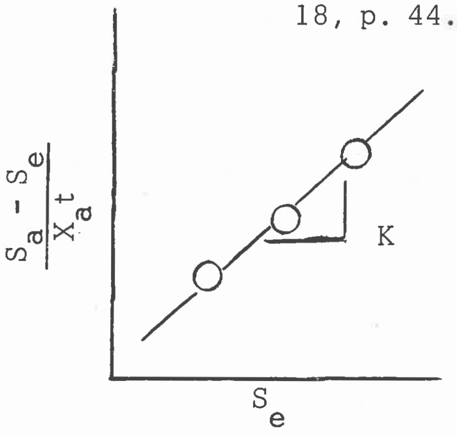
[2] Sample to be withdrawn from the unbaffled tank

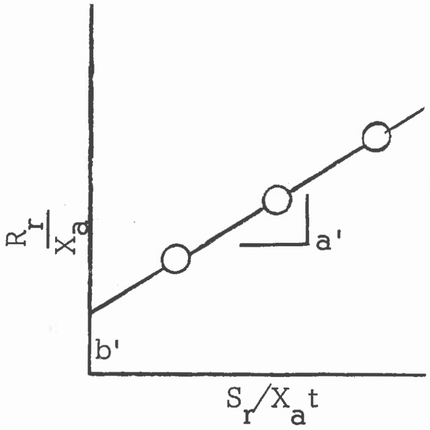
[3] Sample to be withdrawn from effluent bottle

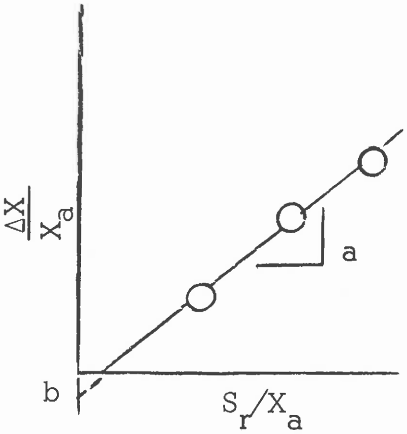
This program should be continued for at least two weeks, or until such time as consistant results are obtained. The data from each group will yield one average uptake rate, one average volume of daily sludge accumulation, and an average corresponding COD removal. The sludge settling curves and sludge volume index of the mixed liquor from each unit should be determined at the end of the experiment.

Calculations

1. Correlate these average values in the manner illustrated below:
   Note: All substrate values represent soluble (filtered) fractions only. A correction for total effluent BOD concentration is shown in Fig. 18, p. 44.







2. From these relationships, determine the necessary constants and mathematically express the COD removal rate, the daily oxygen requirements, and the daily sludge accumulation.
3. What effect did the organic loading have of the sludge settleability as measured by the interface settling velocity and the sludge volume index?
4. Is there any relation between the cellular morphology observed from the microscopic analyses and the organic loading?

AERATED LAGOONS*

Procedure

1. Fill each of the three units described in the activated sludge experiment with acclimated sludge. The sliding baffle is to be withdrawn, and the MLVSS of the total tank contents should be 150-250 mg/l.
2. Feed each of the three units at a flow rate necessary to obtain the organic loadings used in the activated sludge experiment. The volume of the tanks may be adjusted as necessary to obtain these loadings.
3. Determine the influent and effluent COD (filtered and unfiltered) and the MLSS and MLVSS daily. Additionally, measure the oxygen uptake rate of the aeration basin contents.
4. Continue the operation until the MLVSS and effluent COD concentrations become stabilized.

Calculations

1. Compare the efficiencies (in terms of COD removal) of the activated sludge process and aerated lagoon at similar organic loadings.
2. What was the equilibrium MLVSS concentration in each unit? What factors influence this level?
3. Relate, if possible, the lagoon detention time and the mg BOD/mg VSS ratio.

* This formulation represents a completely mixed, aerobic lagoon. Simulation of an aerated lagoon with solids deposition can be made by adjusting the air and mixing rates.

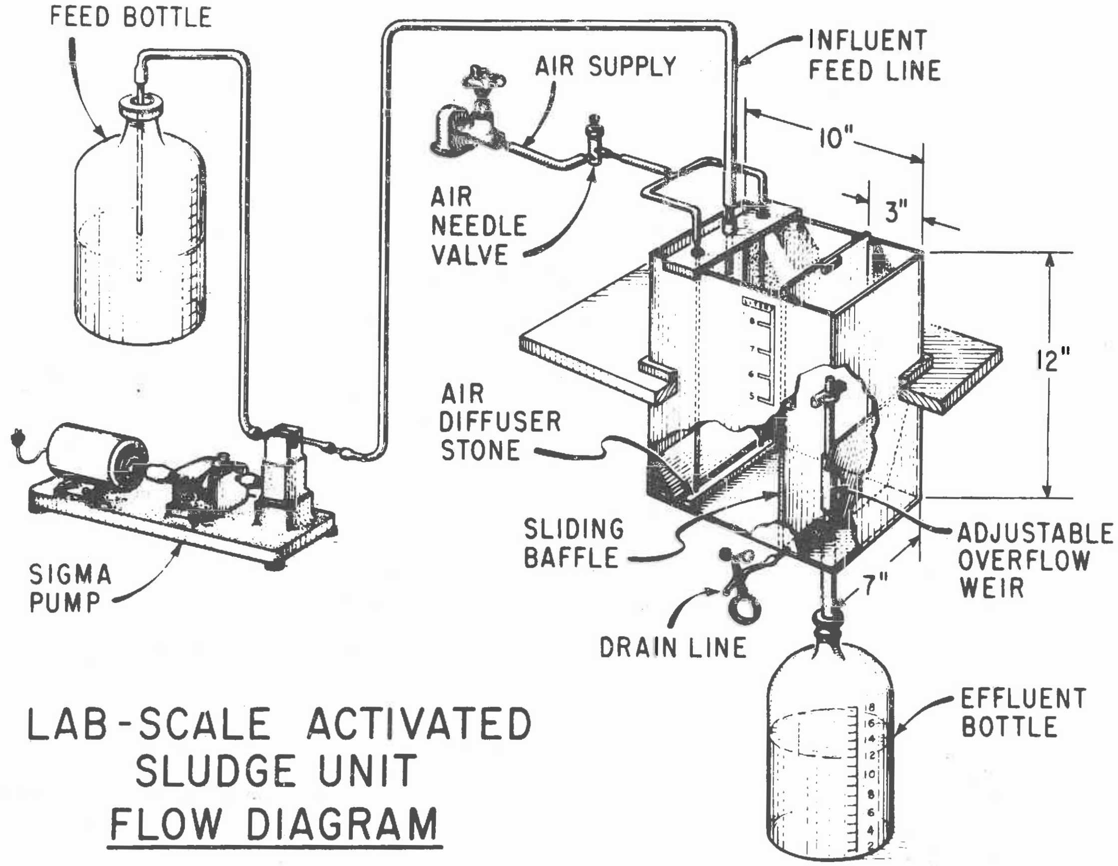


Figure 1 Activated Sludge Flow Diagram

EXPERIMENT 7

ACTIVATED SLUDGE - BATCH FEED

Required Materials

One Lab scale Batch Unit (5 compartments)
Galvanic Cell Oxygen Analyzer
Facilities for measuring BOD, COD, and VSS
pH Meter

Symbols

Same as Activated Sludge Experiment - Continuous Feed

Reference (See attached Figures 1 and 2)

Procedure

1. Partially fill each compartment of the batch unit with an acclimated biological seed. The amount of sludge added will depend on the COD of the wastewater to be analyzed, but the MLVSS concentration of each test cell should generally be 1500 - 2000 mg/l once the seed is mixed with the wastewater. Proportionate volumes of 750 ml seed and 1250 ml wastewater are commonly used.

2. Add wastewater to each of the batch units so that the COD of the mixed contents will be 6000, 4000, 2000, 500, and 200 mg/l respectively. The wastewater dilution requirements necessary to establish these levels can be calculated based on the seed sludge and wastewater filtered COD values.

3. Aerate the mixed contents for a period of 5 days. Measure the filtered COD concentration, oxygen uptake rate, and SS and VSS concentrations in each batch unit every 4 hours for the first day, every 6 hours for the second day, then every 24 hours for the remaining 3 days. Add distilled water as required to compensate for evaporation.

4. Plot the oxygen uptake rate, R_r, as a function of time for the first 24 hours of aeration. Determine the area under the curve for this period and report as total mg Oxygen used/mg avg. VSS for the 24 hour period. Record the corresponding COD removed during this time for each unit. A typical plot is shown in attached Figure 1.

5. Plot the MLVSS concentration versus aeration time for each of the 5 test units. Take the maximum observed increase in MLVSS, note the time required for this to occur, and plot this difference per average MLVSS as a function of COD removal.

6. Make a semi - log plot of the COD remaining (%) as a function of the average MLVSS and detention time for each condition. The removal rate, K, is taken as the slope of the line. A typical plot is shown in attached Figure 2.

Calculations

1. Correlate the data obtained in steps 4 and 5 in a manner similar to that described for continuous activated sludge tests. (See Experiment 6). Determine the coefficients a, a', b, and b'. How do they compare to those evaluated from the continuous activated sludge studies?

2. Evaluate the substrate removal rate, K, from Step 6 above. Does this rate vary with initial substrate concentration? Is this the same removal rate coefficient as defined in the continuous activated sludge studies?

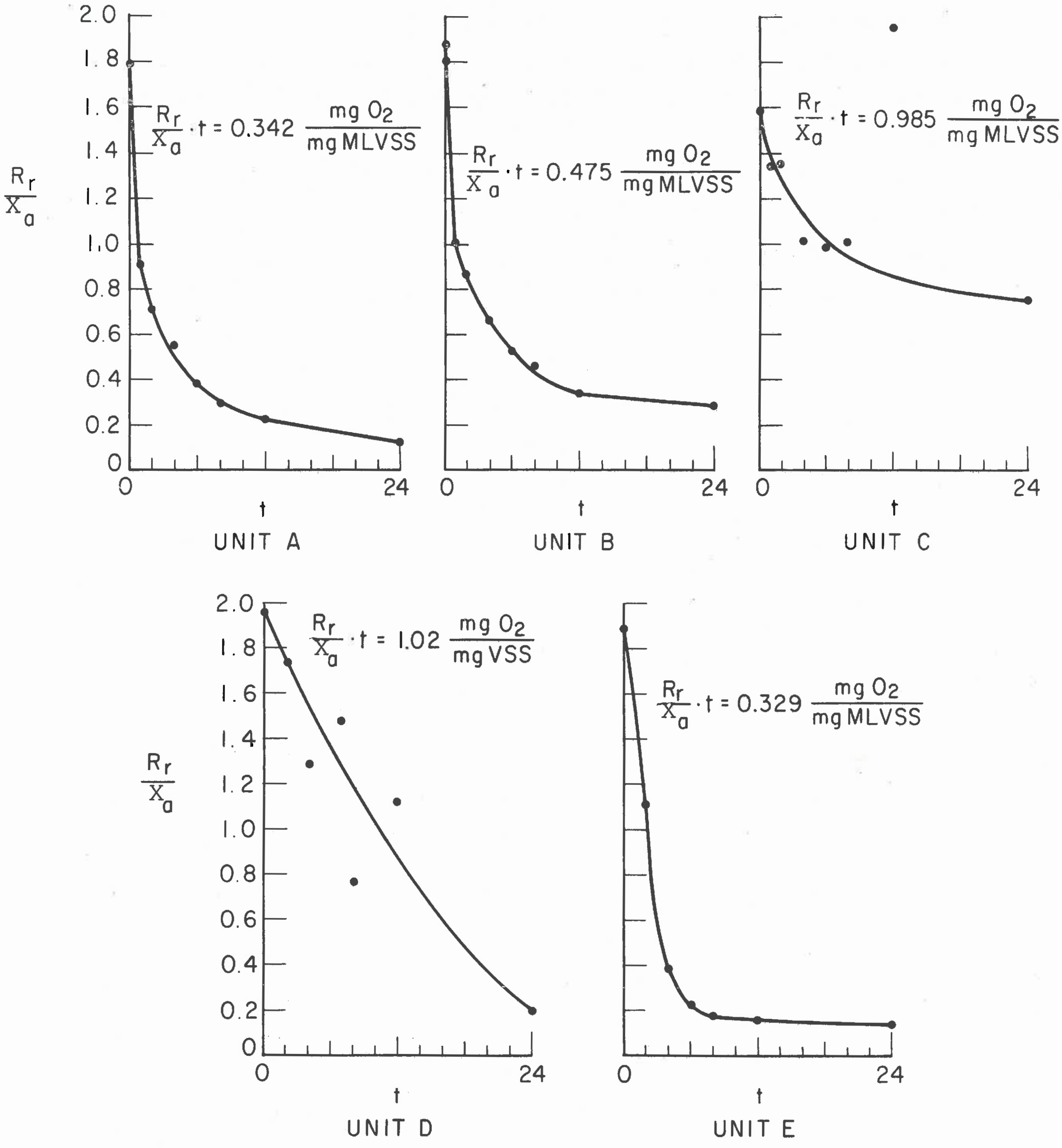


Figure 1 Oxygen Utilization With Time

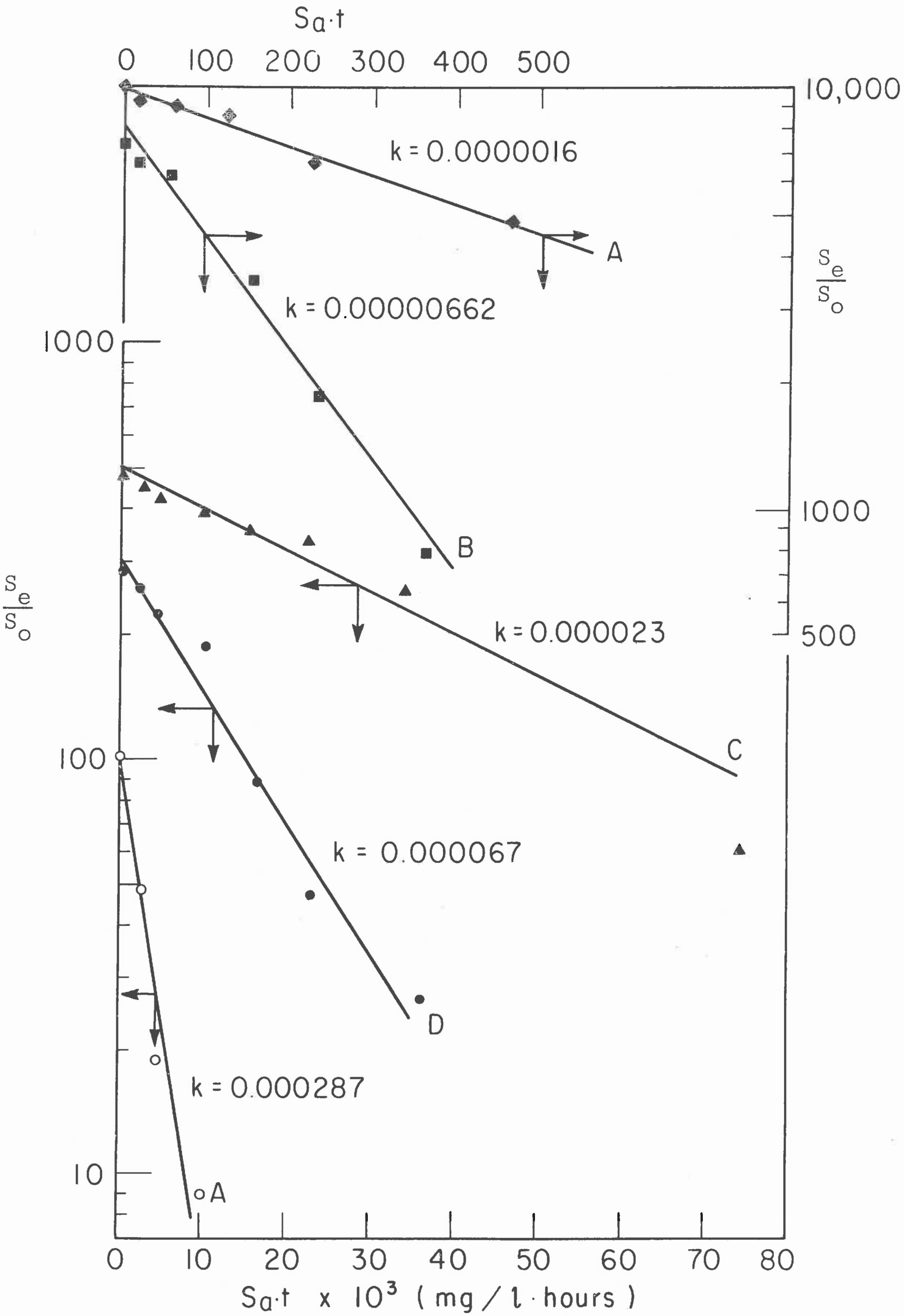


Figure 2 COD Remaining AS A Function Of The Biological Solids And Detention Time

EXPERIMENT 8

ANAEROBIC DIGESTION

Required Materials

- One anaerobic digestor (batch unit)
- Wastewater to be analyzed
- Nitrogen source, if required
- Phosphorus source, if required
- Gas collection apparatus
- Gas chromatograph, if available
- ORP meter
- Capability of analyzing for COD, alkalinity or acidity, solids

Symbols

S_o = initial substrate conc., mg/l

S_e = effluent conc., mg/l

X_a = volatile suspended solids, mg/l

a, K, b = constants

t = retention time

Reference pp 69-75 (See attached Figures 1 and 2)

Procedure

1. Obtain a sample of actively digesting sludge from a municipal wastewater treatment plant and immediately place the sludge in the laboratory digestor or fermentor. Add the wastewater in question, diluted if necessary, and maintain the mixed contents at a temperature of 35°C throughout the test period.
2. If significant amounts of air are entrapped in the digestor, purge the system with an inert gas, preferably helium.
3. The mixture should be mixed continuously either mechanically or recirculating the gas produced. Do not start feeding the waste or withdrawing the mixed liquor until gas production is noted. Be sure and check the entire system for gas leaks.
4. Once gas production is noticed, feed portions of the waste through the influent port, withdrawing an equal portion of mixed liquor through the effluent draw-off port. (See attached Figure 1). The feed rate should be approximately 200 lbs BOD/day/1000 ft^3 for a period of time not less than one hydraulic detention time. This allows for acclimation and system equilibration. It may be desirable to feed the units 2 or 3 times a day, maintaining the desired organic loading.

5. During the entire start-up period the pH should be closely observed, and if the pH falls to a level below 6.6, lime may be added to maintain the pH in the recommended range, viz., pH = 6.6 to 7.6. If it is not desirable to add lime, N_2HCO_3 may be used to adjust the pH and also provide alkalinity.
6. Continue to feed and withdraw daily until the effluent BOD or COD stabilizes. Analyze and record the following:

Raw Waste	Effluent (Mixed tank Contents)
pH	pH
COD or BOD	COD or BOD
	Alkalinity/acidity
	VSS
	Volatile acids

7. Record the volume of gas produced daily and analyze the CO_2 or CH_4 content of the gas if facilities are available.
8. Tabulate the data as shown in Table VIII - 1, 2, p. 74.
9. Determine the constants K, a, and b by plotting the data as shown in Figures 26 and 27.

Calculations

1. Determine the detention time and the VSS concentration based on the observed data. Assume a 90% of the removable COD reduction requirement.
2. Estimate the total gas and methane gas production.
3. Draw a sketch of a workable anaerobic treatment system designed to treat liquid wastewaters. Consider sludge recirculation.

Note. A continuous feed digestor can be assembled as shown in attached Figure 2.

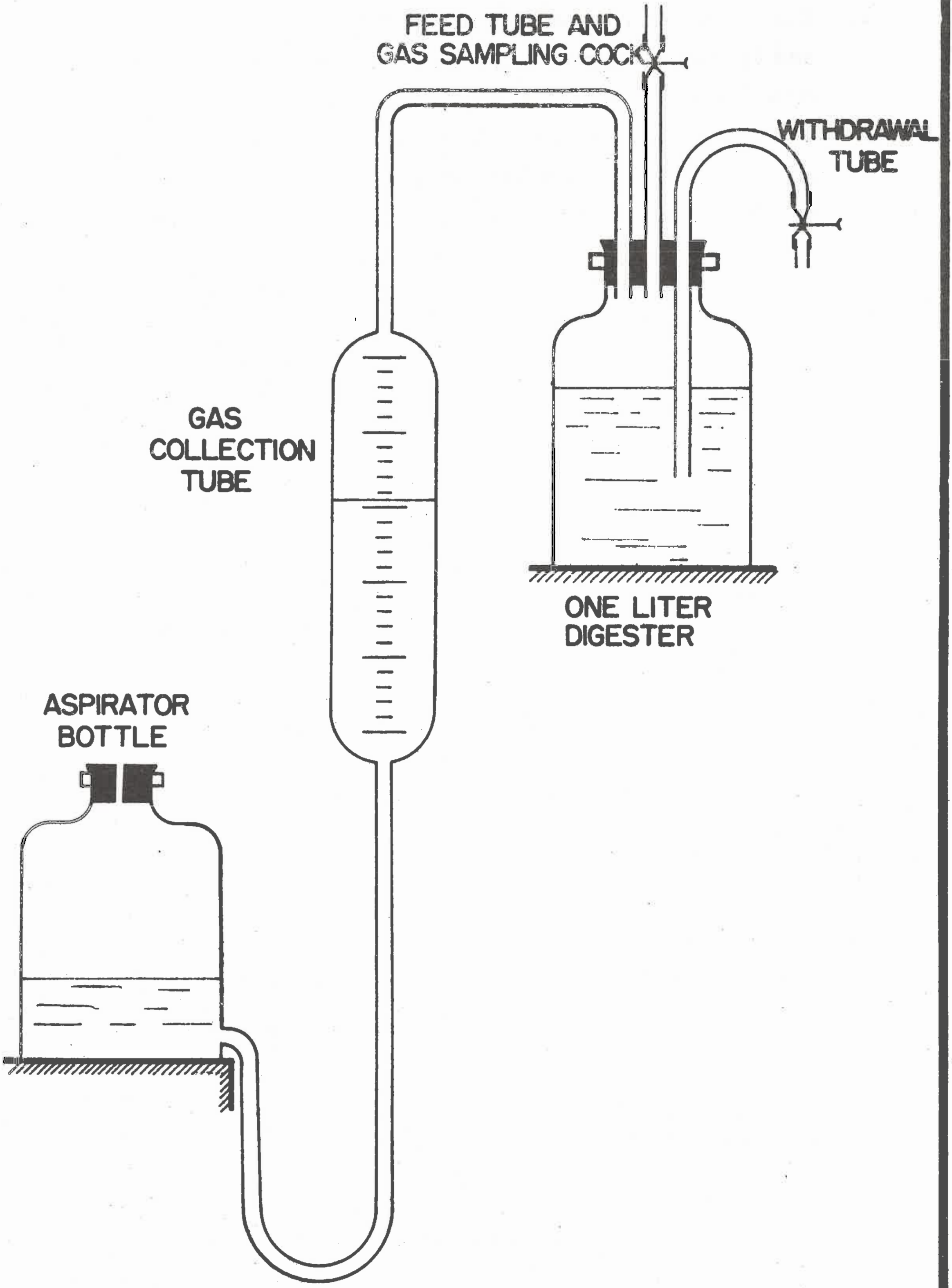


Figure 1.

SCHEMATIC LAYOUT OF ONE LITER BATCH FED DIGESTER

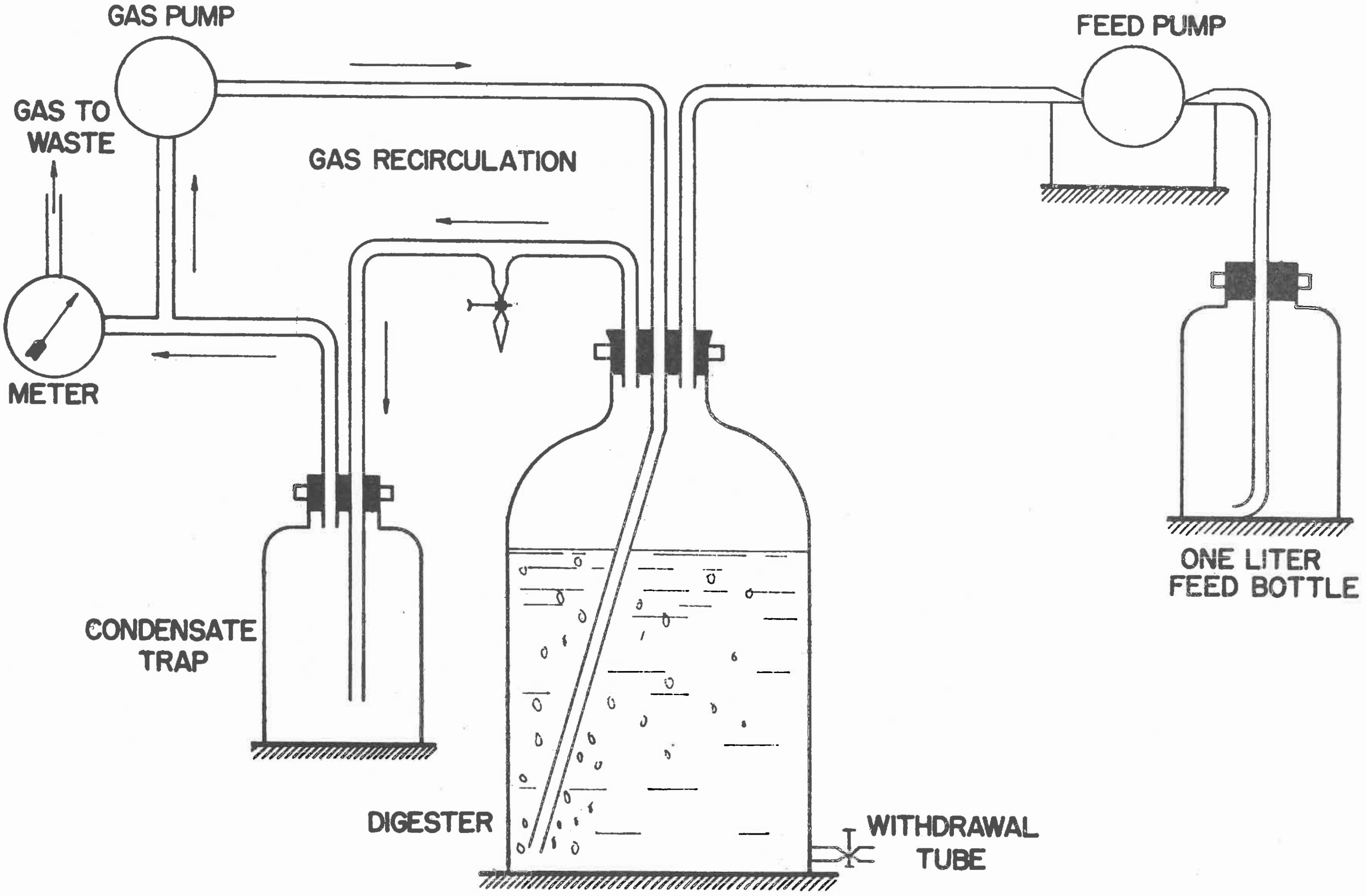


Figure 2.

SCHEMATIC LAYOUT OF CONTINUOUS FEED DIGESTER

EXPERIMENT 9

WASTE STABILIZATION PONDS

Required Materials

- Model pond, ≈ 45 liter capacity, properly baffled (see attached figure)
- Artificial lighting system (both incandescent and flourescent - 600 to 800 ft candle intensity above the pond surface)
- Light timer switch (12 hours on - 12 hours off)
- Pressurized air system to simulate wind effects
- pH - ORP meter
- Beaker, 2 liter capacity
- Facilities for analyzing COD, sulfates, sulfides, TKN

Reference (See attached Figure 1)

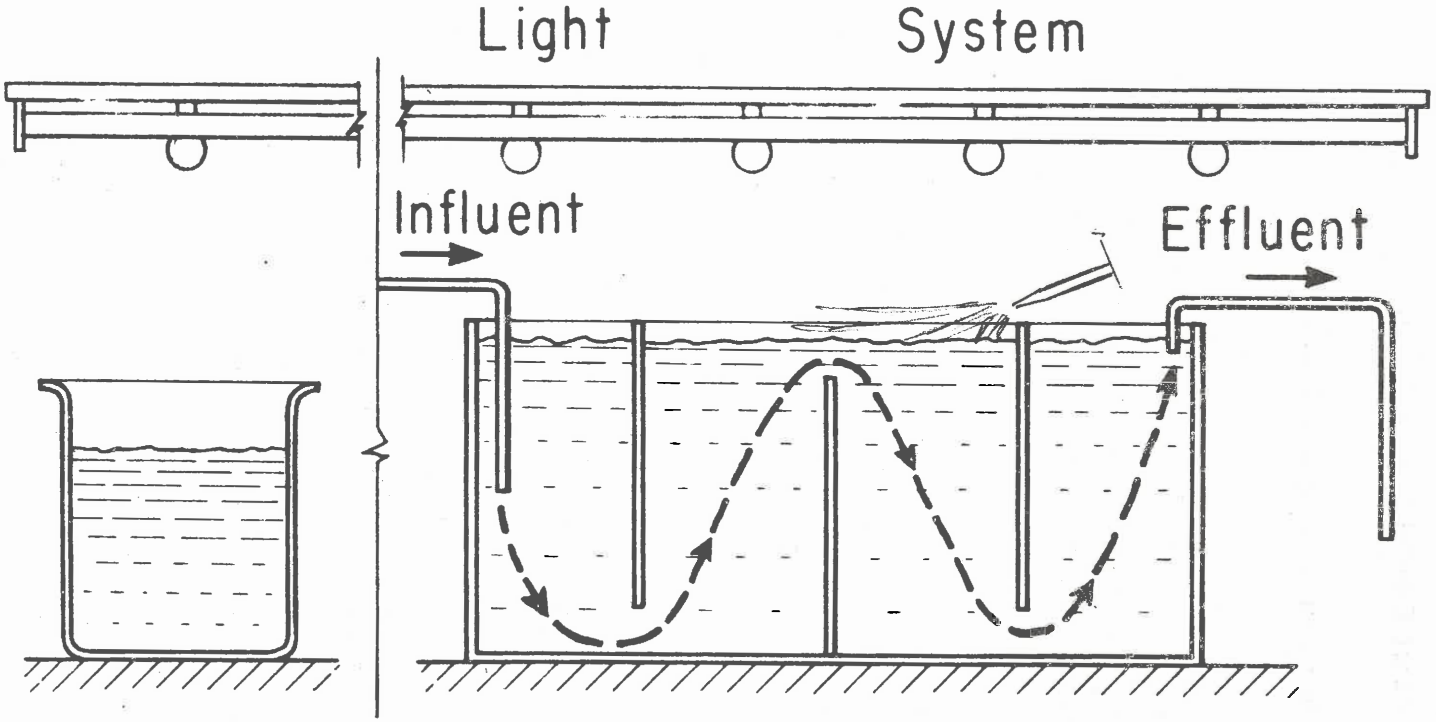
Procedure

1. Fill the continuous and batch model stabilization ponds with contents taken from a well functioning aerobic or faculative pond. Withdraw samples from each unit, filter, and measure the COD of the filtrate.
2. Apply the wastewater to each unit on a continuous and fill and draw basis respectively. Select a surface loading of 100 lbs COD/acre/day and a detention time of 20 days. Add tap water daily to each unit to compensate for evaporation. Provide an air jet just above the surface of both units to prevent scum formation and simulate wind action.
3. Withdraw samples from each unit daily and analyze for soluble COD of the effluent. The soluble COD is determined by filtering the sample. This effluent COD should stabilize following 1.5 - 2.0 detention times.
4. Measure the pH, ORP, and dissolved oxygen of the contents at the end of the "lights on" cycle. Record any periods when noticeable odors are prevalent. Also note any apparent changes in the color of the pond contents (transmittance using a spectrometer may be used to note any change).

5. If the raw wastewater contains sulfate ion ($SO_4^{=}$) in excess of 250 mg/l, check the sulfate and sulfide contents of the pond weekly. Also check the TKN (total Kjeldahl nitrogen) of the pond contents weekly.
6. Measure diurnal changes in O_2 (top and bottom).

Calculations & Report

1. Plot the effluent COD as a function of time (0 days = start-up time). When did the batch system become stabilized? When did the continuous system become stabilized? Compare the efficiencies of the two units at this point in terms of % COD removal.
2. Compare the ORP of the tank bottom to that of the top.
3. Discuss any pertinent aspects of pond operation based on the observed data. What factors constitute good waste stabilization pond design practice?



(b) Batch Model
Vol. = 3 liters
Area = 184 cm^2

(a) Continuous Model
Vol. = 45 liters
Area = 1500 cm^2

WASTE STABILIZATION POND MODELS

EXPERIMENT 10

ION EXCHANGE

Required Materials

Ion exchange column (1' diameter, 24' packed depth)

Sigmamotor pump and tubing

Anion resin

Reagents and equipment for chromium (Colorimetric method)

pH meter

5% NaOH

Deionized or distilled water

Graduate cylinder (100 ml)

Interval timer

Reference pp 76-83 (See attached Figure 1)

Ion exchange can be employed to remove undesirable anions and/or cations from waters. Cations are usually exchanged for hydrogen or sodium ions; anions are usually exchanged for hydroxl ions. In addition to water softening, applications include industrial use where either demineralized water or recovery of a specific ion is desired.

Objective

To determine the exchange capacity of a specific cation and anion resin for sodium dichromate.

Procedure

1. Assemble the ion exchange column as shown in the attached sketch.
2. Pack the columns with wet (after soaking in distilled water for 24 hours) regenerated anion. Connect the effluent part for backwashing. Backwash using a steady flow regulated by the Sigmamotor pump sufficient to fluidize and expand the bed about 50%. Drain. Note the height of resin material resettled.

3. Connect the column for normal operations. Using deionized water, readjust the Sigmamotor pump to a flow of 150 ml/min as measured at the effluent port. Rinse the column for at least 10 minutes or until the pH and the chrome content of the anion resin effluent are the same as the distilled water influent.
4. Prepare a known volume of $Na_2 Cr_2 O_7$, (125 mg/l of Cr^{+6}) sufficient for a 28 hour run at 50 ml/min. Determine the pH and exact Cr^6 concentrations.
5. Connect the influent line to the anion (chrome) source and begin the treatment cycle checking that the flow of the Sigmamotor pump remains at 150 ml/min. Sample at the effluent part every half hour until the breakthrough of chrome begins. Breakthrough will be considered 2 mg/l Cr^{+6}.
6. Connect the effluent port of the column for backwashing. Backwash with distilled water at about 50% bed expansion to eliminate fines and dirt. Be careful not to lose any resin. Use a 5-10 minute backwash period.
7. Reconnect the column for normal operation and set the Sigmamotor pump to deliver 6 ml/min using distilled water influent and measuring effluent at the effluent port.
8. Stop the pump, disconnect the distilled water line, and drain the column to within one inch of the top of the bed. Carefully pour the 5% NaOH regenerant solution into the column until the level is a little below the top of the tube. Then connect the pump to the regenerant reservoir and pump the entire contents through the bed at the rate of 6 ml/min. Collect the spent regenerant at the effluent port and measure the chrome content, the pH and the alkalinity of the composite effluent. The regenerant level will be specified by the instructor.
9. Rinse the volume with distilled water until chrome is no longer detected and until the pH is that of the distilled water influent. Collect this rinse volume and measure the chrome content, the pH, and the alkalinity of the composite.

10. Determine the ultimate exchange capacity by mixing the resins with chrome solution using a magnetic stirrer until equilibrium is reached.

Calculations

1. Compute the resin utilization and regenerant efficiency (use data from all groups for plot.)
2. Design an ion exchange system to deionize 150,000 gpd of water with a chrome content of 90 mg/l.

References

1. Standard Methods for the Examination of Water and Wastewater, 11th Edition, APHA (New York, N.Y. 1960) pp. 44 (alkalinity), 77 (chloride).
2. A Manual for Laboratory use of Permutit Ion Exchangers, the Permutit Company, Paramus, N. J. (1962).

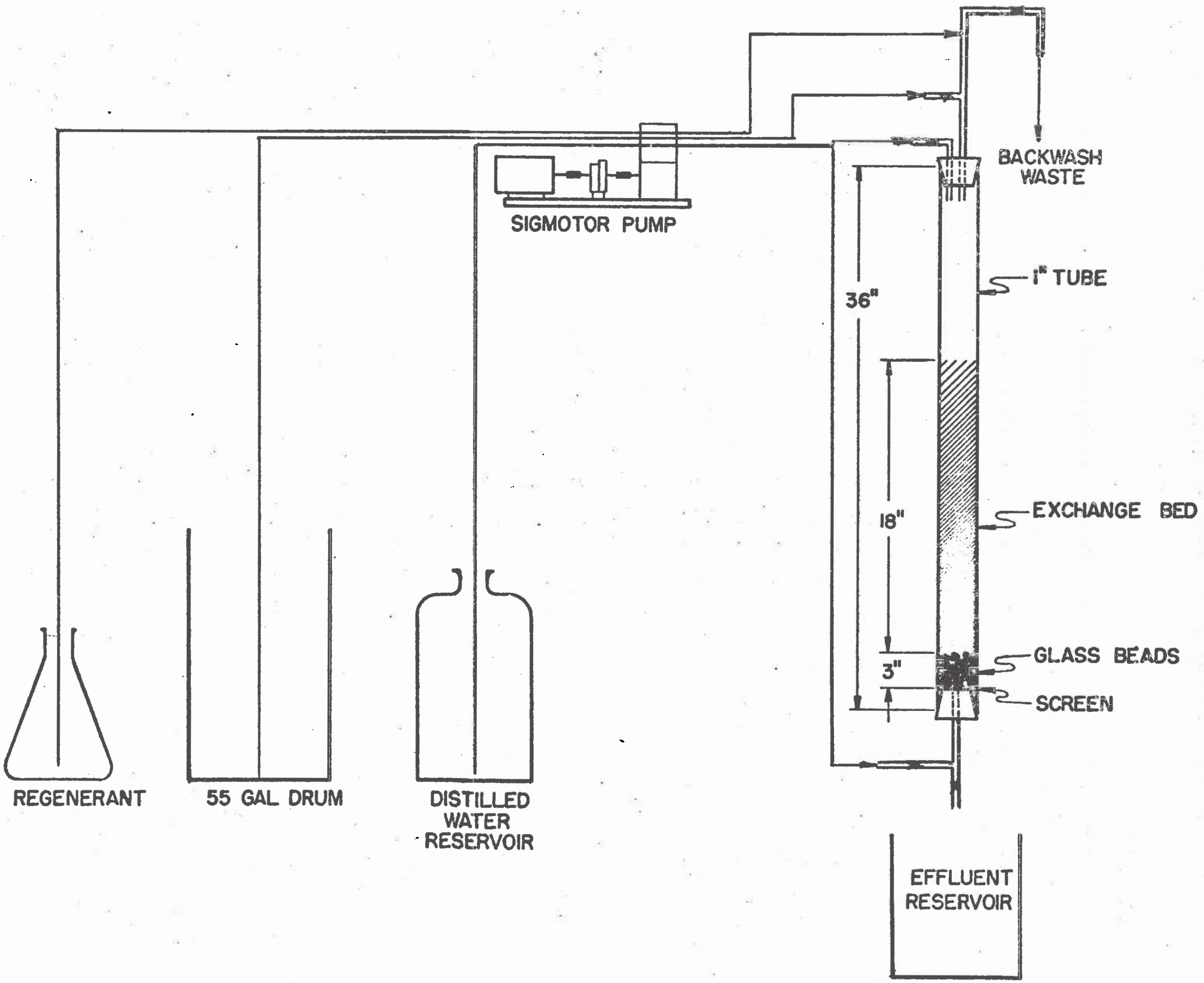


Figure 1 Ion Exchange Column

EXPERIMENT 11

ADSORPTION

Required Materials

- 6-Place shaker assembly
- Three activated carbon column packed to 2 1/2 ft. depth using granular activated carbon
- Activated carbon (3-liter beaker) pre-soaked (24 hours) in distilled water
- 2-4-D solution stock or other wastewater as desired
- Six 1500 ml beakers
- Six 250 ml erlenmeyer flasks
- Six funnels and Whatman No. 1 filter paper
- Tared aluminum dishes
- Sample bottles
- 500 ml graduate cylinder

Symbols

$\frac{x}{M} = \frac{\text{wt. of material adsorbed}}{\text{wt. carbon}}$

C_o = initial waste concentration mg/l

C_e = equilibrium concentration mg/l

A, p = constants

X_o = critical column depth

N_o = adsorptive capacity

K = rate constant

X = bed depth

Reference pp 84-91 (See attached Figures 1 and 2)

Adsorption may be used to remove organic materials from a soluble waste. Design data may be developed by passing a known concentration of the organic material through a known mass of activated carbon and observing the removal.

Procedure

A. Batch Test

1. Take the activated carbon, soak in distilled water for 24 hours, and oven dry at 103°C for 24 hours. Pulverize with a mortar and pestle until the powdered carbon passes a 270 mesh seive (0.053 mm opening)
2. Add various amounts of the powdered carbon to each of the test flasks. As a general rule, the ratio of initial waste COD (mg/l) to carbon concentration in the beaker (mg/l) should range from .05 to 5.0. This may vary, depending on the COD of the wastewater.

3. Add appropriate volumes of the wastewater (filtered to remove suspended material) to each of the beakers and start the shaker assembly.
4. Measure the COD of the filtrate from each flask every 15 minutes, until the equilibrium COD of the contents has been obtained (The COD remains unchanged with time, indicating complete adsorption).

Calculations

1. Plot COD as a function of time for each test flask. Determine the Freundlich isotherm and write the expression. (See example, Figure 1).
2. Based on the initial COD of the wastewater, determine the adsorption capacity of the carbon (lbs COD removed/lb carbon).

B. Continuous Test (if applicable)

1. Pass a known concentration of wastewater through a known mass of activated carbon, contained in a 4 ft. glass column with a 15/16" inside diameter. Each column should have 2 ft. of carbon resting on 6" of spun glass wool. (Use different depths if time permits). At least 3 different flow rates covering the range desired should be employed. The test columns are shown in attached Figure 2.
2. Measure the influent and effluent COD, and monitor each flow rate.

Calculations

1. Correlate the carbon column data from all groups according to the Bohart-Adams equation. Develop this data in conformance with the format as shown on pp 85 and 88.

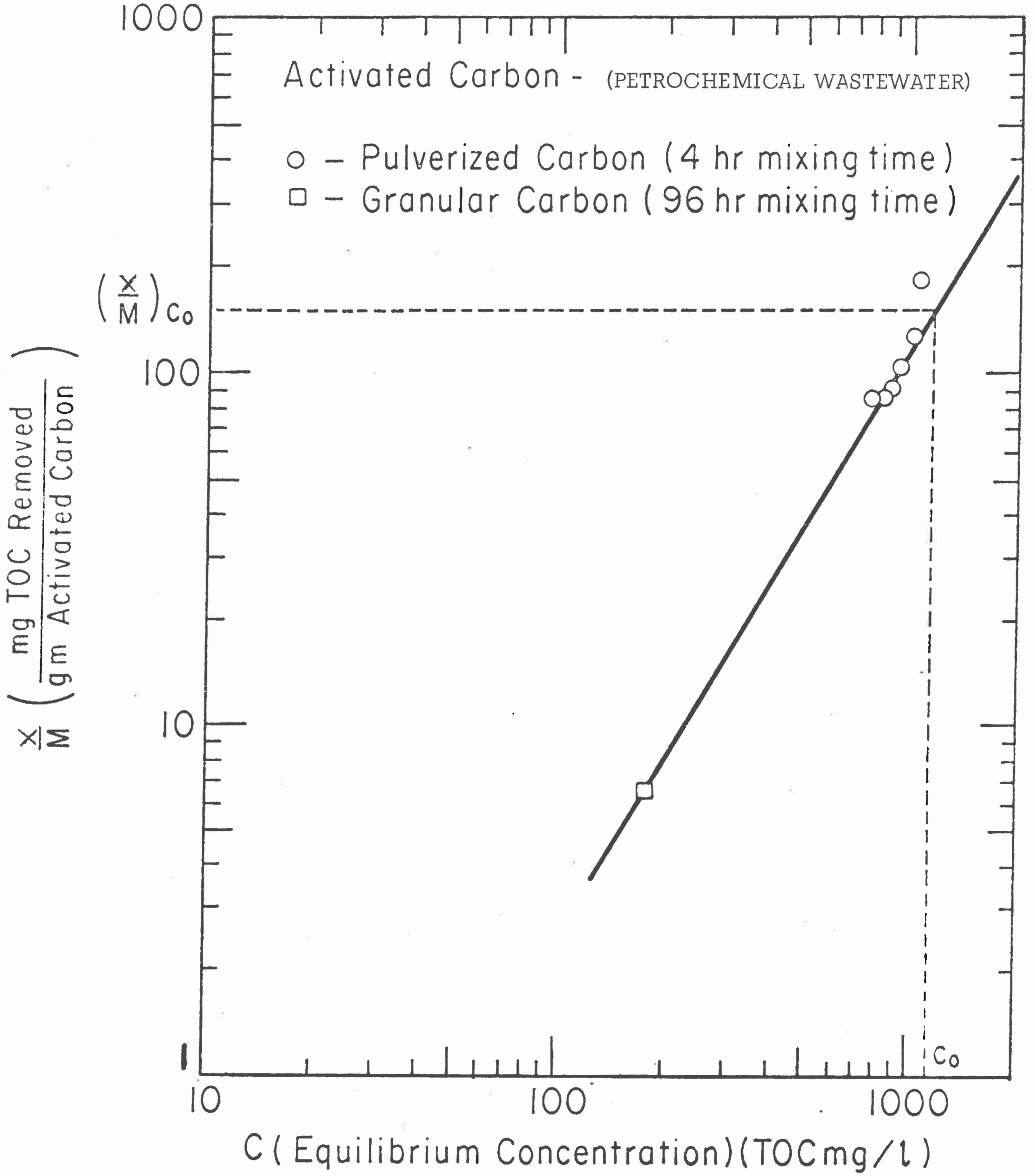


Figure 1 Adsorption Isotherm

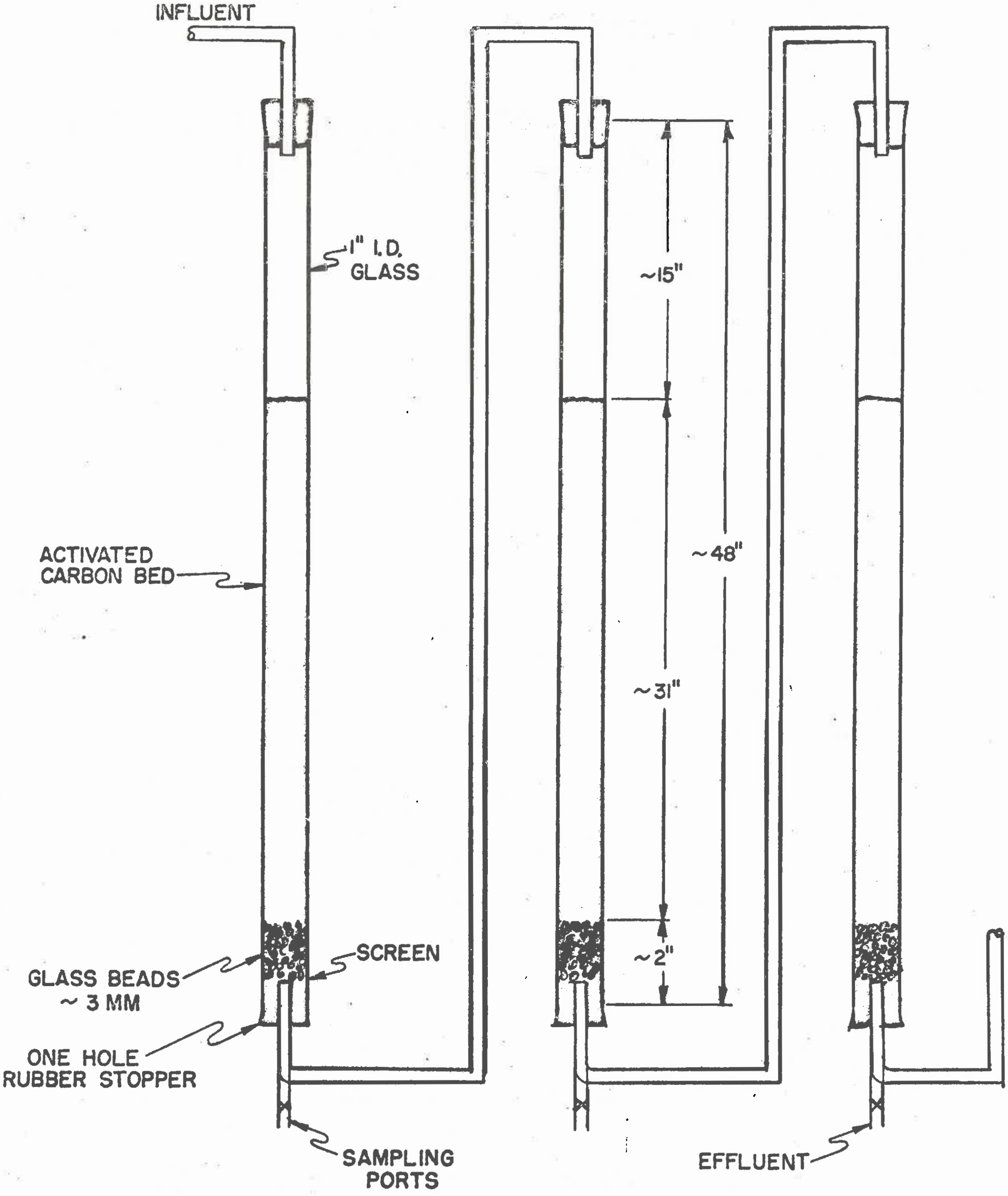


Figure 2 Activated Carbon Columns

EXPERIMENT 12

SLUDGE THICKENING

Required Materials

8' Thickening column (if available) w/stirring mechanism (See attached Figure 1)

Facilities for measuring suspended solids

Stirred liter cylinders (3)

Reference pp 92-102 (See attached Figure 1)

Figs. 34-39

Symbols

UA = unit area, ft^2/lb solids·day

ML = mass loading, $\frac{1}{UA}$ lbs solids/day/ft^2

C_u = underflow concentration, lb/ft^3

C_i = solids concentration at settling velocity, U_i, lb/ft^3

U_i = interfacial settling velocity, ft/day

C_o = initial solids concentration

C_∞ = ultimate solids concentration at infinite time

H_i = height of interface at C_i

H_∞ = height of interface at C_∞

n, K_b = sludge coefficients

K_T = scale-up coefficient

Procedure

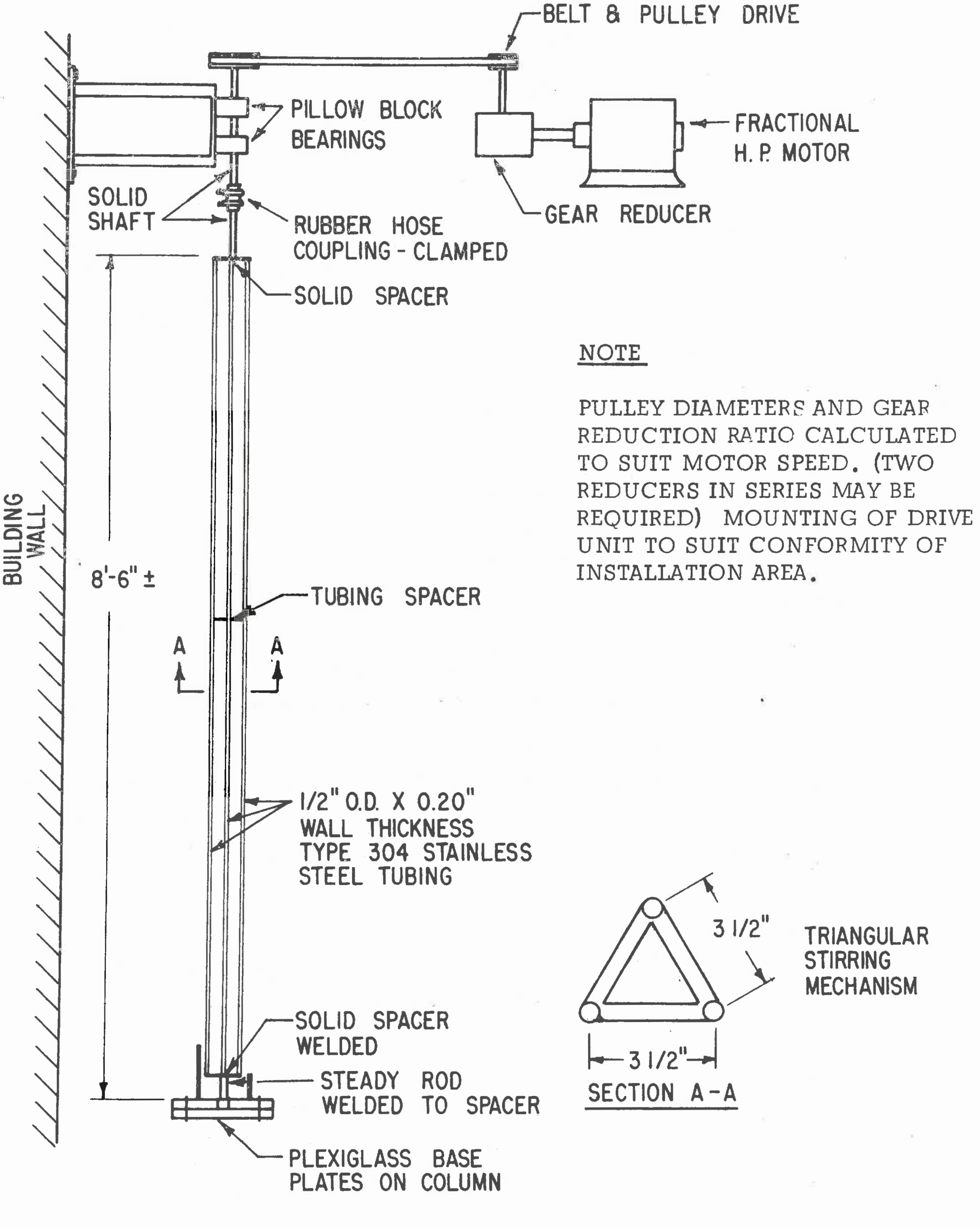
1. Develop a series of batch settling tests for the range of influent solids concentrations, C_o, as might be expected in full scale operations. Begin by filling the test cylinder with the test sludge and thoroughly mix using aeration or mechanical mixing. Make certain that the air flow rate is adjusted so that the turbulence will not alter the settling characteristics of the sludge. After 10, 20, 30, 60, 120, & 240 minutes, withdraw samples from all taps of the 8 ft. column and measure suspended solids. Record the interface height with time.*
2. Repeat step 1 for each C_o value which may be necessary to describe the anticipated range. Stirred liter cylinders alternately may be used to develop the necessary settling curves. Record the temperature at the beginning and end of each run.

* Disregard the last two sentences of Step 1 if an 8' settling column is not available.

Calculations

1. From the settling curves, determine the unit area, underflow concentration, and settling velocity relationship (See Figure 34)
2. Evaluate K_b and n for each C_o by correlating $\frac{C_u}{C_o}$ -1 and mass loading.
3. Plot K_b as a function of the initial solids concentration.

   $K_T \approx$ 0.5 - 0.9 K_b, depending on sludge depth. (See Fig. 34, Step 6)
4. Calculate the mass loading, ML , for the design initial concentration and desired underflow concentration, using the determined K_T and n values.
5. Knowing the solids loading in the waste, calculate the thickener area requirement. Sketch a layout.
6. From the solids data (tap samples) plot the concentration gradients in the settled sludge mass at each of the various times (if applicable).



NOTE

PULLEY DIAMETERS AND GEAR REDUCTION RATIO CALCULATED TO SUIT MOTOR SPEED. (TWO REDUCERS IN SERIES MAY BE REQUIRED) MOUNTING OF DRIVE UNIT TO SUIT CONFORMITY OF INSTALLATION AREA.

Figure 1 Stirrer Mechanism For Settling Column

EXPERIMENT 13

VACUUM FILTRATION

Required Materials

Filter leaf and vacuum assembly (See attached Figure 1)

2-liter beaker and stirrer

$FeCL_3$, lime and polymers

Buchner funnel assembly (See attached Figure 2)

Symbols

t = time, retention time or cycle time, min.

v = volume of filtrate, ml

R_o = specific resistence, sec^2/gm

s, n, m = constants

Reference pp 103-113 (See attached Figures 1 and 2)

Procedure

1. Prepare the Buchner funnel using a wire screen and filter paper.
2. Wet filter paper and adjust vacuum.
3. Add 200 ml sludge to Buchner funnel.
4. Record the filtrate volume until vacuum breaks.
5. Measure the initial solids content.
6. Repeat 1-5 using varying concentrations of coagulant and polymer.
7. Compute the specific resistance on all samples from the slope obtained by plotting t/v vs v. Determine the optimum coagulant dosage.
8. a. Using optimum results from 1-7 above, determine filter loading, using a leaf test. Vary the submergence time.
   b. Compare the specific resistance computed from the leaf test and the Buchner funnel test.
   c. Determine the following: R_o, $1-s$, m, n
   d. Develop the relationship between feed solids & cake thickness; feed solids, cake thickness, and drytime.

Calculations

Design a vacuum filter to dewater 25,000 lbs of sludge at 3% solids. Filter operation will be 98 hours per week.

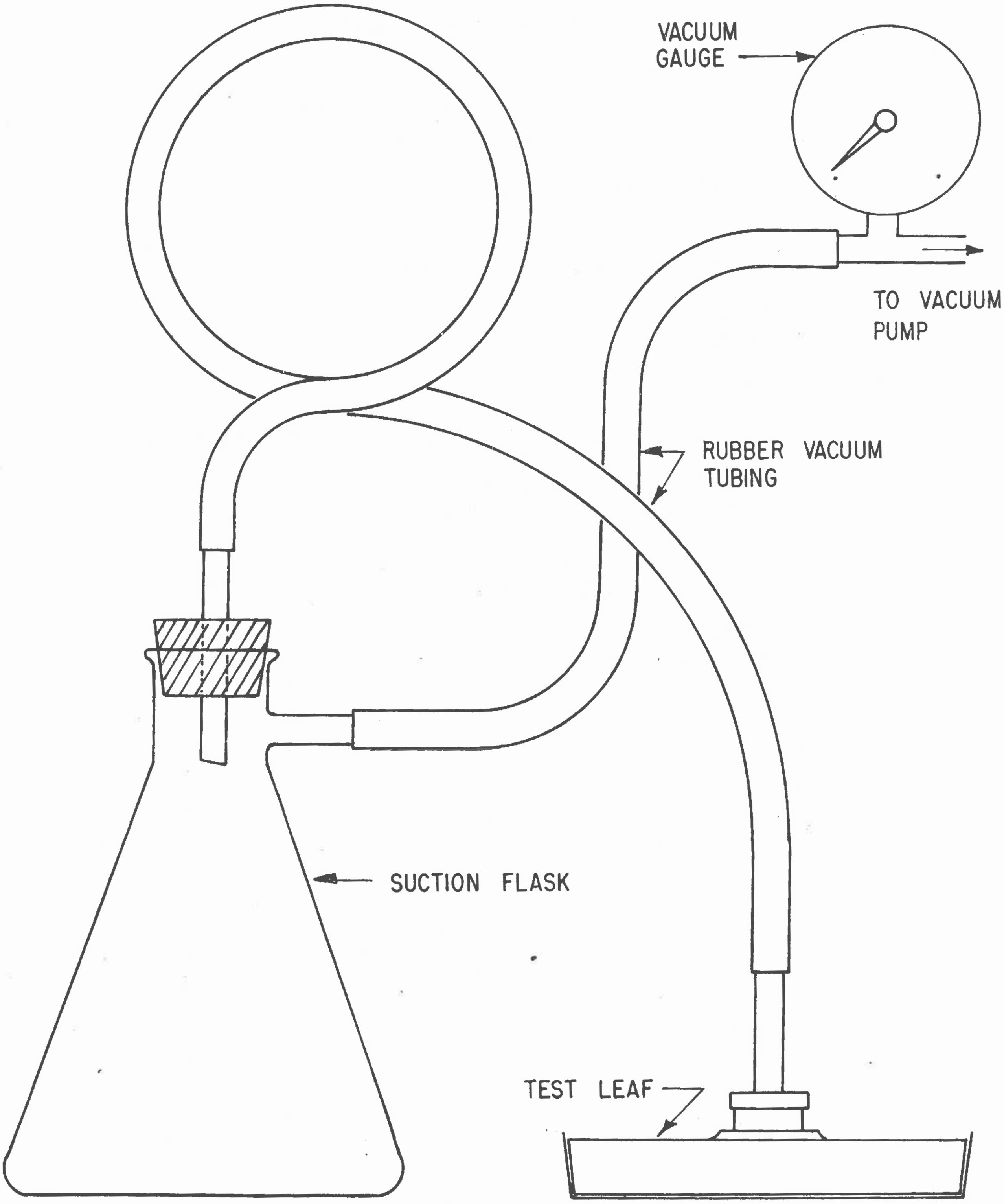


Figure 1 Leaf Test Apparatus

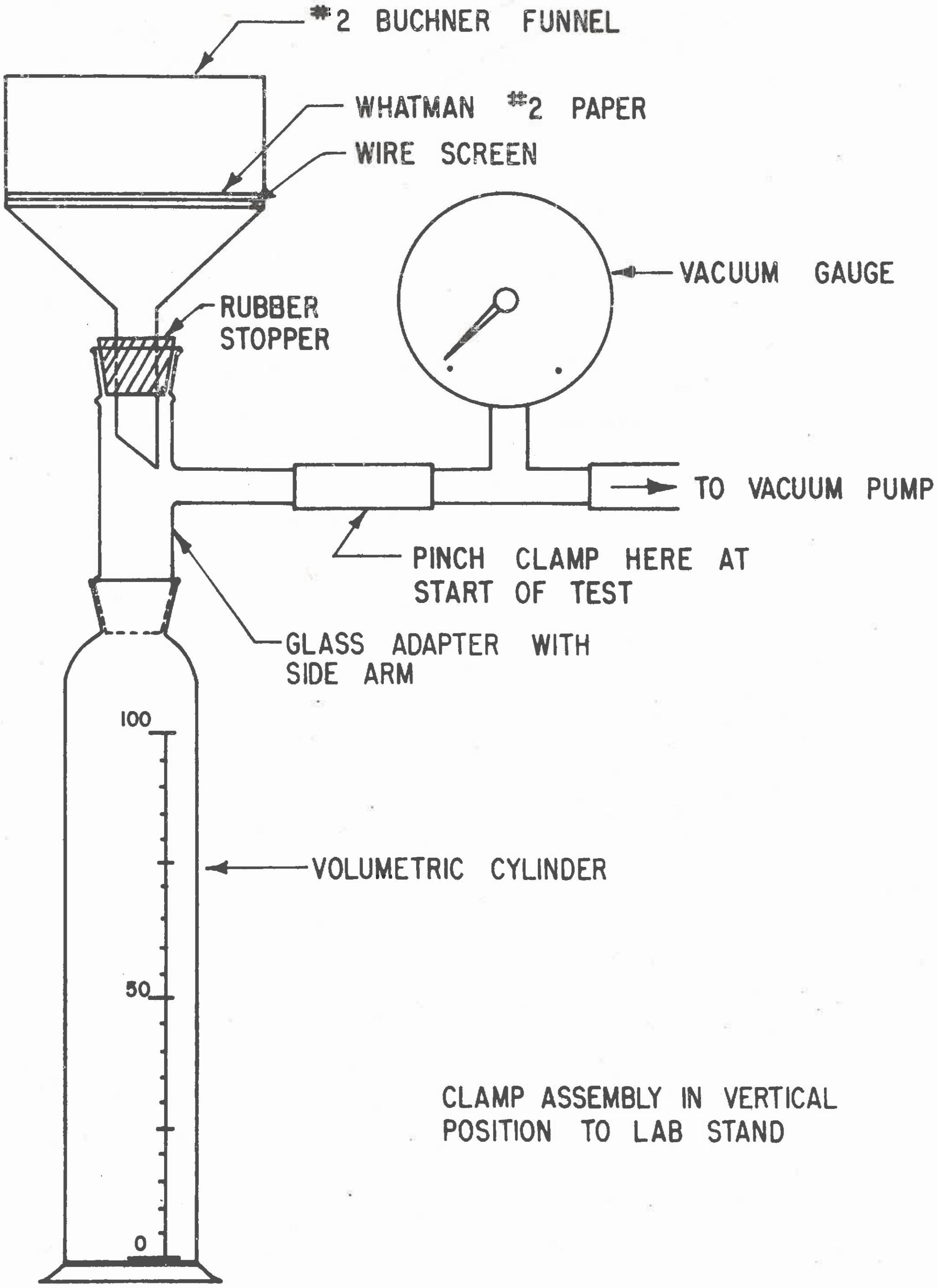


Figure 2 Buchner Funnel Apparatus

EXPERIMENT 14

COAGULATION

Required Materials

Colorimeter
pH meter
6 place stirrer
Beakers, 1500 ml, 6
1% ferrous sulfate solution
Coagulant solution as per instructions
1 N NaOH
1 N H_2OSO_4
Polymers

Appropriate glassware and stirring apparatus
Water sample
Interval timer

Chemical coagulants are used to remove colloidal particles from water and wastewaters. For each coagulant and for each water the optimum pH and minimum dosage required may be determined by laboratory studies.

Procedure

1. Select the proper operation wave length for evaluation of the color of the water using the colorimeter. Determine the % transmission. If a waste is being used determine the COD.
2. Determine the approximate minimum dosage for which a floc will be formed in water. This may be accomplished by slowly stirring 200 ml of the raw water in a beaker and adding the coagulant in 1 ml increments until the first evidence of a floc is noted (pH is not to go below 6; adjust if necessary). Repeat test using other coagulants if time permits.
3. Using the approximate coagulant dosage determined above, make up 1 liter samples of the water or wastewater to be treated. Adjust the pH in six increments of one pH unit from pH 4 to pH 9.
4. Mix the samples rapidly for two minutes and then flocculate slowly for 15 minutes. The rate of flocculation should be such that sheering of the floc will not occur. Record the time of formation of a visible floc in each sample. After flocculation, allow all samples to settle and measure the final pH of the supernatent.

5. Determine the residual color as % transmission or COD in each sample. Plot % transmission or COD vs pH to determine optimum pH.

6. Add 1 liter samples of the colored water or wastewater in each of six 1500 ml beakers. An amount of coagulant is then added to each sample so that the concentrations vary from 25% to 200% of the concentration determined in Step 2.
Example: Approximate concentration 200 mg/l

Trial Concentrations mg/Coagulant	ml Coagulant (Using 1.% Solution)*	ml Coagulant to obtain appropriate pH
50	5	
125	12.5	
175	17.5	
225	22.5	
300	30.0	
400	40.0	

*If the volume of solution required is more than 5% of sample, then a more concentrated stock solution should be used. The varying amounts of coagulant will result in a variation of pH in the samples which should be adjusted. In preliminary runs this adjustment is often omitted.

7. Determine the amount of lime needed to obtain the approximate pH found in Step 5 above.

8. Discard the samples, then add the determined amounts of coagulant and lime to new 1 liter samples.

9. Record the pH of each sample.

10. Mix the samples rapidly for 1 minute and then slowly for 15 minutes. The rate of slow mixing should be such that shearing of the floc will not occur. Record the time of formation of a visual floc in each sample.

11. Add polyelectrolyte at the end of the rapid mix period. Allow the samples to settle and measure the % transmission or COD of the clear liquor.

12. Determine the optimum dosage.
13. Record the settled sludge level and sludge concentration.

Calculations

1. Determine the pounds of chemicals required per gallon of water or wastewater, based on the optimum pH and coagulant dosage.
2. Determine the pounds of sludge produced per gallon of water or wastewater.

SECTION 2

Design Procedures for Wastewater Treatment Processes

Chapter I.

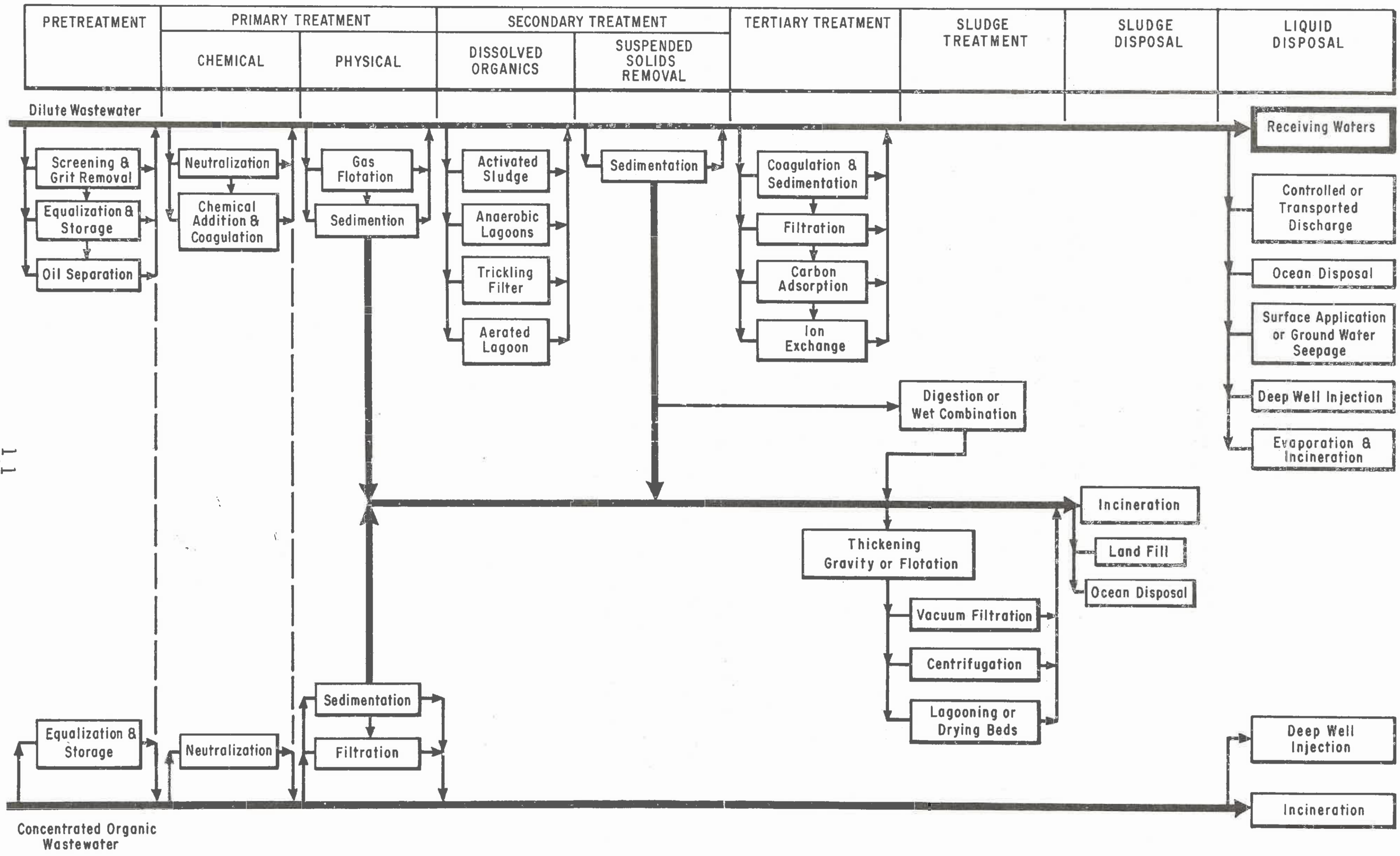
WASTEWATER TREATMENT PROCESSES

The various wastewater treatment processes in common use today for the treatment of municipal and industrial wastewaters are shown in Figure 1. Treatment alternatives for both concentrated and diluted waste streams are shown, including sludge thickening, dewatering, and disposal alternates.

Figure 2 has selected the most common processes and also the mathematical models available for process design where applicable and the effluent qualities that might be expected from an optimal designed process. The symbols used in Figure 2 and throughout the text unless otherwise noted, are detailed in Table 1.

The constants and coefficients for the models are summarized in Table 2. These figures have been developed for wastes from the chemical industries; coefficients and constants for wastes from other industries are now in the process of development.

FIG. I WASTEWATER TREATMENT SEQUENCE / PROCESSES SUBSTITUTION DIAGRAM



Chapter II.

PRIMARY SEDIMENTATION

Procedure for the Design of a Primary Settling Tank

A. Data to be collected

1. Flow and characteristics of the wastes such as suspended solids concentration, BOD (if organic), temperature.

B. Design Procedure

1. Develop the settling rate-time relationship curves for at least three different concentrations of suspended solids to cover the expected range of fluctuation in suspended solids concentrations (Figure 3 a).

2. The overflow rates and detention times for various % removals are computed from the above curves as follows: The overflow rate or the settling velocity V_o is the effective depth (six feet) divided by the time required for a given % to settle through the effective depth. All particles having a settling velocity equal to or greater than V_o will be removed in an ideal basin. Particles with a lesser settling velocity V, will be removed in proportion V/V_o. For a given settling depth, and a detention period t, min, a certain % of the suspended solids are removed completely (Figure 3 a). Particles in each additional 10% range will be removed in the proportion V/V_o or in proportion to the average depth settled to the total settling depth. Each subsequent % range is computed in a similar manner and the total removal determined as below.

$$\text{Total \% removal} = x + \frac{d_1}{d_o}(10) + \frac{d_2}{d_o}(10) + \frac{d_3}{d_o}(10)$$

FIGURE 2 WASTEWATER TREATMENT PROCESSES & MODELS

PRE OR PRIMARY TREATMENT

- NEUTRALIZATION
- EQUALIZATION
- OIL SEPARATION
- SEDIMENTATION
- AIR STRIPPING
- AIR FLOTATION
- COAGULATION & SEDIMENTATION

BIOLOGICAL TREATMENT

INFLUENT CHARACTERISTICS: SUSPENDED SOLIDS < 125 PPM, ALKALINITY < 0.51 lb/lb BOD, SULFIDES < 250 PPM, OIL & GREASE < 50 PPM, BOD VARIATIONS ⩽ 3:1 (lb/day, 4-hr COMPOSITE)

ACTIVATED SLUDGE (COMPLETELY MIXED)

$\frac{\Delta_e}{\Delta_0} = \frac{1}{1 + k x_v t}$

$\Delta X_v = f(VSS) + aS_r - bX_v$

$lb\ O_2/DAY = a'S_r + b'X_v$

$F = \frac{S_a}{X_{v(a)}}$

EXTENDED AERATION

$X_a = \frac{aS_r}{fbv}$

$lb\ O_2/DAY = 1.2\ S_r$

AERATED LAGOON (AEROBIC-FACULTATIVE)

$\frac{\Delta_e}{\Delta_0} = \frac{1}{1 + Kt}$

$lb\ O_2/DAY = 1.2\ S_r$

ANAEROBIC LAGOONS

$\frac{\Delta_e}{\Delta_0} = \frac{1}{1 + Kt}$ (SINGLE POND)

FOR PONDS IN SERIES

$\frac{\Delta_e}{\Delta_0} = \frac{1}{(1+Kt)(1+K_2t_2)\cdots(1+K_nt_n)}$

OR $\frac{\Delta_e}{\Delta_0} = e^{-K_e \Sigma t}$

TRICKLING FILTER

$\frac{\Delta_e}{\Delta_0} = e^{-K'D/Q^n}$

$K' = KA_v^m$

$Q = MGAD$

ANAEROBIC CONTACT

$SRT = \frac{X_{v(a)}}{\Delta X_v}$

EFFLUENT CHARACTERISTICS:

SOLUABLE BOD (mg/l)	10 – 20	10 – 20	{DEPENDS ON DESIGN RETENTION PERIODS & TEMPERATURE}		{DEPENDS ON PURPOSE FOR WHICH FILTER IS USED}	
TOTAL BOD (mg/l)	15 – 25	20 – 40		> 100		> 100
SUSPENDED SOLIDS (mg/l)	< 20	< 70	50-100 (FUNCTION OF POWER LEVEL)	< 50		< 100
COD (mg/l)	⟵ COD_{IN} – ($BOD_{u\,(REMOVED)}$ / 0.9) ⟶					
pH	6 – 8	6 – 8	6 – 8	—	6 – 8	6 – 8
NITROGEN (lb)	⟵ $N_0 - 0.12\,\Delta X_v$ ⟶					
PHOSPHORUS (lb)	⟵ $P_0 - 0.023\,\Delta X_v$ ⟶					

TERTIARY TREATMENT

- ION EXCHANGE
- CARBON ADSORPTION
- COAGULATION & SEDIMENTATION
- FILTRATION
  - SAND
  - MIXED MEDIA
  - DIATOMACEOUS EARTH
- FOAMING
- MEMBRANE PROCESSES
- POLISHING LAGOONS
- IRRIGATION
- STRIPPING NH_3

EFFLUENT CHARACTERISTICS: (BASED ON DATA REPORTED FOR DOMESTIC SEWAGE)

TOTAL BOD (mg/l)	—	< 1	10 — 20	10 — 20	10 — 20	—
SS (mg/l)	< 1	< 5	< 20	< 1	< 1	—
COD (mg/l)	< 50	1 — 25	< 50	30 — 60	< 50	—
TDS (mg/l)	< 500	—	—	—	< 500	—
PHOSPHORUS	< 1	—	< 1	—	—	—
NITROGEN	< 1	—	—	—	—	< 5

- DEEP WELL INJECTION

SLUDGE HANDLING

GRAVITY THICKENING

$\frac{C_v}{C_0} - 1 = \frac{B}{(ML)^n}$

FLOTATION THICKENING

$\frac{F_0}{F_u} - 1 = \frac{B}{(ML)^n}$

AEROBIC DIGESTION

$\frac{X_D}{X_{D(0)}} = e^{-K_D t}$

- ANAEROBIC DIGESTION
- LAGOON & DRYING
- DRYING BEDS
- LAND DISPOSAL
- CENTRIFUGATION

VACUUM FILTRATION

$L = 35.7 \left\{ \frac{p^{(1-S)}}{\mu R_0} \right\}^{1/2} \frac{C^m}{t^n}$

- LANDFILL
- INCINERATION
- WET OXIDATION
- BARGING
- HAULING

Overflow rates as gal/ft^2/day are computed from settling velocities.

3. Figure 3b, overflow rates vs % suspended solids removal and Figure 3c, retention time vs % suspended solids removal are developed.

4. For a required % suspended solids removal, the overflow rate and the detention time for a given initial suspended solids concentration is selected from Figures 3b and 3c.

5. For prototype design, the overflow rate is decreased by a factor 1.25 to 1.75 and the detention time increased by a factor of 1.5 to 2.0 to account for the effects of turbulence, short circuiting, and inlet and outlet losses.

6. Compute the surface area and depth.

Example -- Primary Sedimentation

The laboratory data shown in Table II-1 were obtained on the settling of a waste. Design a settling tank for a flow of 2 MGD to produce a maximum effluent suspended solids concentration of 100 mg/l. Compute the quantity of sludge to be removed at 1.5 % solids concentration.

Table II-1.
Initial SS 300 mg/l

Time, min	% Removal 2 ft	4 ft	6 ft
5	41	19	15
10	55	33	31
20	60	45	38
40	67	58	54
60	72	62	59
90	73	70	63
120	76	74	71

The data is plotted as shown in Figure 4 and the settling velocity, V_o, % suspended solids removal and overflow rate computed as shown in Table II-2.

Table II-2.

Time, min	Velocity ft/hr	% Removal of SS	Overflow rate gpd/ft^2
5	72.0	33	12950
7	51.5	34	9300
9	40.0	39	7200
23	15.6	54	2810
38	9.5	62	1710
63	5.7	69	1025
116	3.1	72	560

The data as computed in Table II-2 is plotted in Figures 5 and 6

calculation:

Design flow - 2 MGD

Initial SS - 300 mg/l

Effluent SS - 100 mg/l

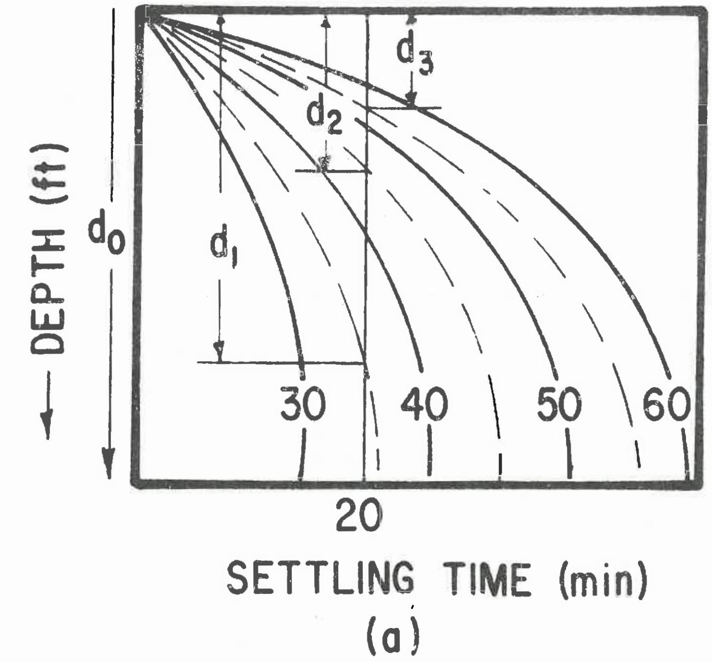
% SS removal - $\frac{(300 - 100)}{300} \times 100 = 66.6$

From Figure 5, the theoretical overflow rate is 1200 gpd/ft^2. The design overflow rate is $\frac{1200}{1.5} = 800$ gpd/ft^2.

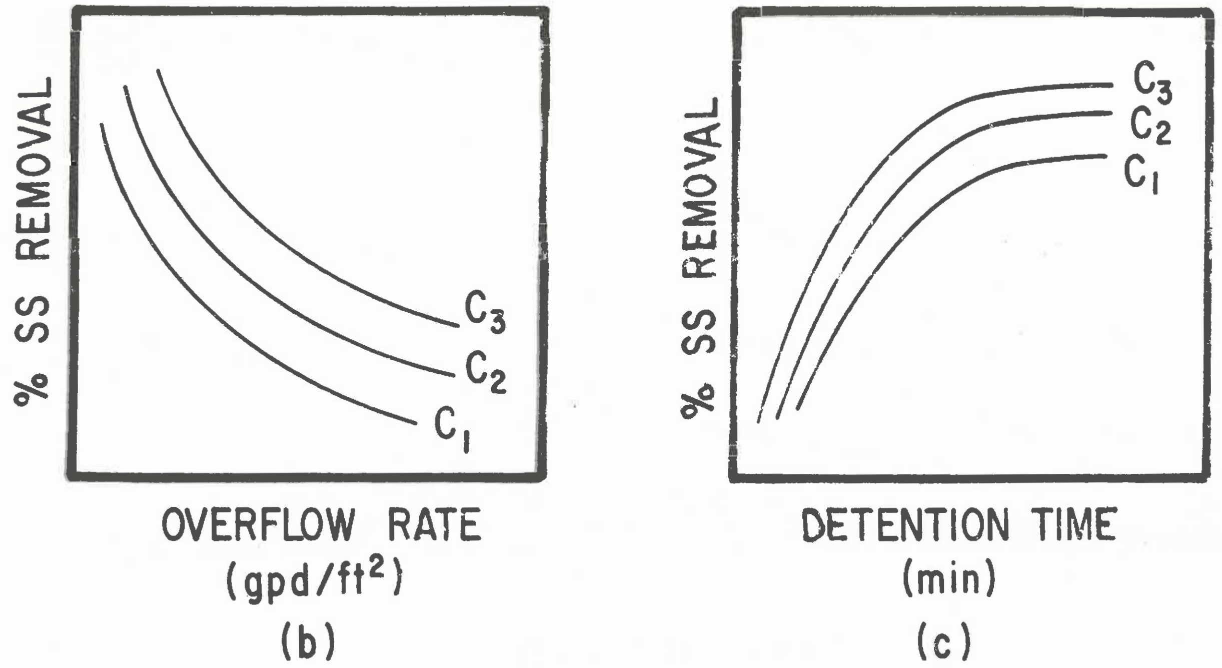
From Figure 6, the theoretical detention period is 50 min and the design detention period is (50) 1.75 = 88 min or 1.5 hr.

The surface area is $\frac{2 \times 10^6 \text{ gpd}}{800 \text{ gpd/ft}^2} = 2500 \text{ ft}^2$

The diameter of the tank is $\sqrt{\frac{2500}{.785}} = \sqrt{3180}$; $\underline{\underline{56.5}}$ feet



PERCENT SS REMOVED VS. TIME & DEPTH



OVERFLOW RATE VS. % REMOVAL

DETENTION TIME VS. % REMOVAL

Figure 3

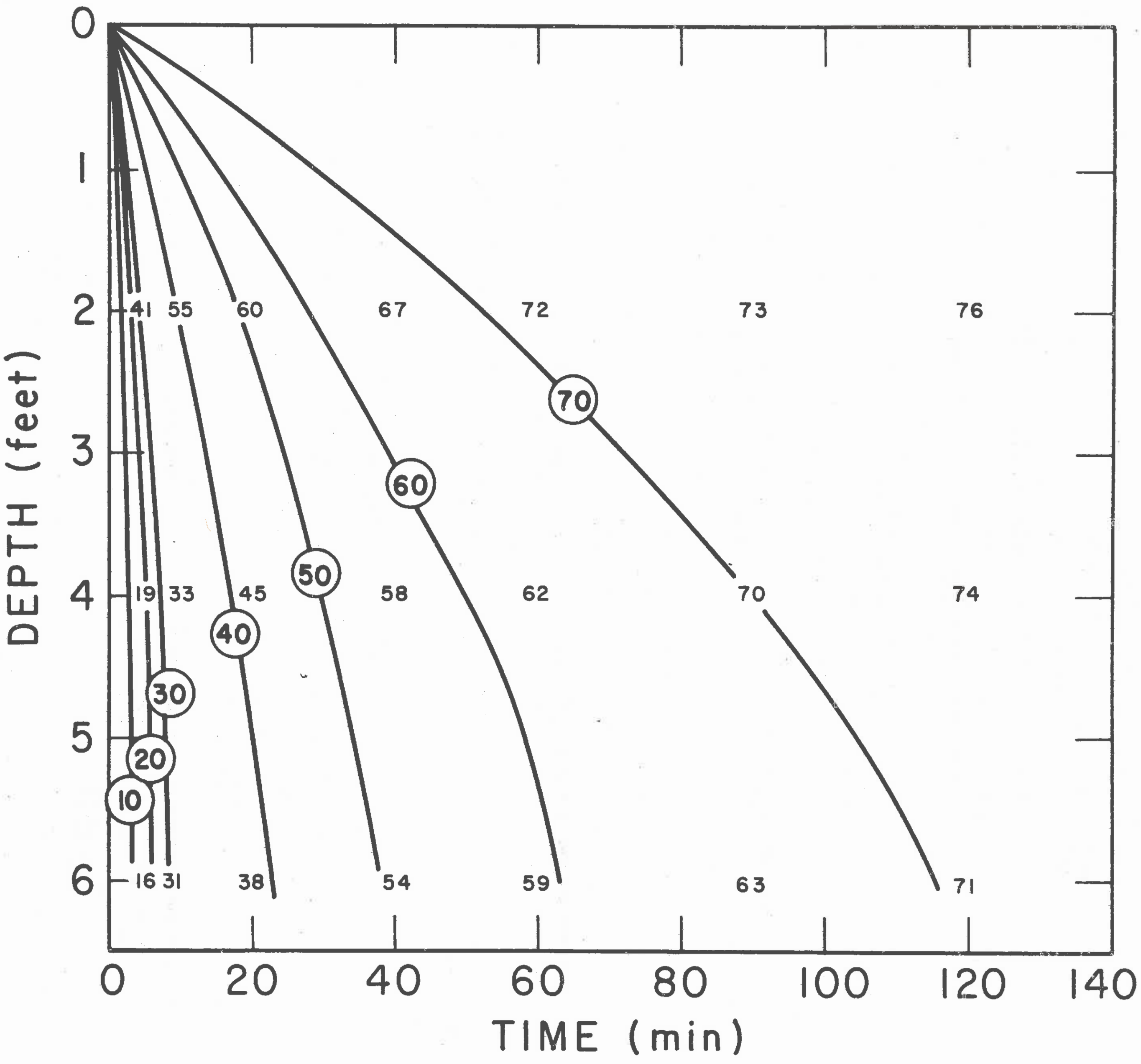

0
1
2
3
4
5
6
DEPTH (feet)
0
20
40
60
80
100
120
140
TIME (min)
10
20
30
40
50
60
70
41
55
60
67
72
73
76
19
33
45
58
62
70
74
16
31
38
54
59
63
71

FIGURE 4

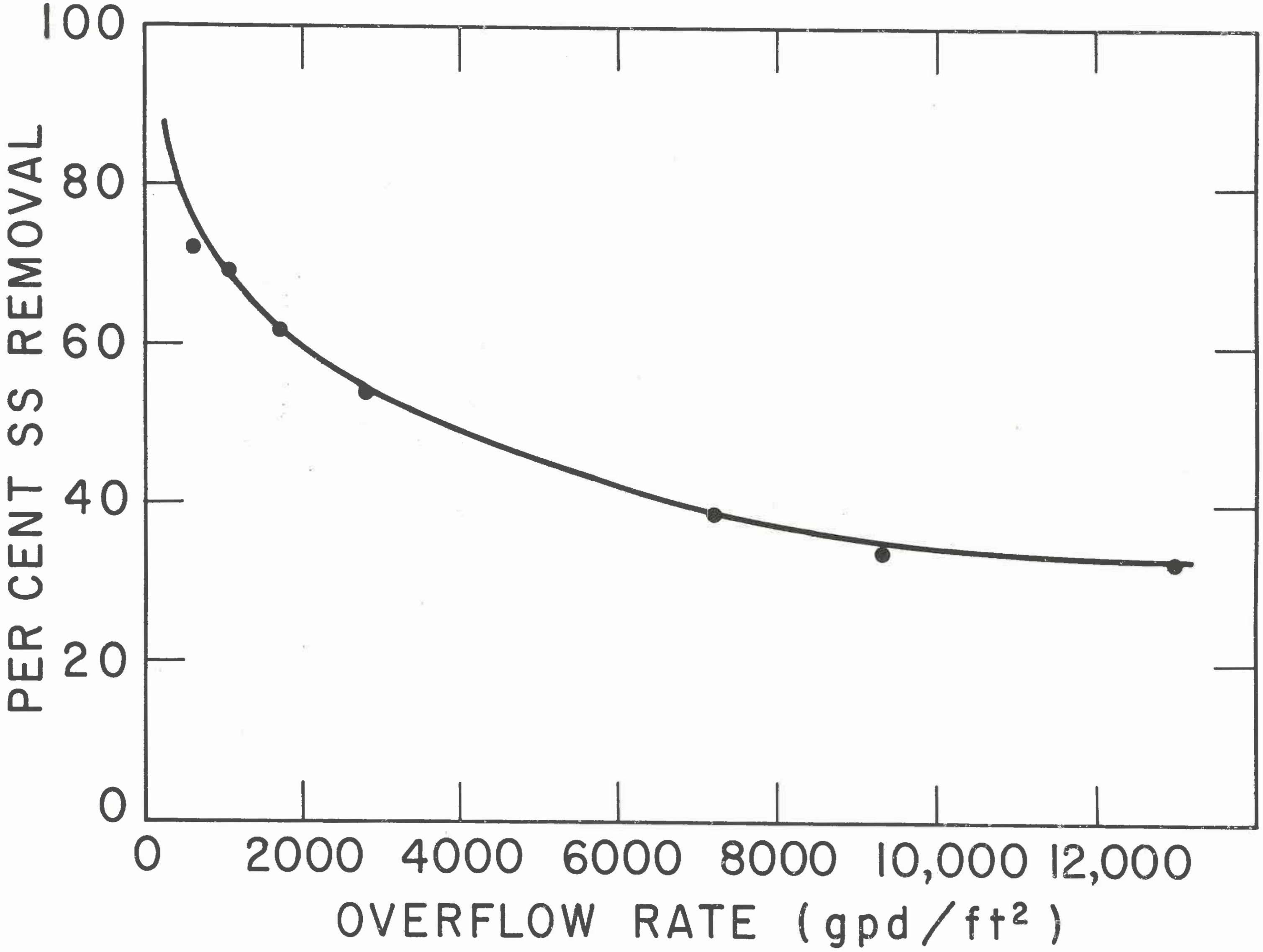

100
80
60
40
20
0
PER CENT SS REMOVAL
0
2000
4000
6000
8000
10,000
12,000
OVERFLOW RATE (gpd/ft²)

FIGURE 5

Chapter III.

FLOTATION

Procedure for the Design of a Flotation System

A. Data to be collected

1. Volume of waste flow.

2. Average suspended solids concentration and variation of solids in the waste.

3. Estimation of the flotation characteristics of the waste by the use of a laboratory flotation cell.

B. Design Procedure

1. Relate the effluent suspended solids concentration and the float solids to the calculated air/solids ratio as shown in Figures 7a and 7b.

2. Use a range of pressure of 30-60 psig. For the desired effluent suspended solids or float solids concentration, determine the optimum air/solids ratio from the plots developed in (1).

3. For the flotation system with a pressurized recycle, select a suitable operating pressure and use equation (1) for computing the recycle flow

$$\text{Air/Solids Ratio} = \frac{A}{S} = \frac{1.3\ s_a\ R\ (fP-1)}{Q\ S_a} \qquad (1)$$

in which s_a is the solubility of air in water at atmospheric pressure, S_a the influent suspended solids and f the fraction of saturation of air in the waste (about 0.5 for a baffled retention tank).

4. For a flotation system without recycle, use the equation (2) for computing the pressure.

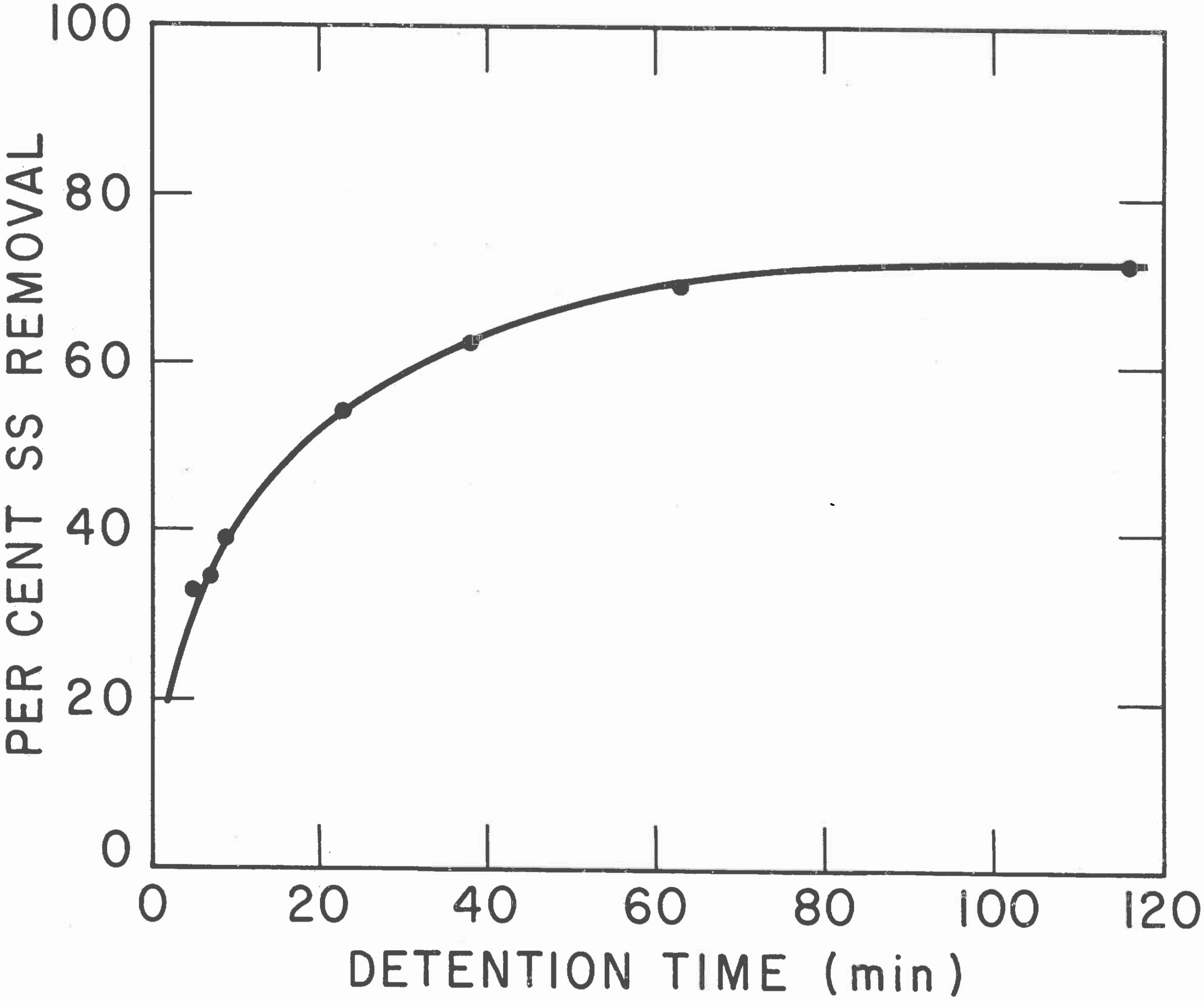

PER CENT SS REMOVAL
100
80
60
40
20
0
0
20
40
60
80
100
120
DETENTION TIME (min)

FIGURE 6

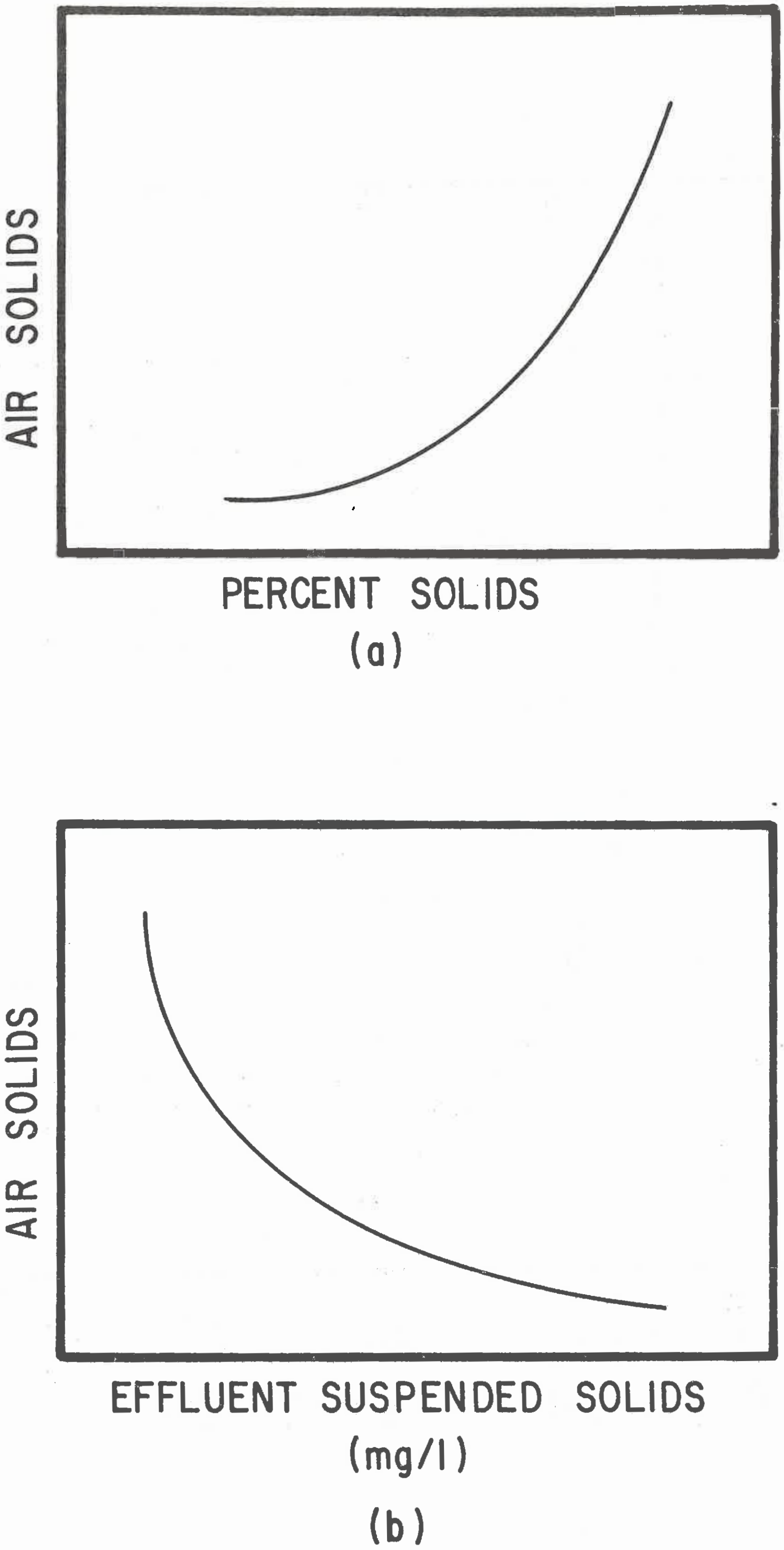

AIR SOLIDS
PERCENT SOLIDS
(a)
AIR SOLIDS
EFFLUENT SUSPENDED SOLIDS
(mg/l)
(b)

Figure 7

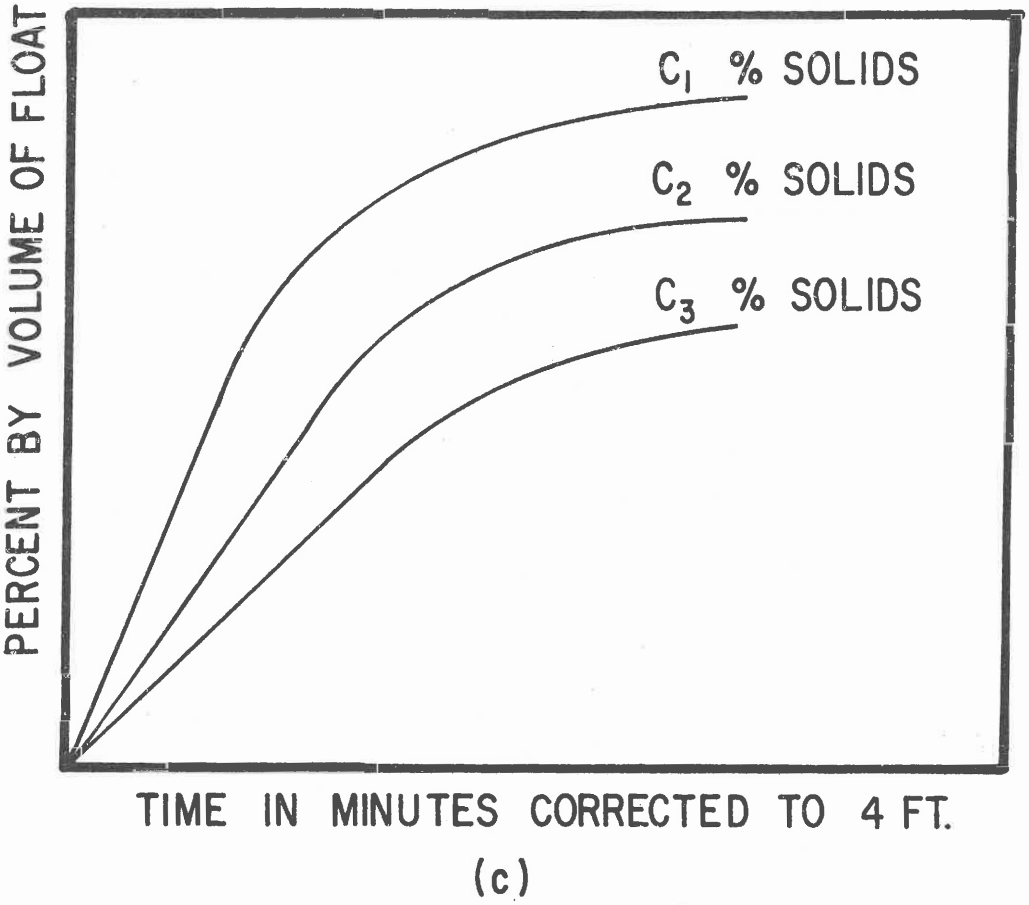


Figure 7 (cont'd)

$$\frac{A}{S} = \frac{1.3\ s_a\ (fP-1)}{S_a} \qquad (2)$$

5. Determine the overflow rate.

6. Determine the required surface area using the overflow rate obtained in (5).

Example--Flotation System

A fiber recovery flotation system produced an optimum effluent of 0.8 lb/1000 gal at an air/solids ratio of 0.04 employing polyelectrolyte for flocculation. The white water from the paper machines contains 5.4 lb/1000 gal in a flow of 400 gpm. (1) Using a flotation pressure of 40 psig and a saturation of 50 % in the retention tank, design a flotation system employing pressurized recycle. The operating temperature is 20°C. (2) Design a system for pressurization of the waste flow without recycle.

A. Influent solids = $400 \times 1440 \times 5.4 \times 10^{-3}$ = 3100 lb/day

$$\frac{A}{S} = \frac{10.7\ s_a\ (fP-1)R}{S_a}$$

$$P = \frac{40 + 14.7}{14.7} = 3.72 \text{ atm}$$

$$0.04 = \frac{(10.7)\ (18.7)\ (0.5 \times 3.72 - 1)R}{3100}$$

$$R = 0.725 \text{ MGD} = 500 \text{ gpm}$$

The total flow to the unit is 900 gpm. At an overflow rate of 3 gpm/ft^2, the required surface area is 300 ft.

B. Direct pressurization of the waste flow:

$$\frac{A}{S} = \frac{10.7\ s_a\ (fP-1)Q}{S_a}$$

$$0.04 = \frac{(10.7)\ (18.7)\ (0.5P-1)\ (0.58)}{3100}$$

$$0.5P - 1 = 1.08$$

$$P = 4.2 = \frac{p + 14.7}{14.7}$$

$$P = 48 \text{ psi (use 50 psi)}$$

At the same overflow rate the required surface area is 133 ft^2.

Chapter IV.

NEUTRALIZATION

Procedure for Neutralization Design

A. Data to be Obtained

1. Volume of waste to be neutralized
   - a. Average daily flow
   - b. Variation in flow
2. Acidity or alkalinity of waste
   - a. Average, after various periods of equalization
   - b. Variation in acidity or alkalinity
3. pH and acidity or alkalinity of neutralized wastewaters (This value will depend in great measure on where the neutralized effluent is to be discharged. If neutralization precedes biological treatment, the acidity or alkalinity must be reduced to a value such that the biological system can buffer the process to pH 7.8-8.0).

B. Design Procedure

Neutralization of acidic wastes is usually accomplished by the addition of slurried lime or by passing the waste through limestone beds.

1. Limestone beds:

   a. A relationship is developed between flow rate, depth of limestone bed and effluent pH (Figure 8a). A flow rate and a corresponding bed depth for the desired effluent pH is selected from this figure. (It should be noted that these data are based on the use of upflow units which flush out the calcium sulfate, etc., and permit escape of CO_2 generated by the neutralization reaction.)

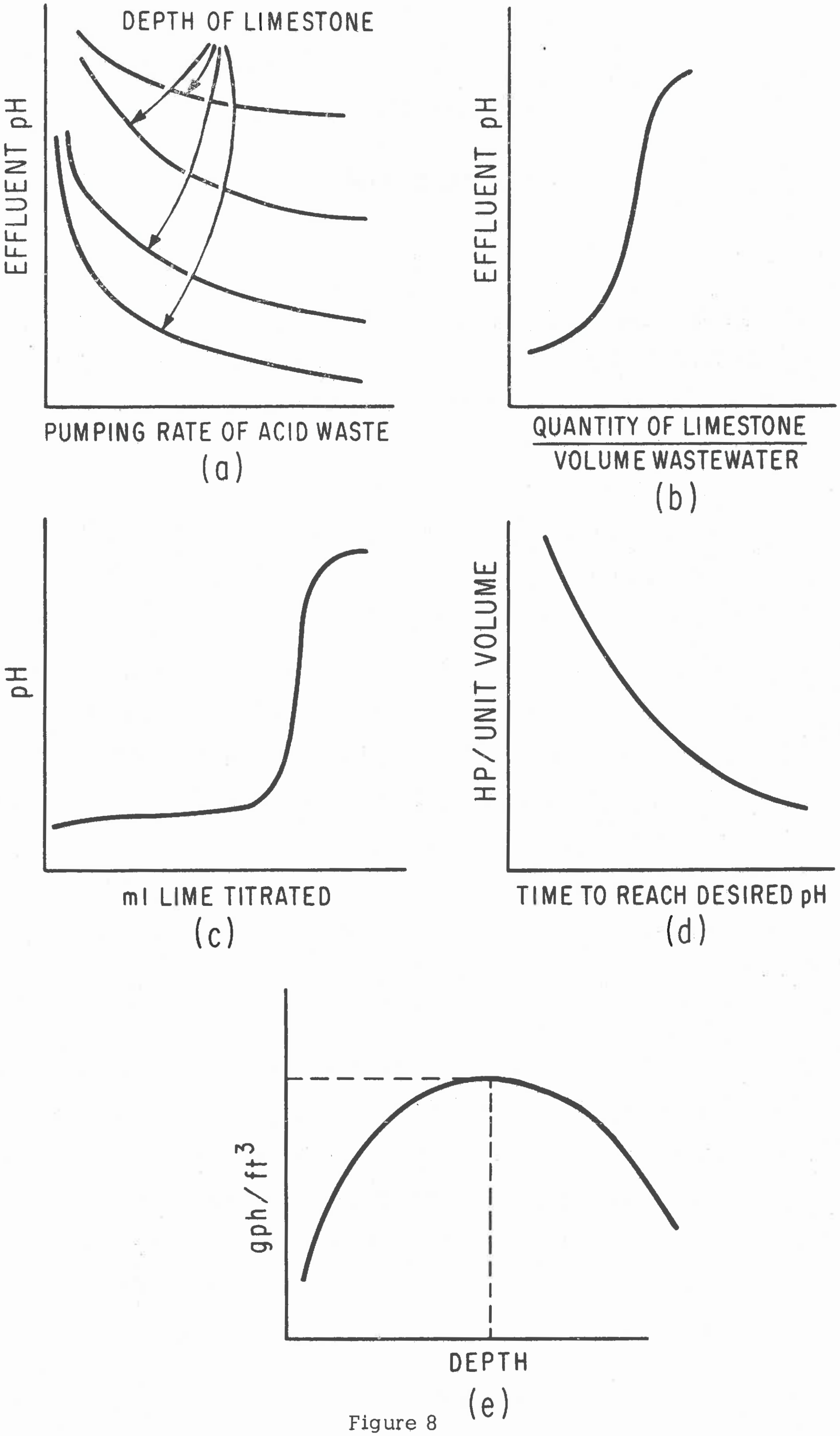

DEPTH OF LIMESTONE
EFFLUENT pH
PUMPING RATE OF ACID WASTE
(a)
EFFLUENT pH
QUANTITY OF LIMESTONE
VOLUME WASTEWATER
(b)
pH
ml LIME TITRATED
(c)
HP/UNIT VOLUME
TIME TO REACH DESIRED pH
(d)
gph/ft^3
DEPTH
(e)

Figure 8

b. The quantity of limestone required is determined from Figure 8b.

2. Lime Neutralization

a. A titration curve is developed by the addition of lime slurry to the acidic waste (Figure 8c).

b. The time of reaction of the lime slurry with the waste to the desired effluent pH is related to the mixing intensity in the neutralization basin (Figure 8d).

c. Depending upon (1) the variation in influent composition, the titration curve characteristic and the desired effluent pH, the neutralization system can be designed as a single or multiple basin process. For highly acidic wastes, a minimum of two basins is desirable, the first to raise the pH to pH 3.0-3.5 and the second to raise the pH to the desired effluent value.

3. Alkaline Wastes

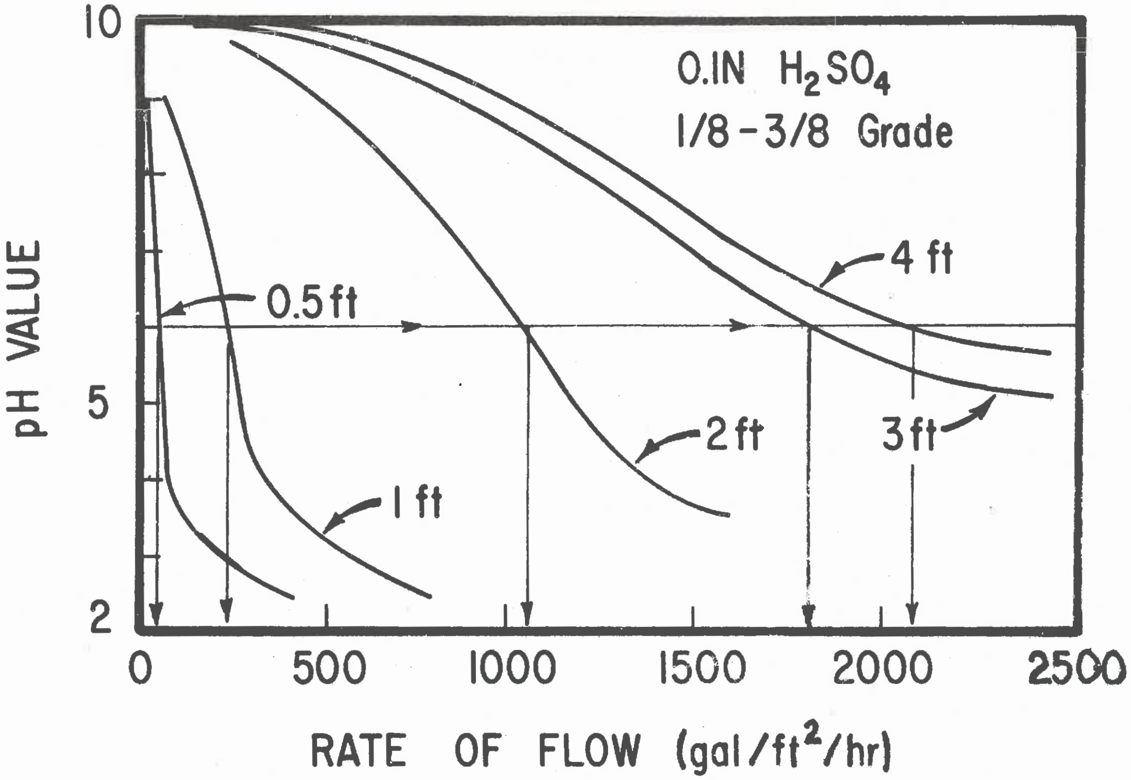
The basic design procedure for alkaline wastes is similar to acidic wastes. A mineral acid such as H_2SO_4 or is some cases CO_2 or scrubbed flue gas can be used.

<u>Problem -- Limestone Bed Neutralization</u>

It is desired to neutralize a waste acid stream containing 0.1N H_2SO_4 to pH 6.0 through a limestone bed. Assuming the neutralization follows the curves shown in Figure 9, design a neutralization system for 50 gpm of waste acid and estimate the annual limestone requirement for the most economical operation.

From Figure 9, for a pH 6.0, depth of limestone and flow rate of acid waste are obtained as follows:

Depth, ft	0.5	1	2	3	4
Flow rate, gph/ft^2	65	250	1040	1850	2230



NEUTRALIZATION THROUGH CALCINED MAGNESITE

FIG. 9

Design flow 50 x 60 = 3000 gph

The volume of the limestone bed at different depths and flow rates (gph/ft^2) required for the design flow are computed as follows:

Depth, ft	0.5	1	2	3	4
Area, ft^2	46	12	2.9	1.62	1.35
Volume of limestone, ft^3	23	12	5.8	4.9	5.4
Flow rate gph/ft^3	130	250	515	610	555

From Figure 10, it is seen that a 3 feet deep limestone bed will be the most economical.

$$\text{Acid to be neutralized per day} = 0.1 \times 49 \times 10^3 \times \frac{8.34}{10^6} \times 50 \times 60 \times 24$$

$$= 2950 \text{ lbs.}$$

$$\therefore \text{Limestone requirement} = 2950 \left(\frac{50}{49}\right)$$

$$= 3000 \text{ lbs/day}$$

At 60% reactivity, the limestone requirement will be $= \frac{3000}{0.6} = 5000$ lbs/day.

$$\text{Annual limestone requirement} = \frac{(5000)\,(365)}{2200}$$

$$= 830 \text{ Tons}$$

Selecting a limestone bed of 3 feet deep, quantity of limestone required per unit volume of wastewater for various effluent pH is computed as below.

pH	flow rate gph/ft^2	flow rate gph/ft^2	$\frac{\text{volume of limestone, } ft^3}{\text{1000 gallons of wastewater}}$
5	3000	1000	1.0
6	1850	617	1.63
7	1500	500	2.0
8	1220	407	2.46
9	860	287	3.5

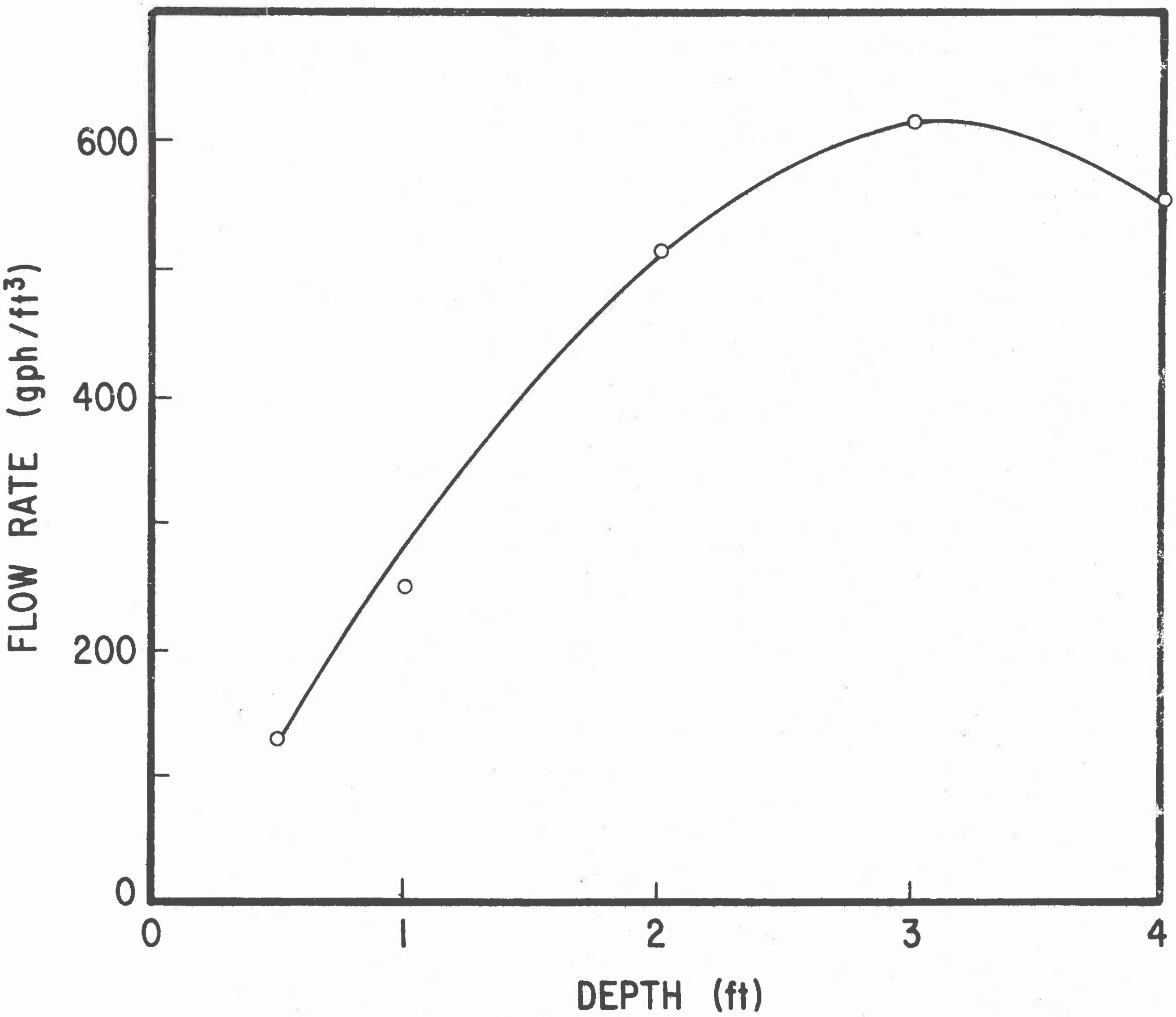


FIG. 10

This data is plotted in Figure 11 and this can be used to compute the lime requirements for various effluent pH values.

Example--Lime Slurry Neutralization

An acidic industrial waste of 100 gpm is to be neutralized to pH 7.0. Design a lime slurry neutralization system.

From the titration curve (Figure 12), lime slurry requirement for neutralizing the waste to a pH of 7.0 in 2250 mg/l

$\therefore$ Lime requirement for 100 gpm waste flow

$$= \left(100 \text{ gpm} \times \frac{60 \text{ min}}{\text{hr}} \times \frac{24 \text{ hrs}}{\text{day}}\right) \frac{8.34}{10^6} \times 2250$$

$$= 100 \times 60 \times 24 \times \frac{8.34}{10^6} \times 2250 \text{ lbs/day}$$

$$= 144 \times 8.34 \times 2.25 = 2700 \text{ lbs/day}$$

Neutralize acid in two steps to insure good control.

A. Reactor Tanks

Use 2 tanks 4.5' diameter x 4' deep

$$\text{Volume of tank} = 0.785\ (4.5^2)\ 4$$

$$= 480 \text{ gallons}$$

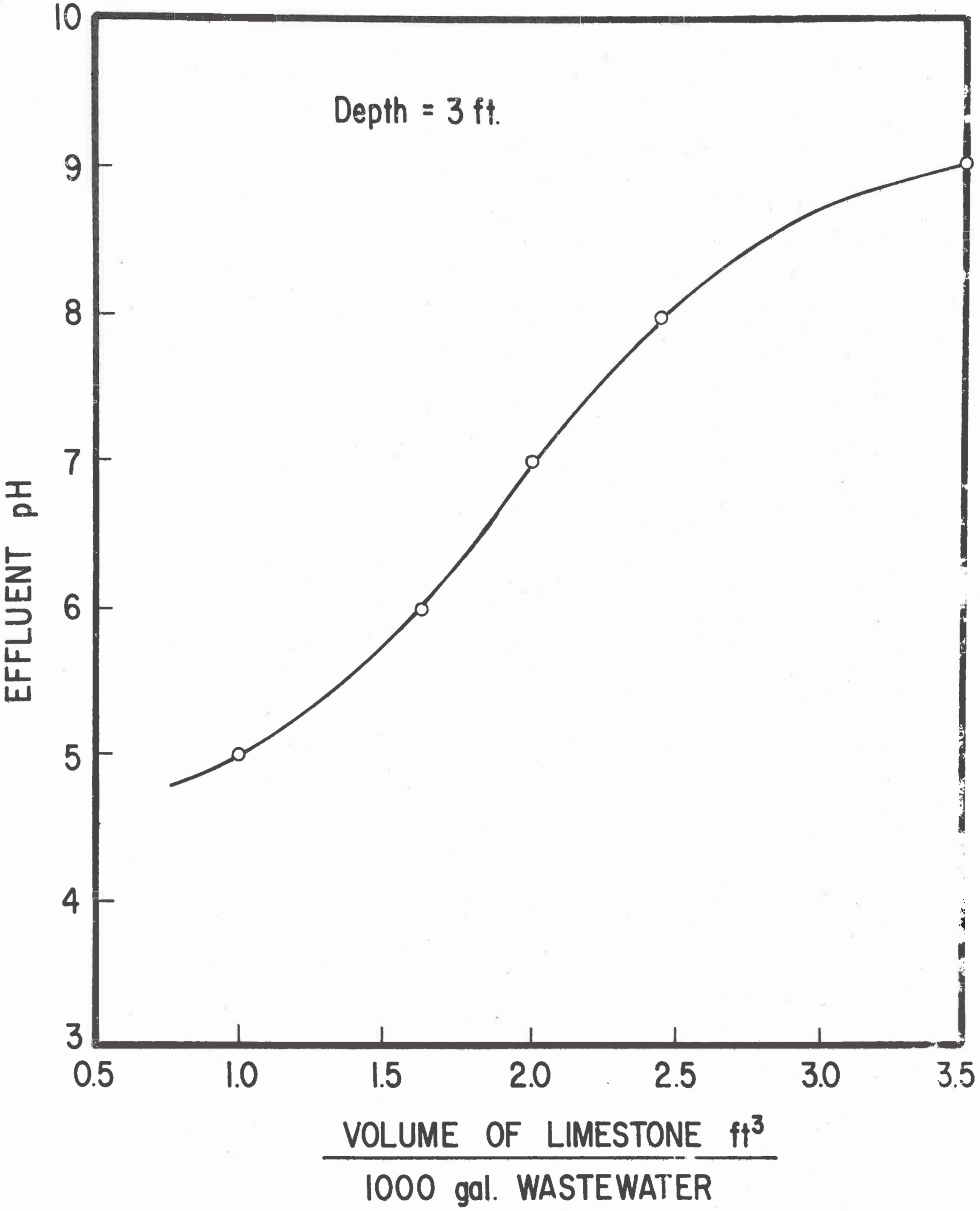
B. Mixer Horsepower Requirements

$$P = \frac{k}{g} \rho \eta^3 D^5$$

where P = Power in ft. lbs/sec
k = Mixer coefficient
ρ = Bulk unit weight
η = Speed in rps
D = Impeller dia., ft

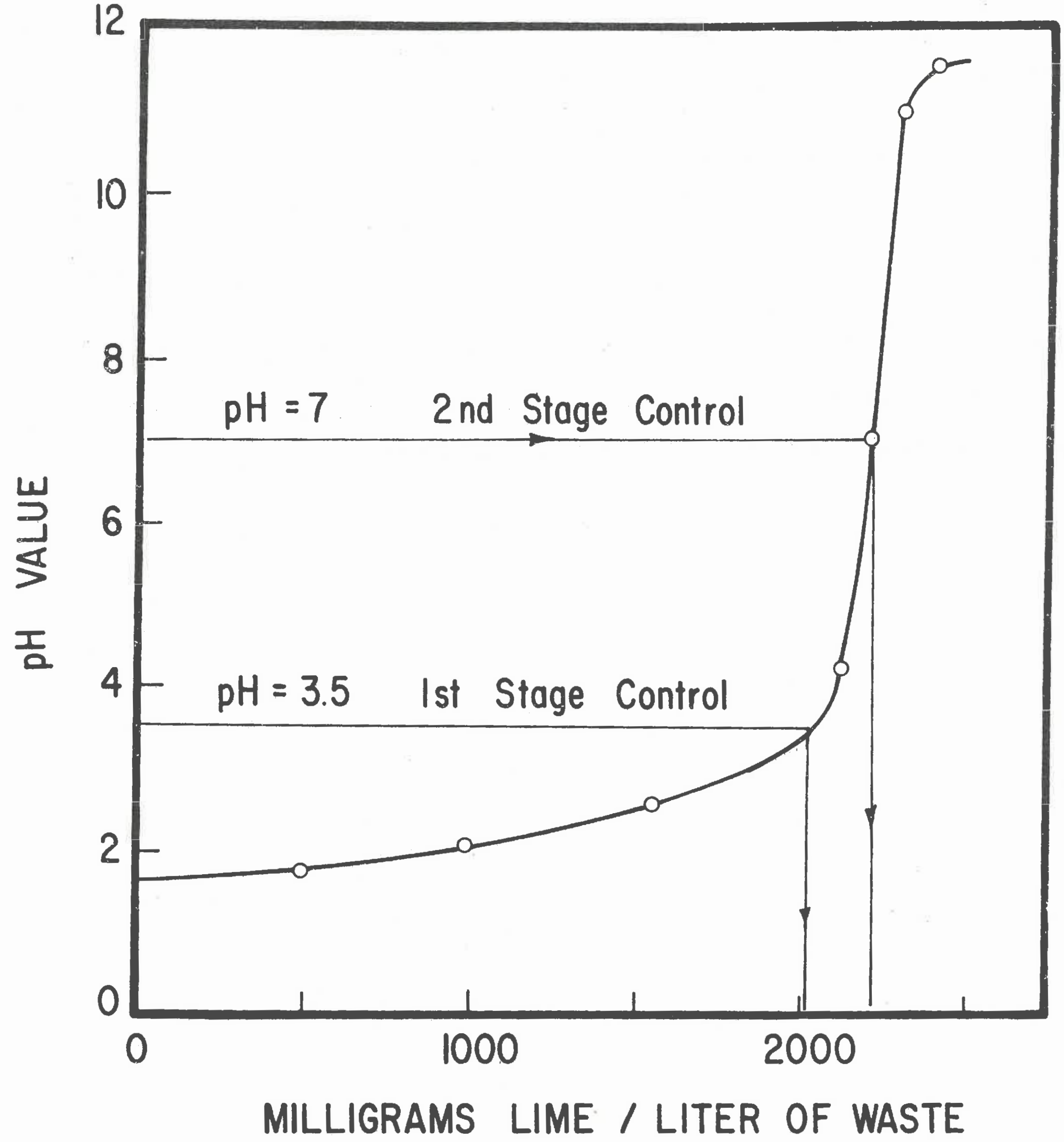
$$P = \frac{0.4}{32.2} (69.5)\ (4)^3\ (2.5)^5$$

$$= 5370 \text{ ft lbs/sec} = 9.75 \text{ HP}$$



10
9
8
7
6
5
4
3
Depth = 3 ft.
EFFLUENT pH
0.5
1.0
1.5
2.0
2.5
3.0
3.5
VOLUME OF LIMESTONE ft³
1000 gal. WASTEWATER

FIG. 11

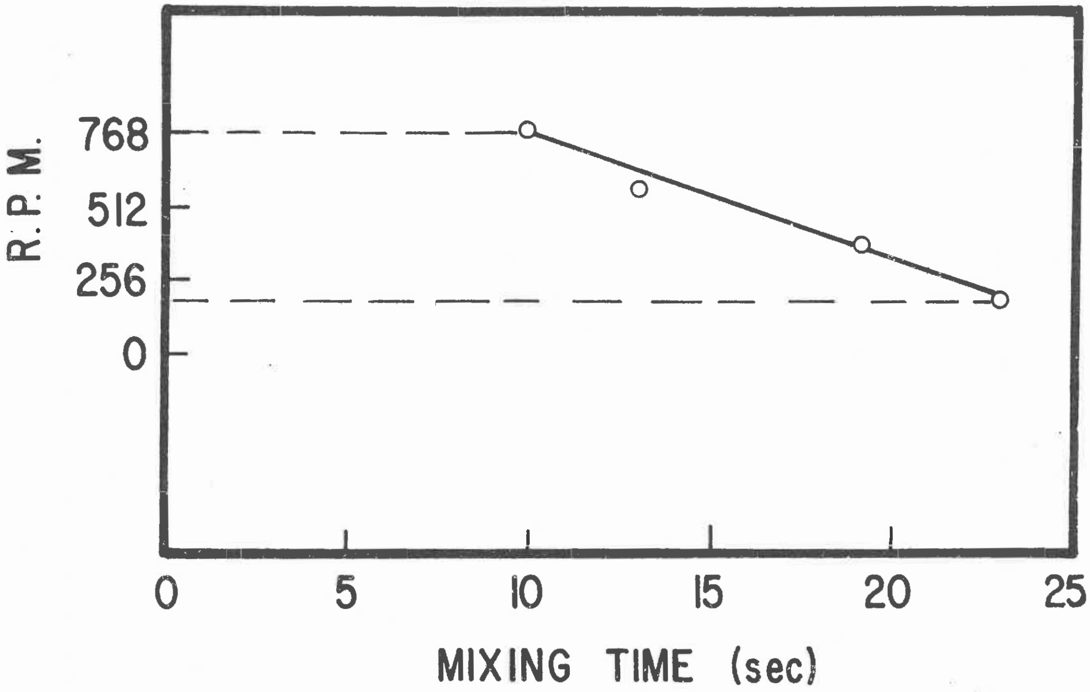


LIME - WASTE TITRATION CURVE

FIG. 12

Mixing time Sec.	Mixing Intensity RPM	HP	HP/100 gal
22.5	240	9.75	2.03
20.5	300	19.1	4.0
16.25	480	78.2	16.2
10.5	720	264.0	55.0

This data is shown in Figure 13 and can be used to compute the horsepower requirements for various reaction times, Figure 14.



MIXING INTENSITY VS REACTION TIME

FIG. 13

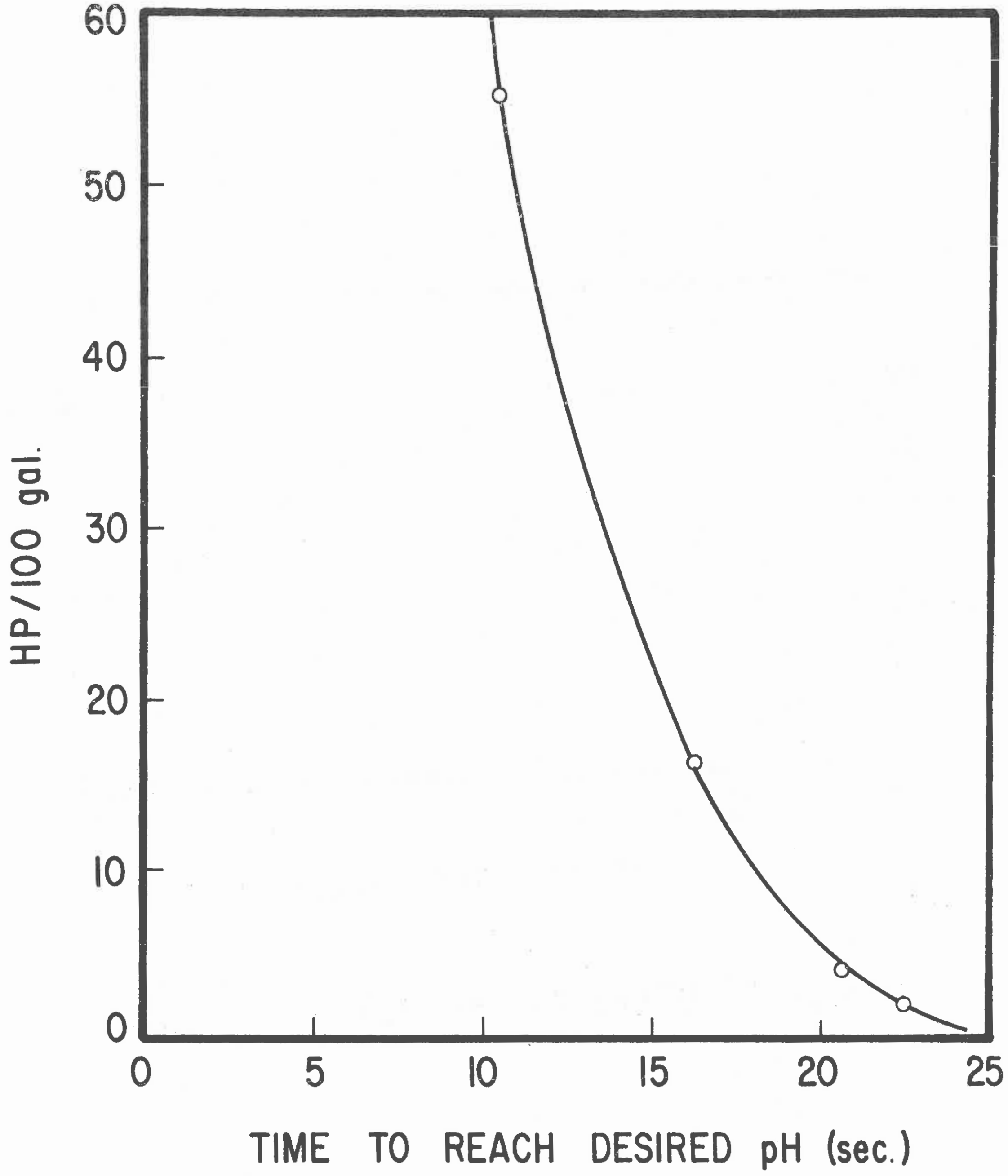

HP/100 gal.
60
50
40
30
20
10
0
0
5
10
15
20
25
TIME TO REACH DESIRED pH (sec.)

FIG. 14

Chapter V.

AERATION

Procedure for the Design of Aeration Systems

A. Data to be collected

1. Volume of aeration tank (from biological oxidation requirements)
2. Oxygen requirements (see biological treatment design)
3. Type of aeration system
4. Operating temperature
5. Oxygen transfer coefficient, α

B. Design Procedure

1. Turbine Aeration

a. Select a D/T ratio (diameter of turbine/diameter of aeration tank) between 0.1 and 0.2.

b. From the equipment manufacturer's data, determine the optimum power split (HP of the rotor/HP of compressor), oxygenation capacity, and the transfer efficiency. (See Figures 15 and 16.)

c. Select a liquid depth between 10 and 16 feet (in some cases deeper liquid depths may be employed). Size the aeration tank either as a square or a rectangle.

d. Determine the number of turbine units based on the approximate basis of one for every 900-2500 ft^2.

e. Determine the air flow rate to supply the required oxygen from the optimum transfer efficiency (Figure 16).

f. Determine the operating compressor horsepower using the relationship $HP = \frac{G_s\ (psi)(144)}{(33,000)\ E}$ and correct it for waste conditions.

in which G_s is the air flow rate and E the motor and blower efficiency.

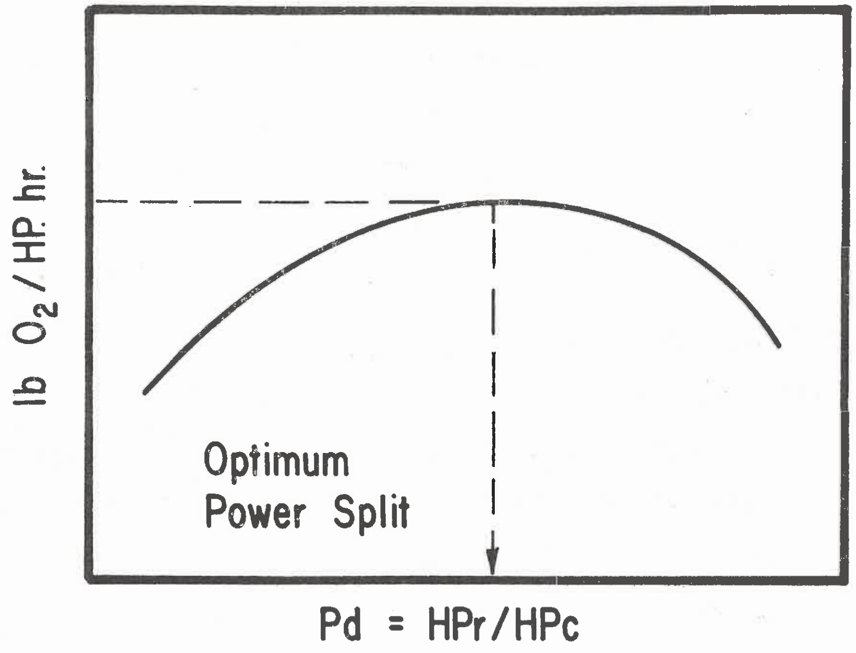
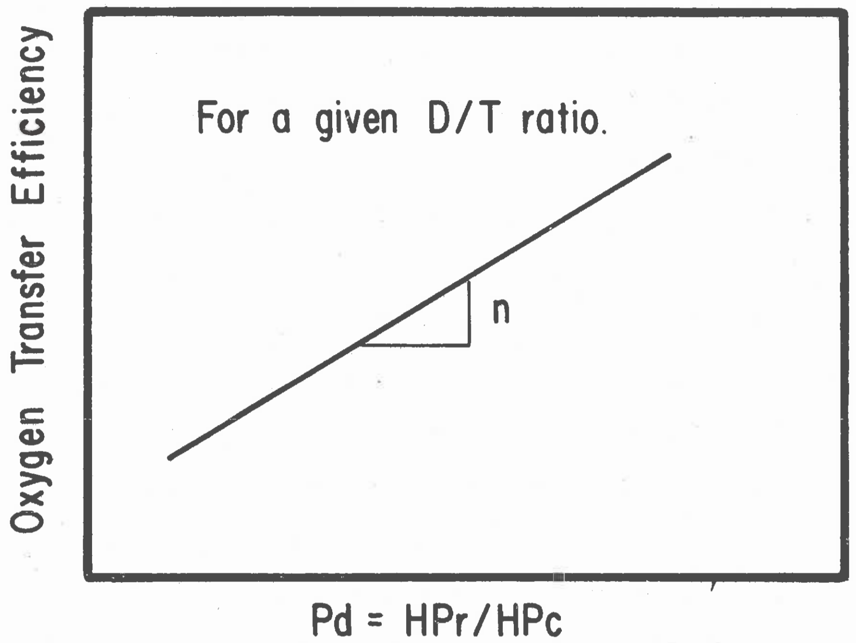


FIGURE 15



Oxygen Transfer Efficiency = $C(HPr/HPc)^n$

FIGURE 16

g. Determine the turbine HP from the power split (Figure 15).

h. Calculate the aeration efficiency correcting for waste conditions.

2. Diffused Aeration

a. Select a tank depth between 10 and 16 feet.

b. Size the aeration tank so that its maximum width is approximately twice the tank depth. This is necessary in maintaining adequate mixing.

c. Select the air flow rate for the air diffusion unit.

d. Compute C_{SM} (oxygen concentrations at tank mid-depth) using the relation:

$$C_{SM} = C_s \left(\frac{P_b}{29.4} + \frac{O_t}{42}\right)$$

where

C_{SM} = saturation of oxygen at aeration tank mid-depth

C_s = saturation concentration of 0_2 in wastewater

P_b = absolute pressure at the depth of air release, lb/sq in

O_t = % concentration of oxygen in the air leaving the aeration tank

e. The oxygenation capacity of the aeration unit (N) is computed using the relationship:

$$N = C\,G_s^{\,n}\ \frac{H^m}{W^p}\ (C_{SM} - C_L)\ \ 1.02^{(T-20)}\,\alpha$$

where

N = lbs of 0_2/hr/aeration unit

C = constant for the aeration unit

G_s = air flow, scfm/aeration unit

H = liquid depth, feet

W = aeration tank width, ft

C_L = dissolved oxygen concentration in liquid

T = temperature °C

α = oxygen transfer coefficient of the waste

n, m, p = exponents characteristic of the aeration device

f. Determine the number of aeration units required to transfer the required amount of oxygen.

g. Determine the spacing of the aeration units from the relationship spacing

$$\text{spacing} = \frac{\text{Length of Tank}}{\text{No. of aeration units}}$$

The minimum and maximum spacings should be six inches and 24 inches to 30 inches respectively in order to maintain solids in suspension and to minimize bubble coalescence. If the spacings calculated fall outside this range, double rows or an adjustment in the number of units should be made.

h. Compute the total air flow and the required horsepower of the blower using the relationship:

$$HP = \frac{(\text{psi})(\text{scfm})(144)}{(33{,}000)\ (\text{Eff.})}$$

Usually a 6-10 psi pressure is adopted on the blower and the efficiency depends on the type of the diffused aeration unit.

i. Compute the oxygenation efficiency.

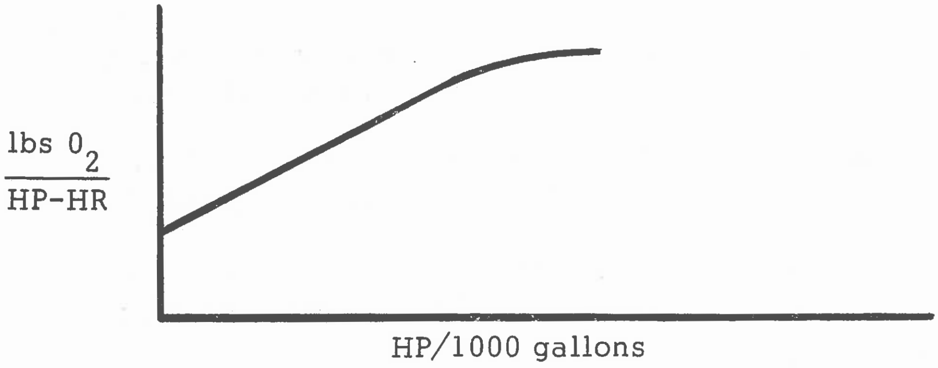
3. Surface Aeration

a. Select a value of C_L depending on the process involved (i.e., BOD removal only or nitrification)

b. Determine $\propto$ (the oxygen transfer coefficient for the waste) from laboratory scale studies.

c. Compute C_{SW} (oxygen saturation concentration for waste)

d. From the performance characteristics curve of the surface aerator, determine the value HP/1000 gallons for a selected N_o (Figure below)



e. Compute N (the oxygen transfer capacity of the aeration unit for field conditions) using the relationship:

$$N = \frac{N_o (C_{SW} - C_L)}{9.1} \propto 1.02^{(T-20)}$$

f. Compute the total HP required to transfer the desired amount of oxygen.

g. Recompute from the HP calculated, the power level as HP/1000 gallons and check to see if the power level is sufficient for mixing and keeping the solids in suspension. Power levels of approximately 0.008 and 0.09 HP/1000 gallons are required to transfer the oxygen by diffusion and to keep the solids in suspension respectively.

Table V-1.

Characteristics of Diffused Aeration Devices

Unit	C	N	Conditions
Saran Tubes	0.160	0.90	9" spacing; wide band 14.4' depth, 24' width
Saran Tubes	0.170	0.81	9" spacing; narrow band 14.4' depth, 24' width
Saran Tubes	0.150	0.92	9" spacing; narrow band 14.4' depth, 24' width
Sparjers	0.081	1.02	24" spacing; wide band 14.8' depth, 24' width
Sparjers	0.062	1.02	9" spacing; narrow band 14.8' depth, 24' width
Sparjers	0.064	1.02	9/32" orifice; 25' width 15' depth
Sparjers	0.068	1.02	13/64" orifice; 25' width 15' depth
Plate Tubes	0.350	0.49	single row; 25' width, 15' depth
Plate Tubes	0.200	0.80	double row; 25' width, 15' depth
INKA System	0.036	0.95	6.8' width 6' depth, 2.6' submergence

Example -- Aeration Design

An aeration tank with a volume of 1.7 MG has an average oxygen requirement of 445 lb 0_2/hr. For a completely mixed system, design

(a) a turbine aeration system

(b) a diffused aeration system

(c) a surface aeration system

The operating conditions are:

Temperature = 30^oC

$\propto$ = 0.85

C_s = 7.4 mg/l

A. Turbine Aeration

1. A D/T ratio of 0.115 is selected

2. From the equipment data (see Figure 15) the optimum power split has been found to be near 1.0.

For this power split, the oxygenation capacity is 2.5 lb 0_2/HP, hr.

3. Selecting a depth of 15 feet, the size of the aeration tank is 60' x 70'.

4. Selecting four units, the capacity required of each = $\frac{445}{4}$ = 111.25 lb 0_2/hr.

5. Air flow rate required:

$$G_s = \frac{\text{lb } 0_2/\text{hr}}{60\ (0.232)(0.0746)(0.20)}$$

$$= \frac{112.5}{(60)(0.232)(0.0746)(0.20)}$$

$$= 525 \text{ scfm}$$

6. Compressor Horsepower - The air will be discharged at a point two feet above the tank bottom

$$\therefore \quad p = \frac{15 - 2.3}{2.3} = 5.5 \text{ psi}$$

Assuming 70 % efficiency:

$$HP = \frac{(525)(5.5)(144)}{(33,000)(0.7)}$$

$$= 17.5$$

correcting for waste conditions

$$HP = \frac{17.5}{\frac{(C_{SM} - C_L)}{C_s} \propto (1.02)^{T-20}}$$

$$= \frac{17.5}{\frac{(8.6 - 1.0)}{10.8}(0.85)(1.02)^{10}}$$

$$= 24$$

C_{SM} = mid depth wastewater saturation value

C_s = mid depth water saturation value

7. Turbine HP; turbine HP = compressor HP = 24

8. The aeration efficiency is $\frac{111.25}{48}$ = 2.30 lb 0_2/HP hr

B. Diffused Aeration

1. Tank depth 15 feet

2. Selecting a width of 30 feet, required aeration tank length is 550 feet.

3. Air flow rate of diffuser is 8 scfm

4. Computation of C_{SM}

C_s = 7.4 mg/l

P_b = $\frac{15}{2.3}$ + 14.7 = 21.2 psi

O_t = $\frac{18.9}{18.9 + 79}$ x 100 = 19.3

C_{SM} = $7.4 \left(\frac{21.2}{29.4} + \frac{19.3}{42}\right)$

= 8.6 mg/l

5. Calculate N

$$N = 0.0069\, G_s^{0.92} \frac{H^{0.72}}{W^{0.36}} (C_{SM} - C_L)\, 1.02^{(T-20)} \alpha$$

$$= 0.0069\ (8)^{0.92} \frac{15^{0.72}}{30^{0.36}} (8.6 - 1.0)(1.02)^{10}(0.85)$$

= 0.75 lb 0_2/hr/unit

6. Number of aration units = $\frac{445}{0.75}$ = 600

7. Spacing = $\frac{550}{600}$ ft = 0.91 ft or 11 in.

8. Total air flow = 600 units x 8 scfm/unit = 4800 scfm

Blower HP (assuming 8 psi on the blower):

$$HP = \frac{(8.0)(4800)(144)}{(33,000)(0.7)} = 240$$

9. The oxygenation efficiency is $\frac{445}{240}$ = 1.86 lb 0_2 HP hr.

C. Surface Aeration

1. C_L = 1.0

2. α = 0.85

3. C_{SW} = 90 % of C_s = (0.90) 7.4 = 6.7 mg/l

4. Select N_o = 2.9 with a corresponding power level = 0.11 HP/1000 gal.

5. Compute N; $= 2.9 \frac{(6.7-1.0)}{9.1} (0.85)(1.22)$

$= 1.9$ lb O_2/HP.Hr.

6. Total HP required $= \frac{445}{1.9} = 235$ select 4 units each 60 HP

7. The power level is $\frac{240}{1700} = 0.142$ HP/1000 gallons which checks the originally assumed value (Step 4)

This is higher than 0.08 HP/1000 gallon, the power level required for keeping the solids in suspension.

Procedure for the Determination of α

A. Laboratory Procedure

1. Deoxygenate tap water in the test basin using nitrogen gas or sodium sulfite.

2. Reaerate the water using a stirring device or compressed air, simulating field conditions as closely as possible.

3. Record the dissolved oxygen level at various time intervals until at least 90% of saturation is reached. An oxygen probe or the Winkler Method can be used to determine the oxygen level.

4. Plot the oxygen deficit against aeration time as shown in Figure 17 and determine K_La.

5. Repeat the same procedure, using the wastewater as the test liquid.

6. Evaluate K_La of the wastewater and divide this value by K_La of pure tap water, thus determining alpha (α).

B. Precautions

1. The test should be run at the same temperature as that expected in the field. Care should be taken in establishing the saturation value of oxygen in the wastewater, as this normally will differ from the saturation value of tap water at the same temperature.

2. It is also important to test the tap water and the wastewater under similar conditions.

Example--Determination of α

The following data was obtained on the oxygen transfer capacity of a waste. Determine α

Temperature 20°C

C_s of waste	8.2 mg/l
C_s of distilled water	9.2 mg/l

	Concentration of Dissolved oxygen C, mg/l	
Time, hr	Waste	Distilled water
1	0.25	0.4
2	0.5	0.7
3	0.70	0.9
4	0.95	1.4
5	1.2	1.7

Solution

A table showing the deficit against time is computed as below

Time, hr	Waste		Distilled water	
	C	C_s-C	C	C_s-C
1	0.25	7.95	0.4	8.8
2	0.5	7.7	0.7	8.5
3	0.7	7.5	0.9	8.1
4	0.95	7.25	1.4	7.8
5	1.2	7.0	1.7	7.5

A plot of t vs C_s - C is developed as in Figure 17. The slope of the line gives the value of K_La

$$K_La \text{ of waste} = 2.3 \frac{\log\left(\frac{8.2}{7.0}\right)}{5} = 0.0322/\text{hr}$$

$$K_La \text{ of distilled water} = 2.3 \frac{\log\left(\frac{9.2}{7.5}\right)}{5} = 0.0405/\text{hr}$$

$$\alpha \text{ of waste} = \frac{K_La \text{ of waste}}{K_La \text{ of distilled water}}$$

$$= \frac{0.0322}{0.0405}$$

$$= \underline{\underline{0.795}}$$

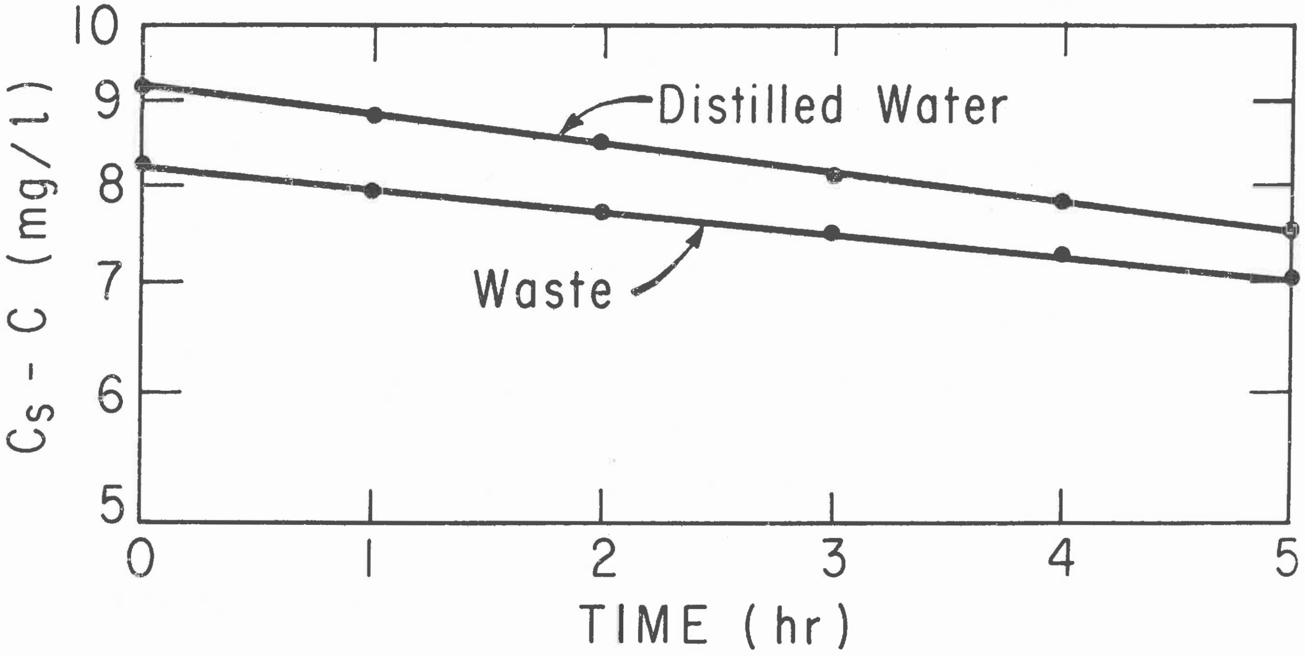


FIG. 17

Chapter VI.

BIOLOGICAL OXIDATION PROCESSES

Procedure for the Design of a Biological Oxidation Plant

A. Data to be collected

1. Waste Flow
   - a. Average daily waste flow, (MGD) - present
     Average daily waste flow, (MGD) - future design
   - b. Maximum average 4-hour waste flow - present
     Maximum average 4-hour waste flow - future design
   - c. Peak flow at any time - future design
2. Waste Strength*
   - a. Average daily BOD_5 (24-hour composite), mg/l
   - b. Maximum 4-hour composite BOD, mg/l
   - c. Maximum 4-hour BOD_5 loading, (lbs/day)
3. Other Waste Characteristics
   - a. Suspended solids and volatile suspended solids (24-hour composite), mg/l
   - b. COD or TOC, (24-hour composite), mg/l
   - c. Nitrogen (NH_3 -N and total organic) and phosphorus (24-hour composite), mg/l
   - d. pH-instantaneous values
   - e. Alaklinity or acidity (if pH is less than 5.5 or greater than 9.0) - instantaneous values
   - f. Presence of heavy metals, oils, toxic organics, etc.

*In some cases waste strength is reported in terms of COD or TOC. The non-biodegradable fraction must be subtracted to get an equivalent BOD. A further adjustment must be made to get BOD_5. For example, in a waste the COD is

1,000 mg/l and the estimated non-biodegradable portion is 200 mg/l. The ultimate BOD would be approximately 800 mg/l and the BOD_5 approximately 800 x 0.8 = 640 mg/l. (The BOD_5 will be approximately 80% of the ultimate BOD for most readily degradable organic wastes). If the results are reported as TOC (Total Organic Carbon), they may be converted to COD by multiplying by O_2/C (32/12) and then by 0.8 to get the BOD_5 for most organics. The same precautions for non-biodegradables must be considered when using TOC.

B. Design Procedure

1. Pre or Primary Treatment

a. If suspended solids are present in excess of approximately 125 mg/l, solids separation by lagooning, sedimentation, or flotation should be considered. For estimating purposes only, a sedimentation tank would have an overflow rate of about 1000 gpd/ft^2.

b. If oil, grease, or floatables exceed 50 mg/l a skimming tank or separator should be provided.

c. Heavy metals (Cu, Zn, Ni, etc.) should be removed prior to biological treatment.

d. (1) If the pH exceeds pH 9.0, neutralization should be provided if the ratio of caustic alkalinity (expressed as $CaCO_3$) to COD removed exceeds 0.7 lbs $CaCO_3$/lb COD or 0.56 lbs $CaCO_3$/lb BOD_5 removed. Neutralization need only reduce the alkalinity to the aforementioned levels.

(2) In some cases where a wide variation in alkalinity is encountered during the day or plant operating schedule, the aforementioned levels can be achieved by providing an equalization tank or pond.

e. If the waste contains organic acids, bio-oxidation will convert these acids to CO_2 and bicarbonate salts, provided the process design reduces these to < 25 mg/l as BOD_5.

f. When mineral acids are present, neutralization or equalization should be provided if the pH is less than pH 4.5.

g. Sulfide should be prestripped or otherwise removed if their concentration exceeds 50 mg/l.

h. If the influent BOD loading in lbs/day, based on four hour composites exceeds a 3:1 ratio, an equalization tank should be considered to bring the variation within this range.

2. Biological Treatment

It is first necessary to select the process or processes applicable to the solution of the particular problem. The options for biological treatment with general considerations in process selection are as follows:

a. Activated sludge - should provide an effluent with a soluble BOD_5 of less than 10 - 15 mg/l and a total BOD_5 including carryover suspended solids of less than 20 mg/l. The process requires treatment and disposal of excess sludge and would generally be considered where high effluent quality is required, available land area is limited, and waste flows exceed 0.1 MGD.

b. Extended aeration or total oxidation - will provide an effluent with a soluble BOD_5 of less than 10 - 15 mg/l and a total BOD_5 of less than 40 mg/l. The suspended solids carry-over may run as high as 50 mg/l (high clarity [low solids] effluent will usually require a post treatment by filtration, coagulation, etc.). This process is usually considered for waste flows less than 2.0 MGD.

c. Contact stabilization - is applicable where a major portion of the BOD is present in colloidal or suspended form. As a general rule the process should be considered when 85% of the BOD_5 is removed after 15 minutes contact with aerated activated sludge. The effluent suspended solids are of the same order as those obtained from activated sludge.

d. Aerobic lagoon - is only applicable where partial treatment (~ 50 - 60% BOD_5 reduction) and a high effluent suspended solids are permissible. This process should be considered as a stage development, which can be converted into an extended aeration plant at some future date by the addition of a clarifier, return sludge pump and additional aeration equipment.

e. Aerated Lagoon - will provide effluent soluble BOD_5 of less than 25 mg/l with a total BOD_5 of less than 50 mg/l depending on the operating temperature. The effluent suspended solids may exceed 100 mg/l. The system is temperature sensitive and treatment efficiencies will decrease during winter operation. A post treatment is necessary if a highly clarified effluent is desired. Large land areas are required for the process.

f. Trickling filters are covered separately in the text.

g. Anaerobic and facultative ponds - for industrial waste treatment should only be considered if odors will not cause a nuisance. If high degree treatment is required, these ponds must be followed up by aerobic treatment (i.e., aerated lagoons, activated sludge, etc.).

3. Biological Treatment Design

After selection of the possible process consideration from (2) the preliminary design calculations should be developed as follows: (This procedure presumes that no pilot plant or laboratory treatment study data is available. If such data is available, the appropriate coefficients and factors developed from the laboratory study should be used).

a. General

(1) Select the appropriate k, a, a', b, and b'. It should be recognized that at this time there is limited data on the treatment of many industrial wastes so that some of the coefficients are necessarily estimations. As more data is accumulated, these coefficients will be refined.

(2) Each of the processes are designed using the equations shown in Table 4. It is important to remember that the coefficients generally apply to all systems, and the principle differences between systems are the changes in concentration of biological solids and retention time with the exception of the aerated lagoon which has a feedback of BOD from the anaerobic sublayer.

b. The Aerobic Lagoon

(1) The required retention period for a specified effluent soluble BOD is estimated by combining Equations (1) and (3).*

---

*See page 51 for summary of design equations.

$$t\text{, days} = \frac{1}{(24\ aKS_e - b)}$$

(2) The equilibrium volatile solids in the lagoon is computed from Equation (3)

$$X_v = \frac{S_o + aS_r}{(1 + bt)}$$

and the total suspended solids is

$$X_a = \frac{X_v}{\text{volatile fraction}}$$

For soluble organic waste the volatile fraction can be assumed as 80%. When suspended solids are present in the waste, this estimate should be revised depending on the volatile fraction of the influent suspended solids.

(3) The effluent BOD will be composed of the soluble BOD plus that contributed by the concentration of suspended solids in the basin which will be approximately the same as in the effluent for a completely mixed system.

$$\text{Effluent BOD} = S_e + f\,X_a$$

when f is the BOD contributed per unit of effluent suspended solids. This is selected from Figure 18 or from a plot developed from experimental data.

(4) The oxygen requirements are calculated from Equation (5) and include the oxygen used by the organisms for growth and those used for endogenous respiration.

c. The Aerated Lagoon

(1) Since the relatively low power level in the system permits the major portion of the suspended solids to settle

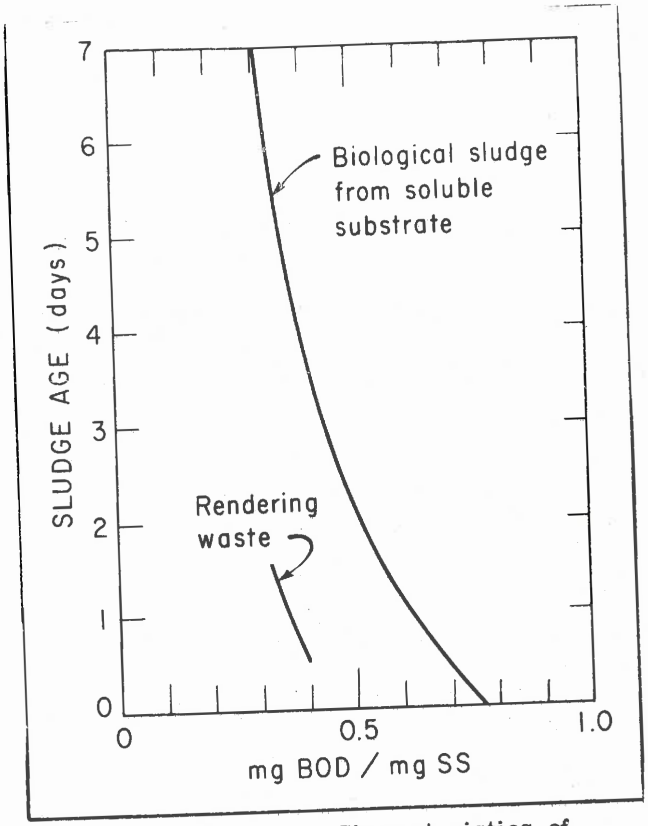


Figure 18. BOD Characteristics of Biological Sludges

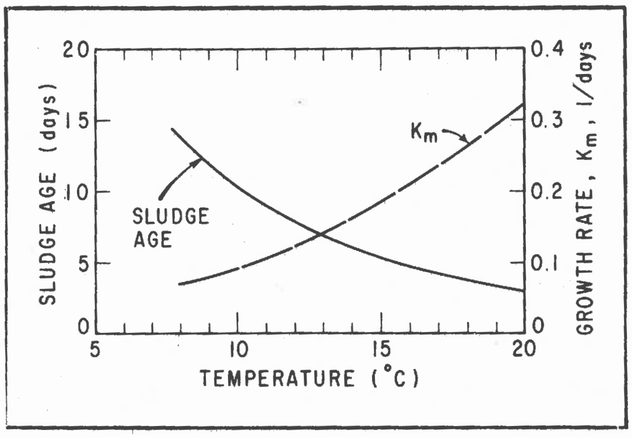


Figure 19. Growth Rate of Nitrosommonas (After Downing)

to the bottom of the basin, the solids level in suspension will be low. The volatile solids maintained in suspension will vary from 50 - 150 mg/l depending on the power level maintained. The effluent soluble BOD is selected and the required retention period calculated from Equation (1).

$$t = \frac{S_r}{24\,KX_v S_e}$$

(2) The total effluent BOD is computed as in B(2) using the effluent volatile suspended solids selected in (1).

(3) Due to the feedback of BOD from the anaerobic sublayer the oxygen requirements will be less than those calculated from Equation (5). The oxygen requirement will be 1.2 - 1.4 times the BOD_5 removed from the waste.

(4) It is important that enough power is supplied to maintain uniform distribution of oxygen throughout the basin, particularly when the waste is low in BOD.

d. The Activated Sludge Process

In the activated sludge process, a primary requirement for effective operation is rapid settling and compaction of the sludge. This means that the sludge loading, lbs BOD/day/lbs MLSS must fall within the flocculation ranges and below the level conducive to the growth of filamentous organisms (see Figure 20). The activated sludge process is schematically shown in Figure 21, which includes all ranges of operation. The design relationships for the various activated sludge modifications are summarized below:

(1) Conventional Activated Sludge

(a) Required basin volume: $F = \frac{24 S_a}{X_a t}$

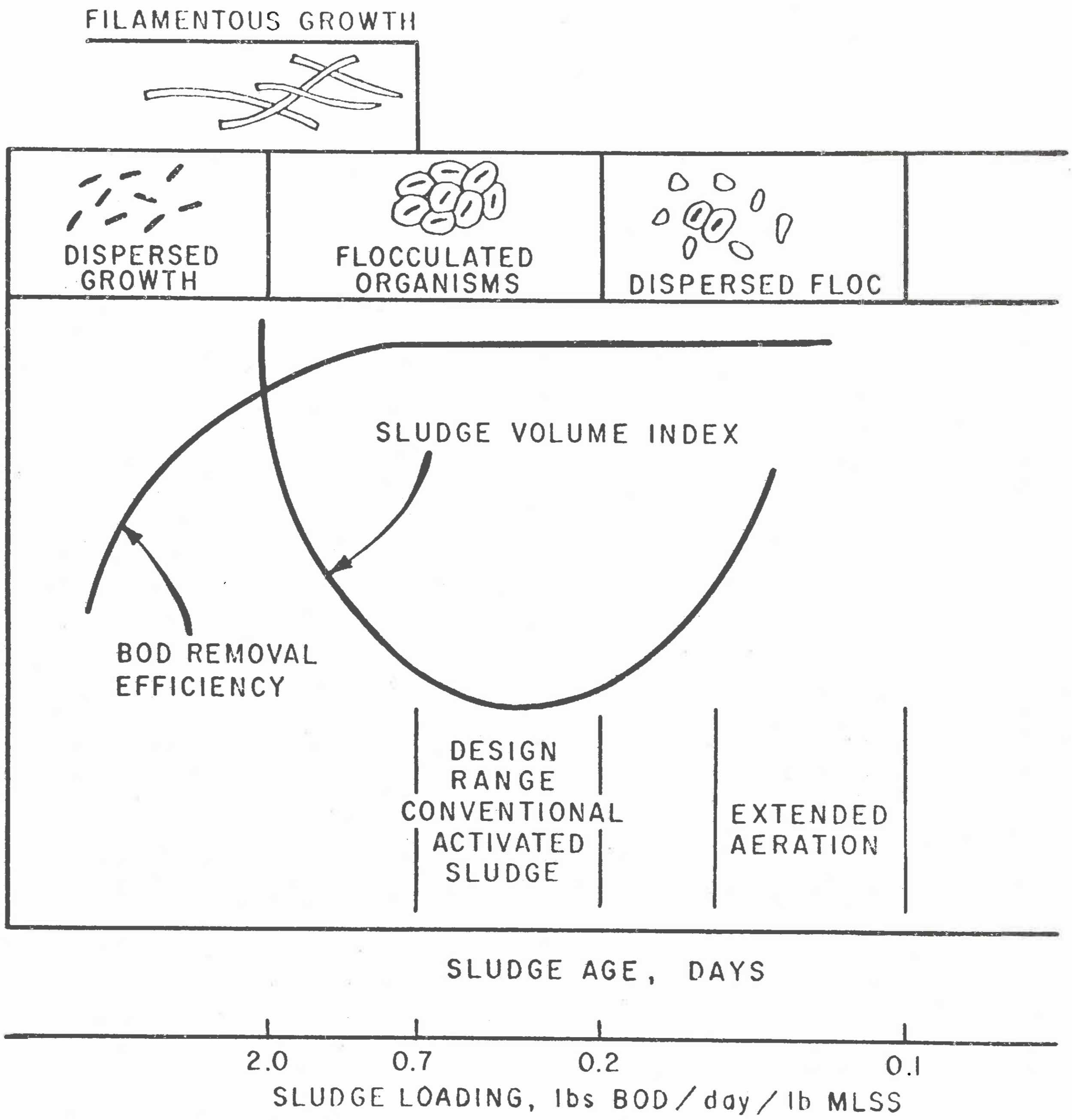

FILAMENTOUS GROWTH
DISPERSED GROWTH
FLOCCULATED ORGANISMS
DISPERSED FLOC
SLUDGE VOLUME INDEX
BOD REMOVAL EFFICIENCY
DESIGN RANGE CONVENTIONAL ACTIVATED SLUDGE
EXTENDED AERATION
SLUDGE AGE, DAYS
2.0
0.7
0.2
0.1
SLUDGE LOADING, lbs BOD / day / lb MLSS

FIGURE 20.

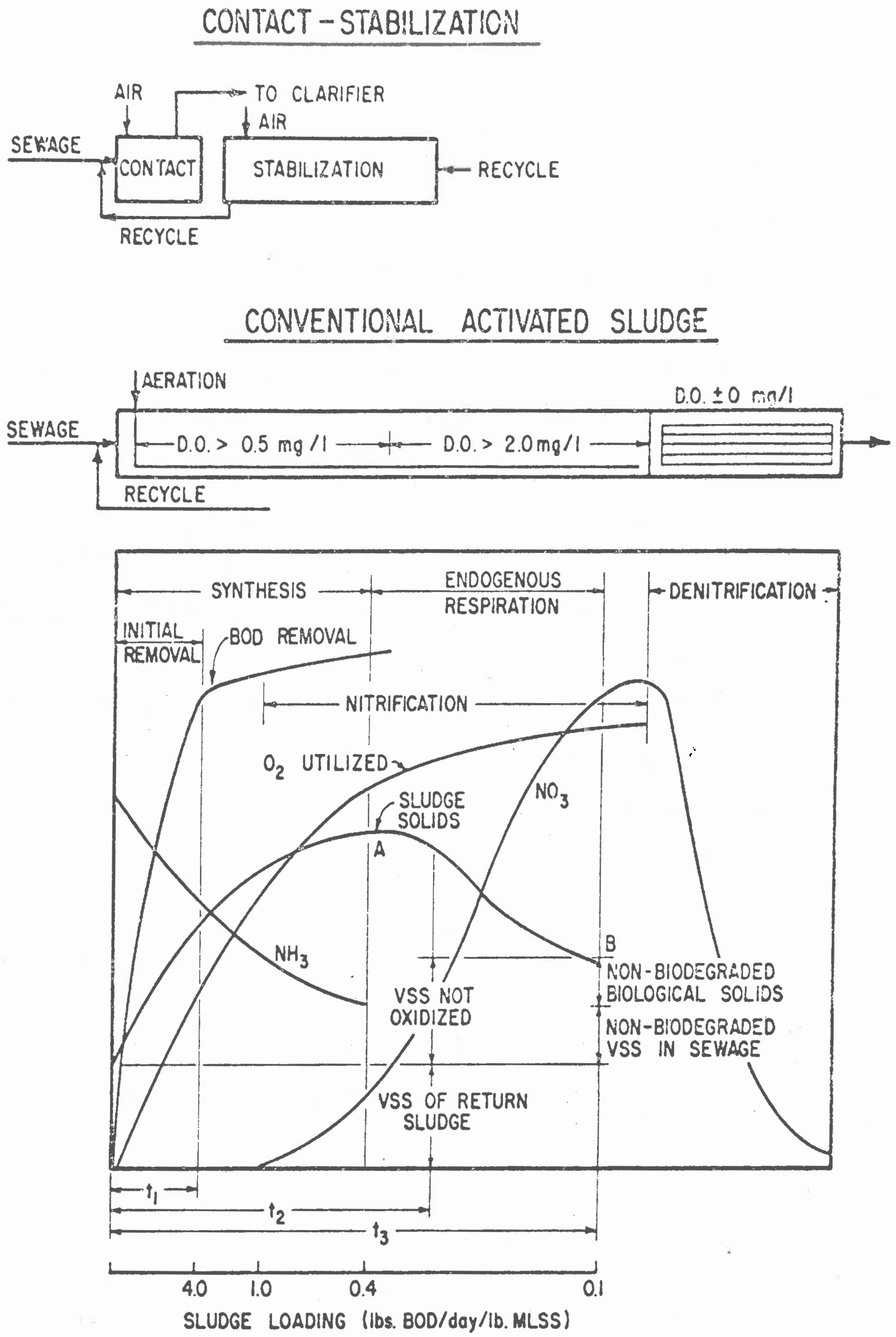


Figure 21.

Schematic Representation of the Activated Sludge Process

F must fall in flocculation range (usually 0.3 - 0.7 lb BOD/day/lb MLSS); X_a is selected based on sludge settling properties; usually between 2,000 and 4,000 mg/l; will usually yield optimum removal.

$$V = A/24\ t$$

where Q is the waste flow in V, the basin volume in (MG).

(b) Oxygen requirements -- Equation 5

(c) Sludge yield -- Equation 2

(d) Nutrient requirements -- Equation 6

(2) Conventional Activated Sludge with Nitrification

(a) Required basin volume -- Equation 4, with sludge age selected from Figure 19.

(b) Oxygen requirements -- Equation 5 plus N oxidized

(c) Sludge yield -- Equation 2.

(3) Extended Aeration

(a) Required basin volume

$$X_a = \frac{a_o S_r}{fb\ (\text{vol. fract})}$$

where

a_o = fraction of BOD converted to degradable solids

f = fraction of degradable solids in system

v = fraction of volatiles

$$V = \frac{X_{a\ (\text{lbs})}}{X_a\ (\text{mg/l})\ 8.34}$$

in which V - MG of basin volume

(b) Oxygen requirements -- Equation 5a.

4. Design - Activated Sludge

(a) It is first necessary to assume a mixed liquor suspended solids concentration; solids in the return sludge and on the return sludge rate. In general, good settling flocculated sludge can be expected to concentrate to 10 - 12,000 mg/l; in the final settling tank with return sludge rates of 20-50% of the average waste flow. Suggested maximum return sludge rate is 150% of the average flow rate. This would indicate an MLSS selection of 3000 mg/l as a conservative operating level.

(b) Since a well-flocculated good settling sludge is essential to effective process operation, the loading level (lbs BOD/day/lb MLSS) must be in the proper range (see Figure 20). For most wastes a value 0.5 based on average waste loadings will yield a conservative design. The required detention time should be calculated from both the loading level, from Equation (1), and the largest value used for design. For a highly degradable waste such as sugar, or brewery, the maximum loading flocculation will probably control; while for a complex waste such as petrochemical, the effluent quality will most likely control the required detention period.

e. Extended Aeration

(1) In the extended aeration process, the retention time or basin volume required is that necessary to oxidize all

of the degradable sludge produced in the system by synthesis. For a soluble waste, the quantity of sludge which must be maintained under aeration to accomplish this oxidation is:

$$X_{av} = \frac{a_o S_r}{fb}$$

in which a_o is the fraction of BOD converted to degradable sludge and can be assumed to be equal to (0.9a).

The total solids under aeration is then:

$$\frac{X_{av}}{\text{Fraction Volatile}}$$

X_{av} = average mixed liquor VSS, mg/l

The volatile fraction for soluble wastes in an extended aeration process will probably average 70%.

Assuming 3500 mg/l mixed liquor suspended solids, the required aeration volume is:

$$V = \frac{X_a \text{ (lbs)}}{3500 \times 8.34} \quad \text{(MG)}$$

(2) The aeration requirements are calculated from Equation 5 (a).

(3) The excess sludge will be the non-biodegradable residue and will be approximately equal to:

$$\Delta X = 0.23\ aS_r - \text{effluent loss}$$

In a continuous completely mixed extended aeration plant with intermittant sludge wasting from the final clarifier both degradable and non-biodegradable sludge will be wasted from the mixture. This will result in a total wastage about twice that calculated from (3).

Table VI -1.

Summary of Design Relationships

The basic relationships applicable to the design of biological waste treatment facilities are summarized below:

(1) BOD or COD removal is a completely mixed basin; aerobic or anaerobic

$$\frac{S_a - S_e}{X_a t} = KS_e \tag{1}$$

(2) Sludge Yield

$\Delta X_v = S_o + aS_r - bX_v$ (conventional systems; aerobic or anaerobic) (2)

(3) Equilibrium solids in an aerobic lagoon or anaerobic flow-through system

$$\frac{S_o + aS_r}{(1 + bt)} = X_a \tag{3}$$

(4) Sludge Age

$$G = \frac{X_v}{\Delta X_v} = \frac{X_v}{aS_r - bX_v} \tag{4}$$

(5) Oxygen requirements (aerobic systems)

$O_2 = a'S_r + b'X_v$ (conventional systems) (5)

$O_2 = a'S_r + b'X_d$ (total oxidation systems) (5a)

(6) Nutrient Requirements

Nitrogen (N) = $0.12\ \Delta X_v + 1.0$ mg/l (6)

Phosphorus (P) = $0.02\ \Delta X_v + 0.5$ mg/l (6a)

Examples -- Alternate Designs for Aerobic Biological Treatment

Flow = 0.75 MGD

Influent BOD_5 = 650 mg/l

K_1 = 0.002

a = 0.55

b = 0.10/day

a' = 0.40

b' = 0.14/day

Influent COD = 765 mg/l

A. Aerobic Lagoon

The required detention period for an effluent soluble BOD_5 of 50 mg/l can be computed from the relationship developed by combining equations (1) and (3). This process will ultimately be modified to extended aeration when higher effluent quality is required.

$$t, \text{ days} = \frac{1}{24\, aKS_e - b}$$

For a = 0.55 and b = 0.15/day, the required detention period is:

$$t = \frac{1}{(24 \cdot 0.55 \cdot 0.002 \cdot 50 - 0.10)} = 0.82 \text{ days} \approx 20 \text{ hrs.}$$

$$V = 0.82\ (0.75) = 0.615 \text{ MG}$$

The equilibrium volatile solids in the lagoon is computed from Equation 3, p. 51.

$$X_v = \frac{S_o + aS_r}{1 + bt}$$

$$= \frac{0 + 0.55\ (600)}{1 + 0.1 \cdot 0.82}$$

$$= 305 \text{ mg/l}$$

The total suspended solids will be:

$$X_a = \frac{X_v}{\text{volatile fraction}}$$

assume: volatile fraction = 80%

$$X_a = \frac{305}{0.80}$$

$$= 381 \text{ mg/l}$$

The BOD_5 of the mixed effluent will be:

$$BOD = 50 + 0.27 \cdot 381 = 153 \text{ mg/l}$$

where 0.27 converts effluent SS to BOD_5 in mg/l

1. Oxygen requirements

The estimated oxygen requirements are:

$$O_2/\text{day} = (0.40 \cdot 600 \cdot 0.75 \cdot 8.34) + (0.14 \cdot 305 \cdot 0.615 \cdot 8.34)$$

(Eq. 5, p.51)

$$= 1500 + 219$$

$$= 1719 \text{ lbs/day} = 71.6 \text{ lb/hr}$$

Using aerators which transfer 2.5 lb 0_2 HP-hr @ 20°C @ 0 mg/l D.O.

C_s = 8.5 mg/l @ 20°C

α = 0.8 and C_L = 1.5 mg/l

$$\text{lbs } 0_2/\text{HP-hr} = N = N_o \frac{(8.5 - 1.5 \text{ mg/l})}{9.0} \, 0.8 \cdot 1.0 = 0.62 \, N_o$$

$$HP = \frac{71.6}{2.5\,(0.62)} = 46.2$$

and the operating power level is:

$$\frac{46.2}{0.615 \cdot 1000} = 0.075 \text{ HP/1000 gallons}$$

∴ Ample for mixing and oxygen diffusion

B. Activated Sludge

Assume a mixed liquor volatile suspended solids concentration of 3000 mg/l and a loading F, of 0.5 lb BOD_5/lb MLVSS day for optimum floc density.

The required detention time from Equation 7 is:

$$t = \frac{24\ S_a}{X_a F} = \frac{24\ (650)}{3000\ (0.5)}$$

$$t = 10.4 \text{ hrs} = 0.43 \text{ days}$$

The aeration basin volume is:

$$V = 0.75\ (0.43) = 0.322 \text{ MG}$$

From Equation (1):

The effluent BOD_5 (soluble, after clarification) is:

$$S_e = \frac{S_a}{1 + 24KX_a t} = \frac{650}{1 + 24 \cdot 0.002 \cdot 3000 \cdot 0.43}$$

$$S_e = 10 \text{ mg/l}$$

1. Oxygen Requirements

(Assume 75% of volatile solids are degradable)

$$\text{lbs } O_2/\text{day} = (0.40 \times 640 \times 0.75 \times 8.34) + (0.14 \times 0.322 \times 3000 \times 8.34)$$

$$= 1600 + 1130 = 2730 \text{ lb } O_2/\text{day} = 114 \text{ lb } O_2/\text{hr}$$

The required aeration horsepower is (assuming 0.015 HP/1000 gallons and $N_o = 2.5$ lb O_2/HP-hr)

$$HP = \frac{114}{0.62\ (2.5)} = 74.3$$

$$\frac{74.3}{322} = 0.229 \text{ HP}/10^3 \text{ gallons}$$

2. Excess Sludge

(Based on 85% volatile solids)

$$\text{lbs VSS/day} = (0.55 \cdot 640 \cdot 0.75 \cdot 8.34) - (0.1 \cdot 0.322 \cdot 3000 \cdot 8.34)$$

$$= 2200 - 805 = 1395 \text{ lbs/day}$$

$$\text{or TSS} = \frac{1395}{0.85} = 1640 \text{ lbs/day}$$

$$\text{Sludge age} = \frac{X_v}{aS_r - bX_v} = \frac{8050}{1395} = 5.76 \text{ days}$$

C. Aerobic - Facultative Lagoon

The volatile solids level in the lagoon is assumed to be 50 mg/l. Field experience has indicated that the expected effluent suspended solids concentration will range from 50 - 100 mg/l depending on the power level in the lagoon.

The retention period for an effluent soluble BOD of 30 mg/l will be: (from Eq. 1, p. 51)

$$t = \frac{S_r}{24\ KX_vS_e} = \frac{620}{24 \cdot 0.002 \cdot 50 \cdot 30}$$

$$t = 8.6 \text{ days}$$

The aeration basin volume will be:

$$V = 0.75\ (8.6) = 6.45 \text{ MG}$$

1. Oxygen Requirements

In the aerobic-facultative lagoon the biological solids concentration is low and BOD from anaerobically decomposing bottom sludge is diffusing into the liquid so the oxygen requirements for these lagoons are related to BOD removal alone lbs O_2/day = a" lbs BOD removed/day. The value of a" varies from 0.9 - 1.4 for most industrial wastes. For this example a" is assumed to be 1.2. (a" = lbs O_2 required/lbs BOD removed).

$$\text{lb } O_2\text{/day} = (1.2 \cdot 620 \cdot 0.75 \cdot 8.34) = 4650 \text{ lbs/day} = 194 \text{ lb/hr}$$

Assume: N_o = 2.5 lb 0_2 HP-hr

$$HP = \frac{194}{2.5\ (0.62)} = 125$$

and the power level will be:

$$\frac{125}{6.45 \cdot 1000} = 0.0194\ \text{HP/1000 gal - ample for mixing}$$

2. Excess Sludge

(85% volatile solids basis) from Equation 2, p. 51.

$$\text{lbs VSS/day} = [(0.55)(620)(0.75)(8.34)]-[(0.1)(6.45)(50)(8.34)]$$

$$= 2140 - 269 = 1871\ \text{lbs/day}$$

$$\text{or TSS} = \frac{1871}{0.85} = 2200\ \text{lbs/day}$$

D. Extended Aeration

Assume an effluent soluble BOD of 15 mg/l.

$$X_{av} = \frac{a_o S_r}{fb}$$

where a_o = conversion to degradable sludge and f = fraction of volatile solids which are degradable:

$$a_o = 0.88a = 0.88 \cdot 0.55 = 0.48$$

$$f = 0.58$$

$$X_{av} = \frac{0.48\ (635)(0.75)(8.34)}{(0.58)(0.10)}$$

$$= 33{,}000\ \text{lbs}$$

if the sludge is 85% volatile

$$\text{Total MLSS} = \frac{33{,}000}{0.85} = 38{,}500\ \text{lbs}$$

at 4000 mg/l MLSS the aeration volume is:

$$V = \frac{38,500}{4000 \times 8.34} = 1.15 \text{ MG}$$

$$\text{the organic loading} = \frac{635\ (0.75)(8.34)}{33,000} = 0.12 \text{ lb BOD/lb MLVSS day}$$

1. Aeration Requirements

$$\text{lbs } 0_2\text{/day} = 0.4\ (635 \cdot 0.75 \cdot 8.34) + 1.4 \times 0.48\ (635 \times 0.75 \cdot 8.34)$$

$$= 1590 + 2650 = 4240 \text{ lbs/day} = 177 \text{ lbs/hrs}$$

Using $N_o = 2.60$ lbs 0_2/HP-hr

$$HP = \frac{177}{2.6\ (0.62)} = 110 \text{ HP}$$

Power Level:

$$\frac{110}{(1150)} = 0.096 \text{ HP/1000 gallons}$$

Assuming 50 mg/l suspended solids in the effluent and for an endogenous sludge an mg/l BOD_5/mg/TSS ratio of 0.15, the total effluent BOD_5 is:

Total Effluent BOD = 15 mg/l + 50 (0.15)= 22 mg/l

The excess sludge = $aS_r - bX_d$ - Effluent Loss

$= 0.55 \times 635 \times 8.34 \times 0.75 - 0.1 \times 0.75 \times 0.85 \times 4000 \times 0.8 \times 8.34 - 50 \times 0.75 \times 8.34$

$= 2180 - 1705 - 313$

= 162 lbs/day of non-volatiles or approximately 324 lbs/day total

$$\text{The Sludge Age} = \frac{33,000}{324} = 100 \text{ days}$$

Table VI - 2.

Design Parameters - Example Problem

Parameter	Activated Sludge	Extended Aeration	Aerobic Lagoon	Aerobic Faculta. Lagoon
lb BOD/lb MLVSS day	0.5	0.120	2.55	1.51
Aeration Volatile Solids mg/l	3000	3400	305	50
BOD Removal Efficiency %	98.5	96.6	76.5	93
Effluent				
Soluble BOD mg/l	10	15	50	30
Total BOD mg/l	14	22	153	45
SS mg/l	20	50	381	50
COD mg/l	125		268	145
non-biodegradable COD/mg/l	115	115	115	115
Sludge Yield (TSS) lb/lb BOD removed	0.41	0.082	0.60	0.565
O_2 Requirement % BOD removed	67.3	102	46.0	120
Aeration HP	74.3	110	46.2	125
Power Level - HP/1000 gallons	0.229	0.096	0.075	0.0194
Sludge Age, days	5.76	100	1.95	------

Chapter VII.

TRICKLING FILTRATION

Procedure for the Design of a Trickling Filter

A. Data to be Collected

1. Volume and characteristics of waste
2. Determine whether recirculation is necessary

B. Pre or Primary Treatment

1. The BOD applied to the filter should generally not exceed 500 mg/l in order to insure aerobic conditions. Recirculation should be employed when wastes have BOD values in excess of this value.

2. When a filter packing with a high specific surface is used, a minimum hydraulic loading is essential to avoid the accumulation of excessive biological growths and resulting filter ponding.

3. The pH of the wastewater applied to the filter should be such that the excess alkalinity or acidity is neutralized in the top few feet of the filter media. Excessive acidity or alkalinity can frequently be neutralized by recirculation of filter effluent.

C. Design Procedure (a)

1. Collect data to develop a relationship between BOD removal, depth, and hydraulic loading. At least three different hydraulic loading rates should be tried.

2. Develop a plot of depth vs % BOD remaining for the different hydraulic loading rates (Figure 22).

3. Determine the slopes of the lines in Figure 22.

4. Plot Q vs slopes and determine 'n' (Figure 23).

5. Develop a plot D/Q^n vs % BOD remaining and the slope of the line gives the coefficient 'K' (Figure 24).

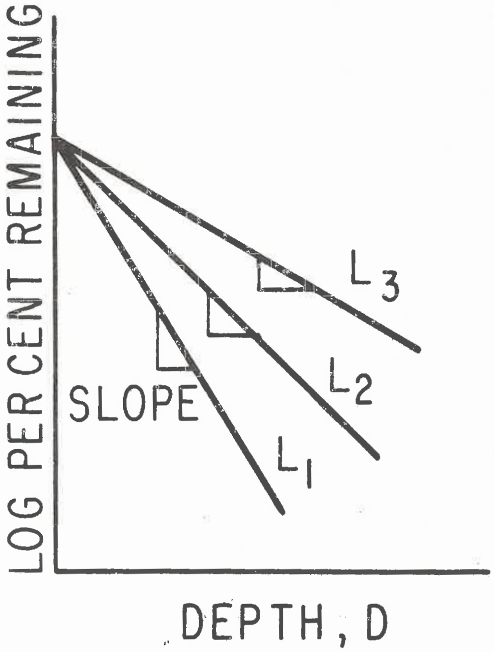


FIG. 22

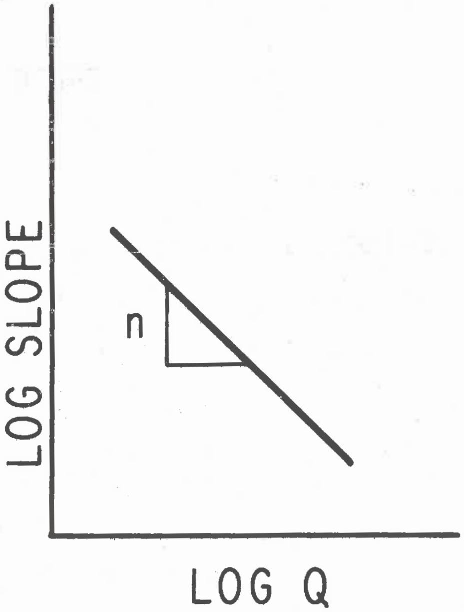


FIG. 23

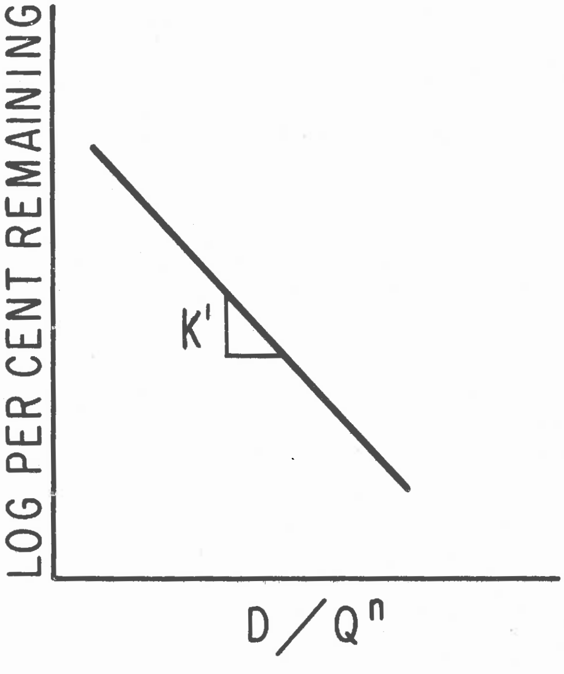


FIG. 24

6. The design equation for the % BOD removal is given by

$$\% \text{ BOD removal} = 1 - e^{-K'D/Q^n}$$

7. Selecting a suitable depth, determine the flow rate necessary for a given % BOD removal.

8. From the flow rate and the total volume of flow to be treated, determine the size of the filter.

Design Procedure (b)

1. In cases where experimental data are not available, a first estimate of filter requirements can be made using the coefficients shown in Table VII-1 with Equations 1 and 2.

2. Select a filter depth; for rock filters, depths will usually vary from 3-10 feet, while for plastic packing, depths of up to 40 feet have been employed.

3. Using Equation 1, compute the hydraulic loading to the filter; an alternate design might consider recirculation as calculated from Equation 2.

4. The total filter area required is computed:

$$\text{Area (acres)} = \frac{\text{wastewater flow including recirculation (MGD)}}{\text{hydraulic loading (MGAD)}}$$

5. If the filter is used as a roughing filter with high organic loadings (50-60% BOD removal), the same procedure (Steps 1-4) are followed.

Example -- Design Procedure (a)

The following data was obtained in a trickling filter on the treatment of settled sewage.

TABLE VII-1.

Trickling Filtration Design

Design Formulations:

Without recirculation

$$\frac{S}{S_o} = e^{-K'D/Q^n} \qquad (1)$$

With recirculation

$$\frac{S}{S_o} = \frac{e^{-K'D/Q^n}}{(1 + N) - N e^{-K'D/Q^n}} \qquad (2)$$

Waste	Filter media	Specific surface ft^2/ft^3	K'	n
Sewage	surfpac	28.0	0.079	0.5
Fruit canning	surfpac	28.0	0.0177	0.5
Box board	surfpac	28.0	0.0197	0.5
Steel coke plant	surfpac	28.0	0.0211	0.5
Textile	surfpac	28.0	0.0156	0.5
Textile	surfpac	28.0	0.0394	0.5
Textile	surfpac	28.0	0.0268	0.6
Pharmaceutical	surfpac	28.0	0.0292	0.5
Slaughterhouse	surfpac	28.0	0.0246	0.5

Depth feet	% BOD remaining (average) Hydraulic Loading gpm/ft^2 1	2	3	4
6	62	70	75	80
12	36	46	57	63
18	24	32	43	48.5
24	14	22	32	39

1. Correlate the data and develop a relationship between BOD removal, depth, and hydraulic loading.

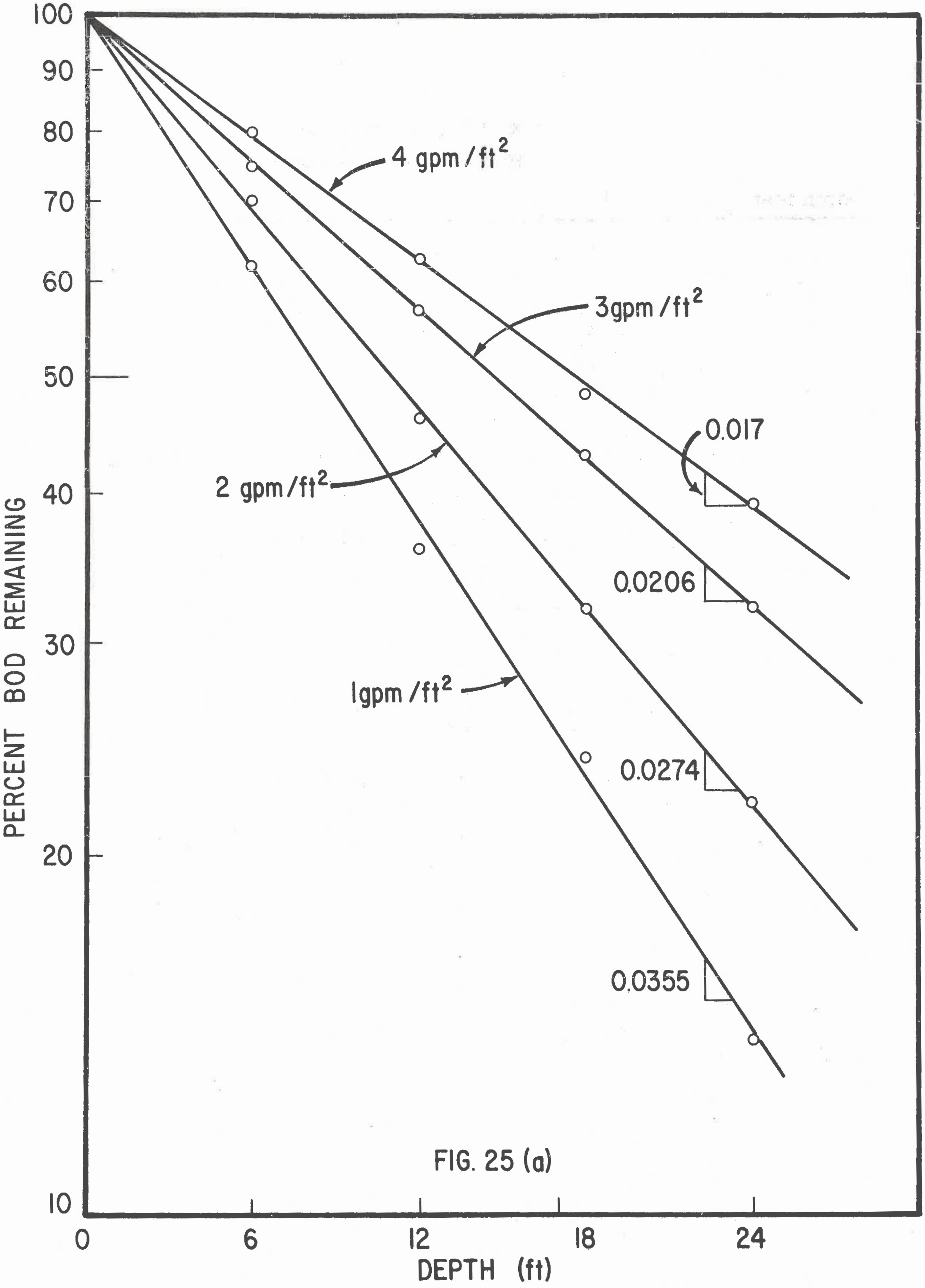
2. Design a filter to obtain 80 % BOD reduction from 5 MGD of settled sewage with an initial BOD of 220 mg/l, using a depth of 20 feet

a. Without recirculation

b. With a recirculation ratio of 1.5

Design equation $\frac{S}{S_o} = e^{-0.082\, D/Q^{.5}}$

The correlation of the data is shown in Figures 25 (a), (b), (c).



100
90
80
70
60
50
40
30
20
10
0
6
12
18
24
DEPTH (ft)
PERCENT BOD REMAINING
4 gpm/ft²
3gpm/ft²
2 gpm/ft²
Igpm/ft²
0.017
0.0206
0.0274
0.0355

FIG. 25 (a)

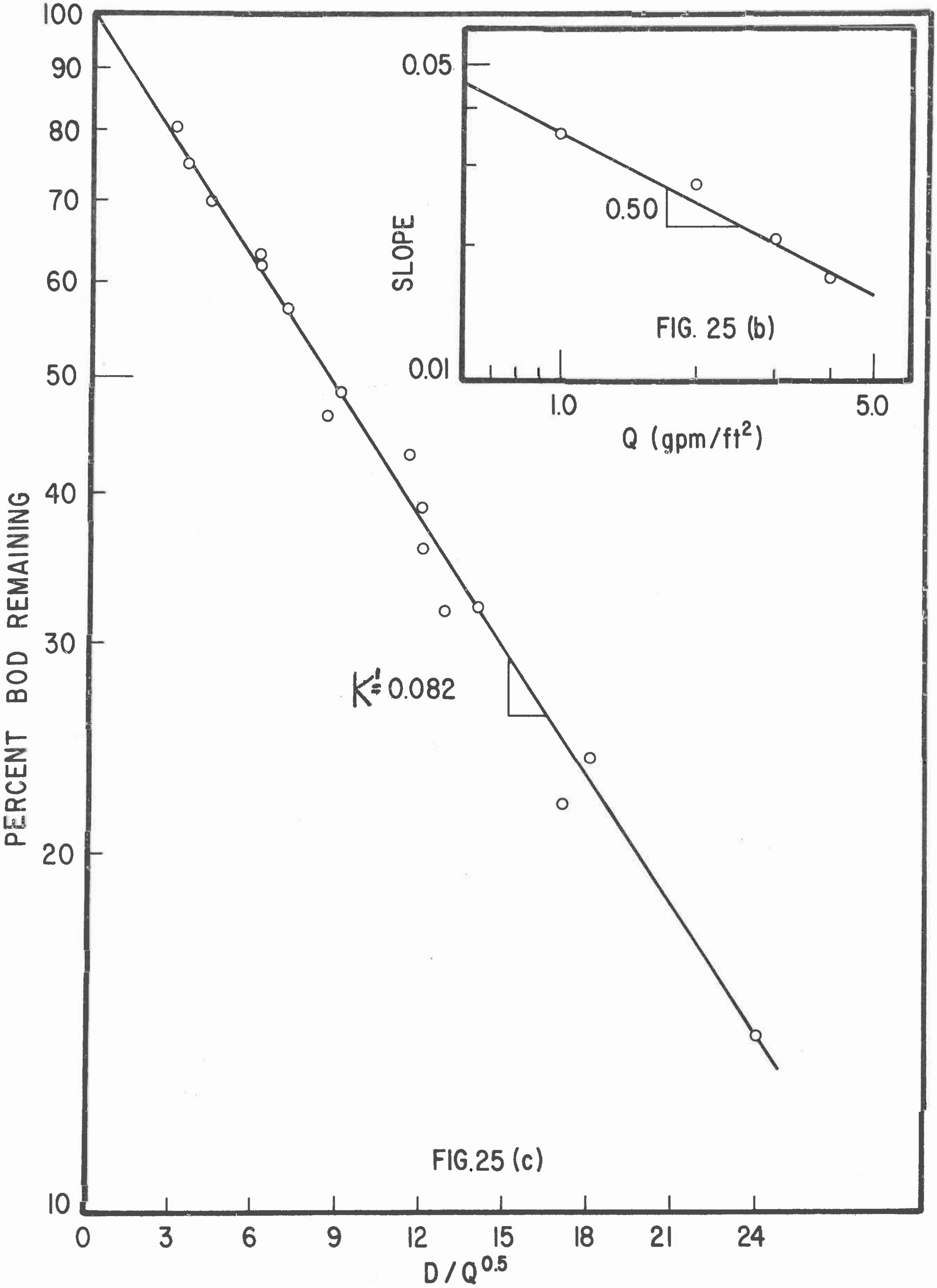


FIG. 25 (b)

FIG. 25 (c)

TABLE VII-2.

Data for the Computation of 'K'

Depth, feet	Q gpm/ft^2	$Q^{0.5}$	$D/Q^{.5}$	% BOD remaining
6	1.0	1.0	6.0	62
12	1.0		12.0	36
18	1.0		18.0	24
24	1.0		24	14
6	2.0	1.414	4.25	70
12	2.0		8.50	46
18	2.0		12.75	32
24	2.0		17.00	22
6	3.0	1.732	3.47	75
12	3.0		6.94	57
18	3.0		11.41	43
24	3.0		13.88	32
6	4.0	2.00	3.0	80
12	4.0		6.0	63
18	4.0		9.0	48.5
24	4.0		12.0	39

2 (a) without recirculation

Depth 20 feet

$$\frac{S}{S_o} = \frac{1}{e^{0.082\, D/Q^{0.5}}}$$

for 80% BOD removal

$$0.2 = \frac{1}{e^{(0.082)(20)/Q^{0.5}}}$$

$$e^{1.64/Q^{0.5}} = 5$$

$$1.64/Q^{0.5} = 1.61$$

$$Q^{0.5} = \frac{1.64}{1.61} = 1.02$$

$$Q = 1.0 \text{ gpm/ft}^2$$

$$\text{or } 1440 \text{ gpd/ft}^2$$

$$\text{Area of filter required} = \frac{5 \times 10^6}{1440}$$

$$= 3475 \text{ ft}^2$$

$$\text{Diameter of filter} = \sqrt{\frac{3475}{0.785}}$$

$$= 66.5 \text{ feet}$$

2 (b) with a recirculation factor of 1.5

$$S_o = \frac{S_a + N S_e}{1 + N}$$

$$= \frac{220 + 1.5\,(44)}{1 + 1.5} = 114.5$$

$$S_e/S_o = 44/114.5 = 0.385$$

$$e^{1.64/Q^{0.5}} = \frac{S_o}{S_e} = 2.60$$

$$1.64/Q^{0.5} = 0.955$$

$$Q^{0.5} = \frac{1.64}{0.955} = 1.72$$

$$Q = 2.96 \text{ gpm/ft}^2$$

Area of filter required $= \dfrac{12.5 \text{x } 10^6}{(2.96)\ 1440}$

$$= 2940 \text{ ft}^2$$

Diameter of filter $= \sqrt{\dfrac{2940}{0.785}}$

$=$ 61 feet or 60 feet

Chapter VIII.

ANAEROBIC SYSTEM

Procedure for the Design of Anaerobic Processes

A. Data to be collected

1. Waste flow (MGD) and fluctuations
2. Waste characteristics
    a. Suspended solids and volatile suspended solids, mg/l
    b. COD or TOC, mg/l
    c. Nitrogen (Total Kjeldahl nitrogen and organic nitrogen), mg/l
    d. Phosphorus, mg/l
    e. Alkalinity or acidity, mg/l
    f. Presence of heavy metals, oils and toxics, etc.

B. Pre or Primary Treatment

1. If suspended solids are present in excess of approximately 250 mg/l, solids separation devices such as a sedimentation, or flotation may be provided.

2. If oil, grease or flotables exceed 100 mg/l, a skimming tank or separator should be provided.

3. Heavy metals (Cu, Zn, Ni, Pb) should be removed prior to biological treatment.

4. Neutralization of excess alkalinity or acidity should be provided; pH control is critical in the anaerobic process.

C. Design of the Anaerobic Process

1. Estimate the coefficients a, b, and k of the process for the waste from pilot plant or laboratory scale studies (See Figures 26 and 27).

2. Combining the sludge growth equation and substrate removal equation, get a relationship between t, k, a, b, and S_e as follows:

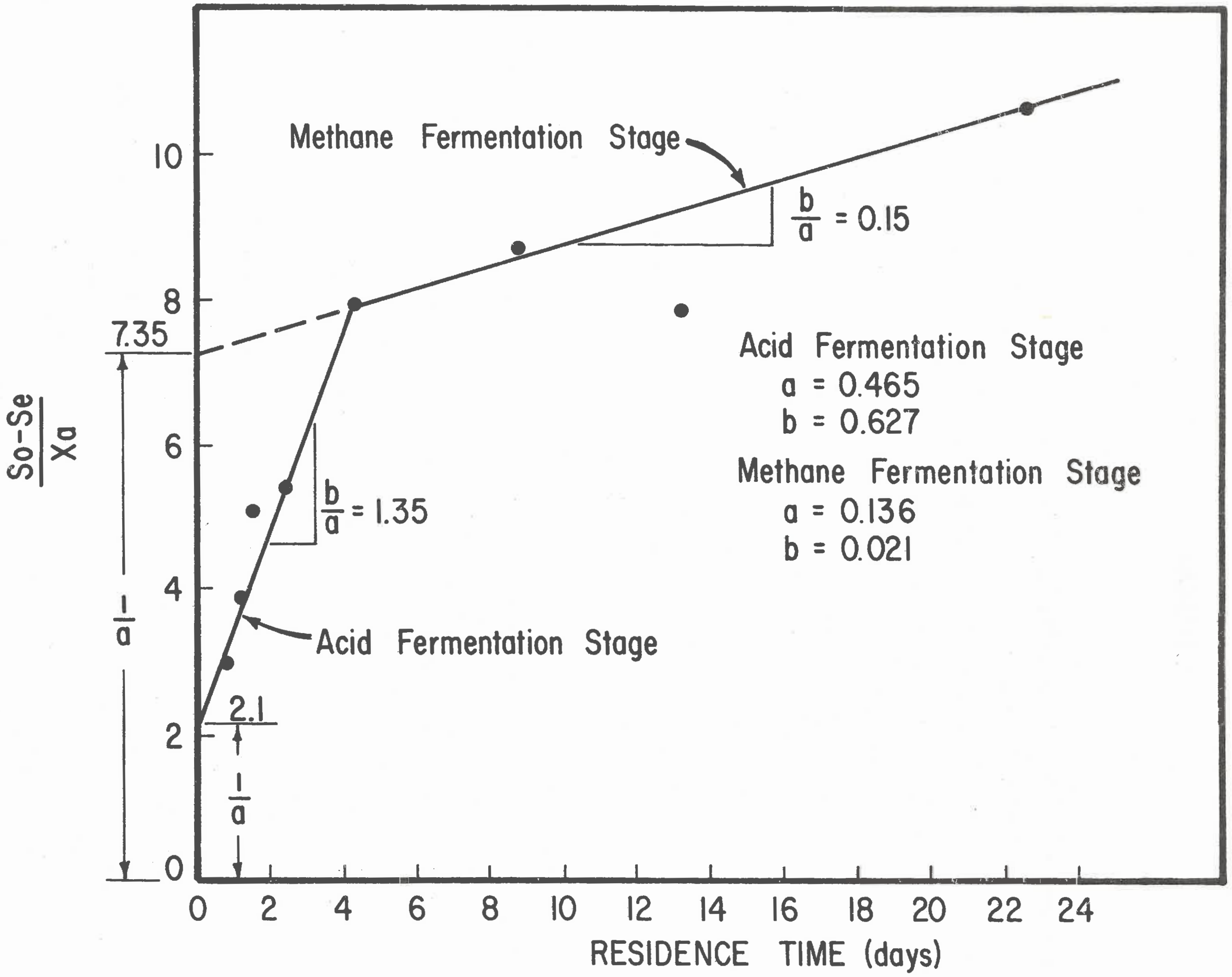

Methane Fermentation Stage
$\frac{b}{a} = 0.15$
Acid Fermentation Stage
a = 0.465
b = 0.627
Methane Fermentation Stage
a = 0.136
b = 0.021
$\frac{b}{a} = 1.35$
Acid Fermentation Stage
2.1
$\frac{1}{a}$
7.35
$\frac{1}{a}$
$\frac{So-Se}{Xa}$
10
8
6
4
2
0
0
2
4
6
8
10
12
14
16
18
20
22
24
RESIDENCE TIME (days)

FIGURE 26

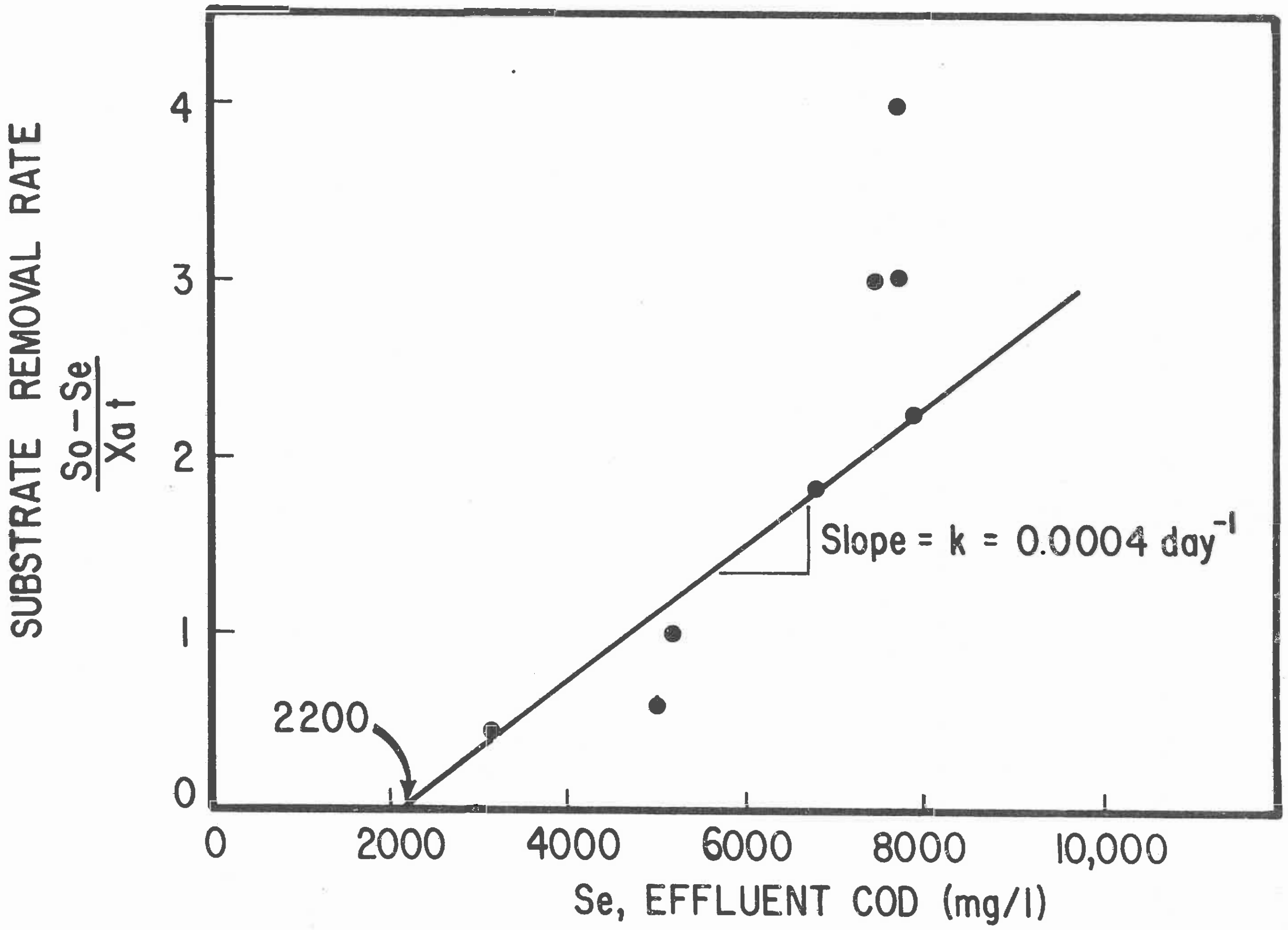

SUBSTRATE REMOVAL RATE
$\frac{So - Se}{Xa\,t}$
4
3
2
1
0
0
2000
4000
6000
8000
10,000
Se, EFFLUENT COD (mg/l)
2200
Slope = k = 0.0004 day^{-1}

FIGURE 27

Sludge Growth Equation $\frac{S_o - S_e}{X_a} = \frac{1}{a} + \frac{b}{a} t$ (1)

Substrate Removal Equation $\frac{S_o - S_e}{X_a t} = k \; S_e$

or

$$\frac{S_o - S_e}{X_a} = k \; S_e \; t \qquad (2)$$

3. Determine the detention period from the Equation 3

4. Calculate X_a of the system using the relationship $X_a = \frac{a(S_o - S_e)}{1 + b\,t}$

Example--Anaerobic System

The following data was obtained from laboratory studies:

Characteristic	Residence Time, days							
	0.75	1.28	1.68	2.37	4.33	8.81	13.2	22.5
Influent COD, mg/l	9820	10410	10410	10500	10500	10430	10370	10070
Effluent COD, mg/l	7780	5700	7720	7930	6790	5220	5060	3210
Mixture								
SS, mg/l	780	792	621	525	527	627	752	740
VSS, mg/l	683	747	528	477	467	597	680	648
gas CH_4(%)	25.4	26.1	27.1	24.6	41.6	52.7	56.7	68.7
gas ft^3 STP/day	0.834	0.905	0.813	0.553	0.569	0.577	0.476	0.361
liter/day feed	42.6	43.2	33.0	28.0	15.3	8.71	4.99	3.42

Design a flow-through process for 90% of the removable COD and estimate the gas production.

The laboratory data is reduced as shown in Tables I and II. For plotting the evaluation of the process constants a, b, and k are shown in Figures 26 and 27.

TABLE VIII - 1

time days	S_o	S_e	S_o-S_e	X_a (VSS)	$\frac{S_o-S_e}{X_a}$	$\frac{S_o-S_e}{X_a t}$
0.75	9820	7780	2040	683	2.99	4.00
1.28	10415	7500	2910	747	3.90	3.05
1.68	10410	7720	2690	528	5.10	3.04
2.87	10500	7930	2570	477	5.38	2.27
4.33	10500	6790	3710	467	7.93	1.83
8.81	10430	5220	5210	597	8.70	0.988
13.2	10370	5060	5310	680	7.80	0.59
22.5	10070	3210	6860	642	10.58	0.47

TABLE VIII- 2

COD removed mg/l	feed liters day	lbs of COD removed per day	ft^3 of gas	CH_4 %	ft^3 of CH_4	calculated ft^3 of CH_4 per lb COD removed
2040	42.6	0.191	0.834	26.4	0.22	
2910	43.2	0.276	0.905	26.1	0.236	
2690	33.0	0.195	0.813	27.1	0.22	
2570	28.0	0.159	0.553	24.6	0.136	
3710	15.3	0.125	0.569	41.6	0.237	
5210	8.71	0.100	0.577	52.7	0.302	3.02
5310	4.99	0.058	0.476	56.7	0.27	5.65
6860	3.42	0.051	0.361	68.7	0.246	5.50

Gas Production G = 5.58 (S_r - 1.42 X_P)

where G = Gas produced per lb of COD

S_r = lb of COD removed

X_P = lb of solids in the reactor

Total COD (average) = 10,300 mg/l

Non removable COD = 2200 mg/l

$\therefore$ Removable COD = 10,300 - 2200 = 8,100

COD to be removed = (0.9)(8100) = 7290 mg/l

The methane fermentation stage is controlling

$\therefore$ a = 0.136

b = 0.021

k = 0.0004 day $^{-1}$

combining growth equation and removal equation for a flow-through system we have

$$t = \frac{1}{(ak\,S_e - b)}$$

$$t = \frac{1}{(.0004)(.136)(800) - .021}$$

$$= \frac{1}{0.044 - .021}$$

$$= \frac{1}{0.023} = 43.3 \text{ days}$$

$$X_a \text{ of the system} = \frac{(0.136)(7920)}{1 + (.021)\,43.3}$$

$$= \frac{1078}{1.909} = 565 \text{ mg/l}$$

Chapter IX.

ION EXCHANGE

Procedure for the Design of an Ion Exchange Process

A. Data to be obtained

1. Volume of waste and the concentration of anions and cations present.

B. Design Procedure

1. Develop a breakthrough curve as in Figure 28(a) and select an operating time to breakthrough.
2. Develop the following relationships:
    (a) Regeneration efficiency and resin utilization (Figure 28b).
    (b) Regeneration efficiency vs regenerant added (Figure 28c).
    (c) Total resin capacity vs regenerant used (Figure 28d).
3. Determine the total amount of ions to be removed.
4. Select a regenerant requirement and obtain regneration efficiency from Figure 28c.
5. From Figure 28b, obtain resin utilization for the regeneration efficiency obtained in step 4.

Example -- Ion Exchange

Design an anion exchange system to treat 150,000 gpd of water with a chrome content of 90 mg/l. Laboratory investigations are shown in Figures 29 through 32. Assume 24-hours operation; capacity of resin = 1.08 eq/l.

1. Chrome content to be removed

$$= (1.5 \times 10^{+5} \text{ gal.}) (340 \text{ mg/gal.})$$

$$= 5.1 \times 10^{7} \text{ mg}$$

$$= \frac{5.1 \times 10^{7}}{52 \times 10^{3}} = 980 \text{ eq/day}$$

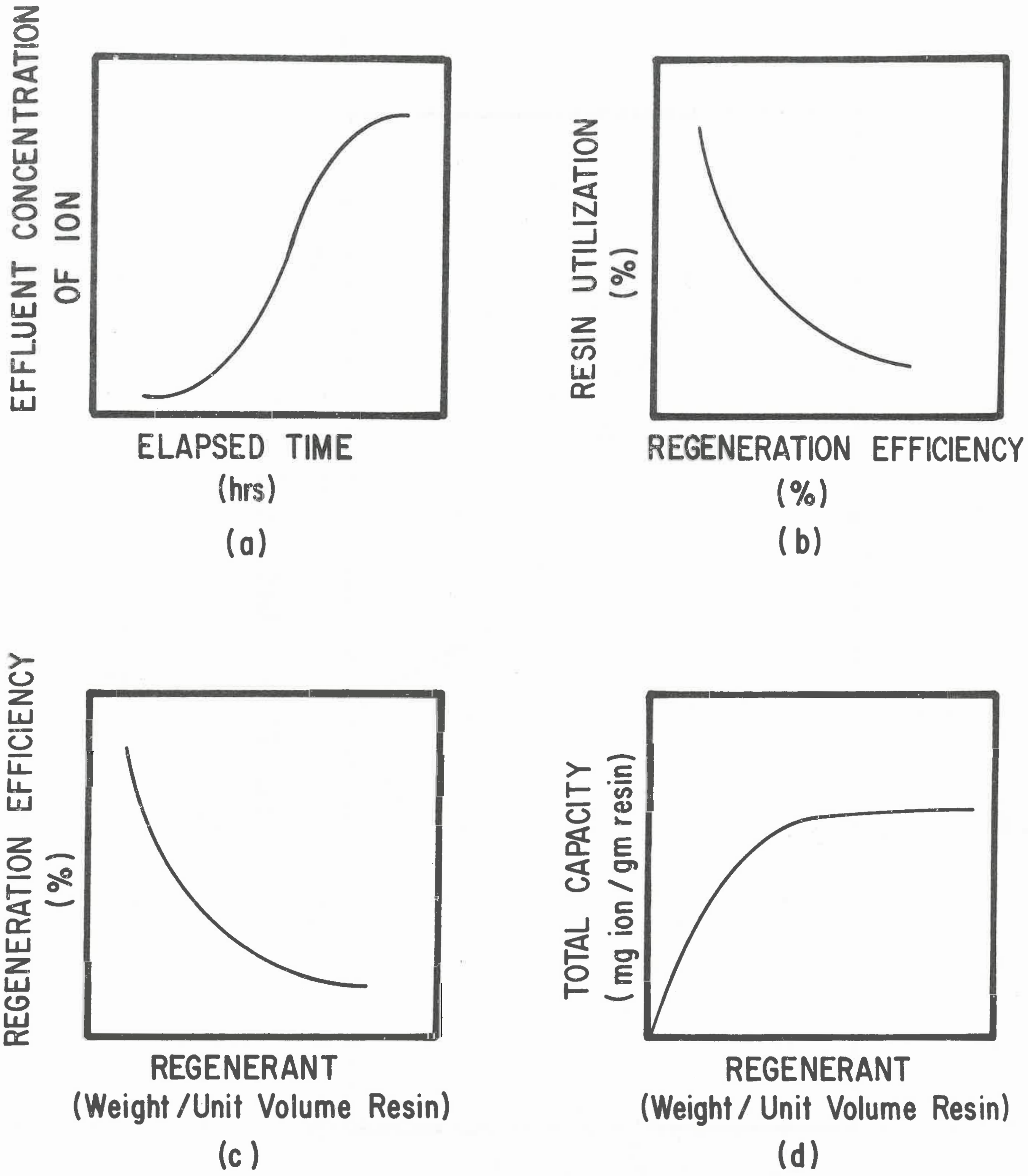

EFFLUENT CONCENTRATION OF ION
ELAPSED TIME (hrs)
(a)
RESIN UTILIZATION (%)
REGENERATION EFFICIENCY (%)
(b)
REGENERATION EFFICIENCY (%)
REGENERANT (Weight / Unit Volume Resin)
(c)
TOTAL CAPACITY (mg ion / gm resin)
REGENERANT (Weight / Unit Volume Resin)
(d)

Figure 28

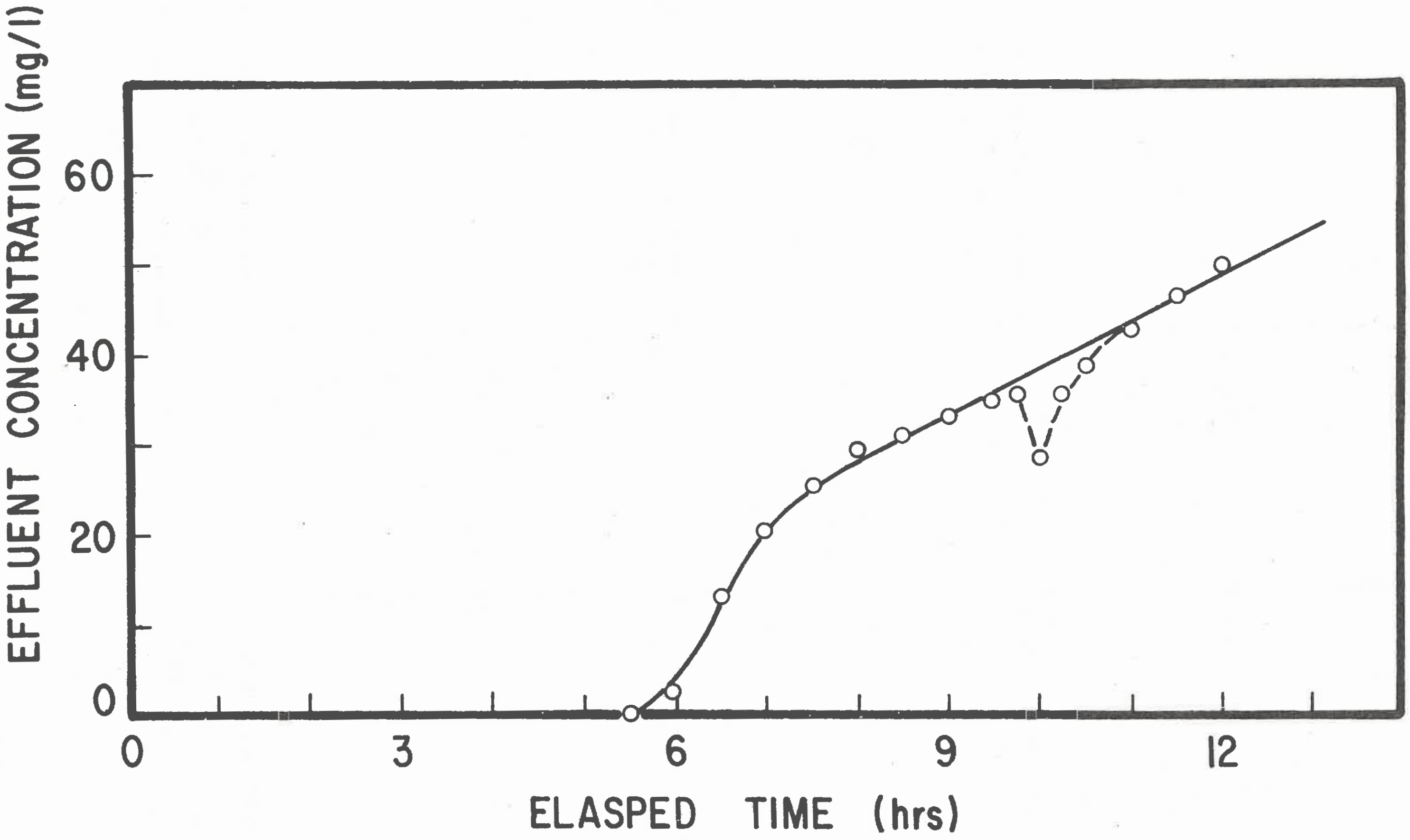

EFFLUENT CONCENTRATION (mg/l)
60
40
20
0
0
3
6
9
12
ELASPED TIME (hrs)

FIG. 29

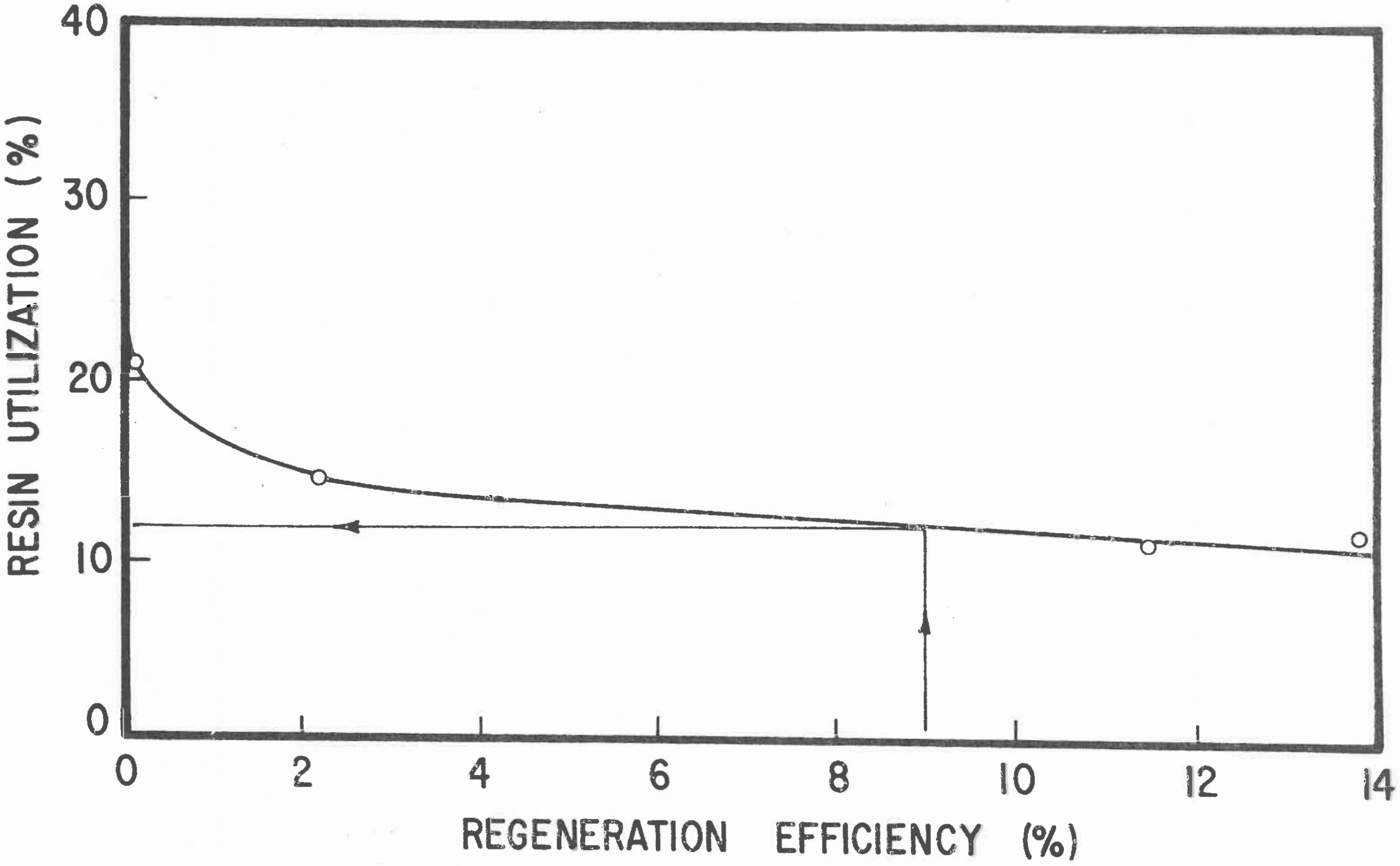

RESIN UTILIZATION (%)
40
30
20
10
0
0
2
4
6
8
10
12
14
REGENERATION EFFICIENCY (%)

FIG. 30

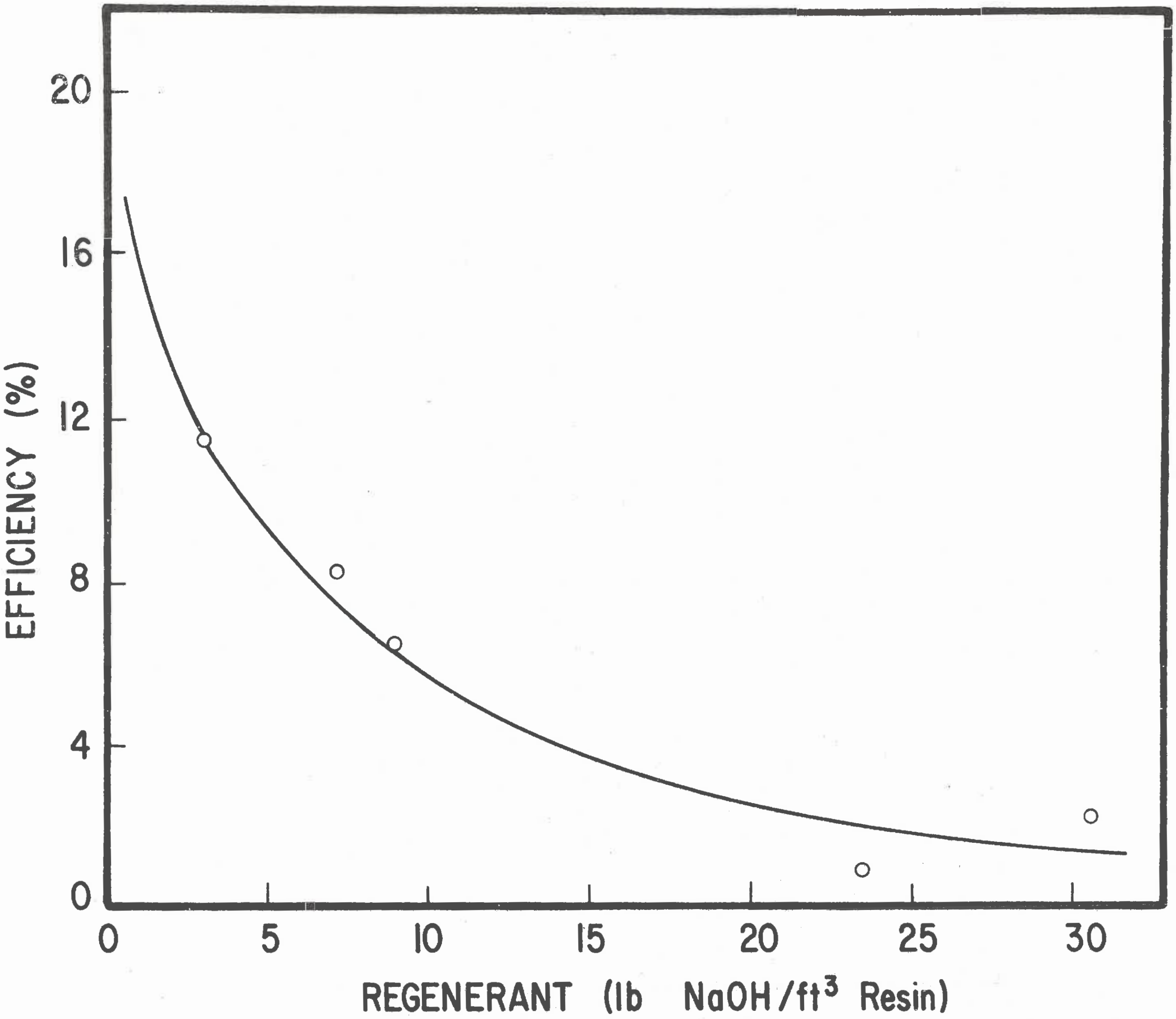

EFFICIENCY (%)
0
4
8
12
16
20
0
5
10
15
20
25
30
REGENERANT (lb NaOH/ft³ Resin)

FIG. 31

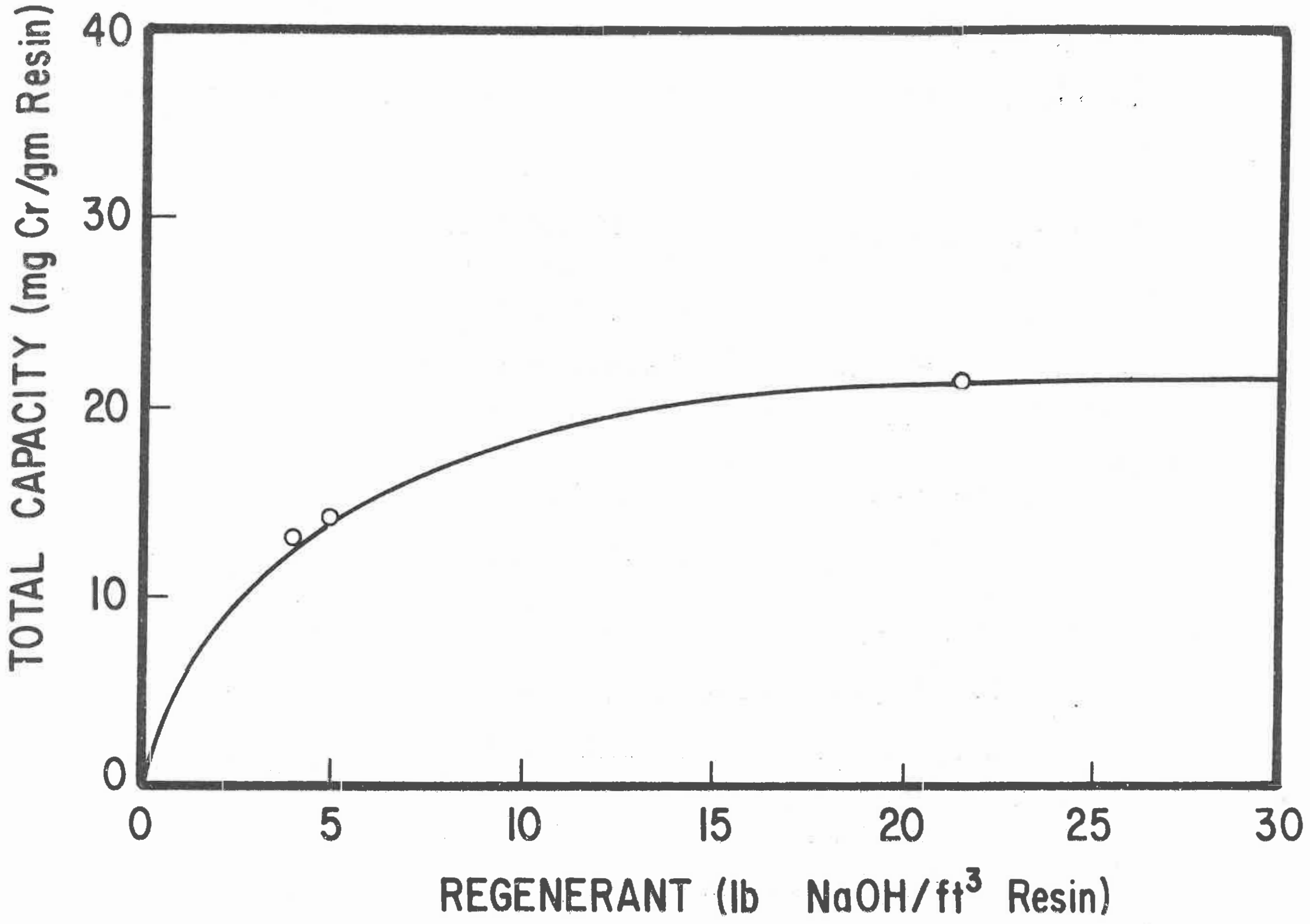

TOTAL CAPACITY (mg Cr/gm Resin)
0
10
20
30
40
0
5
10
15
20
25
30
REGENERANT (lb NaOH/ft^3 Resin)

FIG. 32

2. Selecting NaOH requirement of 5 lbs/ft^3 of resin (Figure 31) regenerant efficiency = 9%

From Figure 30, for a regenerant efficiency of 9%, resin utilization = 12%.

$\therefore$ Total capacity reqd. $= \dfrac{980}{0.12} = 8150$ eq/day

Total capacity = (volume of resin) (capacity of resin)

Capacity of resin $= 1.08\ \text{eq/l} = 1.08\ \dfrac{\text{eq}}{\text{l}} \cdot \dfrac{28.32\ \text{l}}{\text{ft}^3} =$

$306\ \text{eq/ft}^3$

$\therefore$ Volume of resin $= \dfrac{8150\ \text{eq/day}}{306\ \text{eq/ft}^3}$

$= 266\ \text{ft}^3\text{/day}$

3. Use two units operating alternately for 4 hours.

Resin volume/unit $= \dfrac{266}{6} = 44.5\ \text{ft}^3$

4. a Flow rate = 104 gpm

$= \dfrac{104\ \text{gpm}}{44.5\ \text{ft}^3} = 2.34\ \text{gpm/ft}^3$

b Rinse water:

Use 50 gal/ft^3 of resin at a flow rate of 1.5 gal/min/ft^3

Time required $= \dfrac{50\ \text{gal/ft}^3}{1.5\ \text{gpm/ft}^3} = 33$ min.

c Regeneration

Regenerant requirement using 5 lb of NaOH/ft^3 at a 5% concentration

$= \dfrac{(5\ \text{lb/ft}^3)\ (44.5\ \text{ft}^3)}{(0.05)\ (8.34\ \text{lb/gal})}$

= 535 gal/regeneration

Regeneration time $= \dfrac{535\ \text{gal}}{(1.5\ \text{gpm/ft}^3)\ (44.5\ \text{ft}^3)}$

= 8.0 min.

5. Size of treatment process units

a Anion exchanger

Volume of resin/unit = 44.5 ft^3

Allowing 50% for bed expansion total volume for

each exchanger = 66.75 ft^3.

Use 2 units 3 ft. in diameter and 10 ft. high

b Regenerant storage

A mixing tank of 535 gal capacity for each regeneration period.

c Rinse water

Use a storage that will provide one day's requirement.

Rinse water/unit = (50 gal/ft^3) (44.5 ft^3)

= 2225 gallons

There will be six rinsings per day

$\therefore$ Minimum capacity of rinse water storage tank required

= 2225 x 6

= $\underline{\underline{13,350}}$ gals

Chapter X.

ADSORPTION

Procedure for the Design of a Continuous Carbon Filter

A. Data to be Obtained

1. Volume and characteristics (defined as COD, TOC, or specific constituents) of wastewater; variation in waste volume.

2. Pretreatment-removal by filtration of suspended solids.

B. Design Procedure

(Application of Bohart-Adams Relationship to the removal of synthetic detergent by adsorption in continuous carbon columns).

1. Obtain data for the removal of organics by passing a known concentration of the organic material through a known mass of activated carbon. At least three different flow rates covering the range desired must be employed.

2. The design relationship is given by

$$t = \frac{N_o}{C_o V} \left[X - \frac{V}{KN_o} \ln\left(\frac{C_o}{C_B} - 1 \right) \right]$$

The adsorptive capacity N_o and the rate constant K are calculated for each one of the flow rates as shown in Figure 33(a).

3. Determine the critical depth X_o (theoretical depth which is sufficient to prevent the effluent solute concentration to exceed C_B at zero time) using the relationship

$$X_o = \frac{V}{KN_o} \ln \left(\frac{C_o}{C_B} - 1 \right)$$

4. Develop the relationships to show the effect of flow rate on the value of K, N_o and X_o (Figure 33 (b)).

5. Select a flow rate and size the filter.

6. Calculate the annual carbon requirement and adsorption efficiency.

Application of Bohart-Adams Relationship to the Removal of Synthetic Detergent by Adsorption in Continuous Carbon Columns.

Column Capacity Data for the Adsorption of ABS from a 10 mg/l Aqueous Solution [1]

Flow Rate gpm/ft^2	Bed Depth ft.	Throughput Volume gallons	Time hrs.	ABS Adsorbed g/100gC	Total ABS Adsorbed^2 g/100gC	Carbon Dosage g/1000 gal.
				Capacity to 0.5 ppm Breakpoint		
2.5	2.5	363	440	8.9	24.0	0.97
	5.0	1216	1480	14.3	20.5	0.62
	7.5	2148	2620	15.9	15.9	0.53
5.0	2.5	141	87	3.1	19.7	2.66
	5.0	730	440	8.1	17.7	1.03
	10.0	2190	1340	12.7	12.7	0.69
10.0	5.0	332	102	3.7	14.3	2.22
	10.0	1380	420	7.7	12.8	1.09
	15.0	2760	835	10.4	10.4	0.82

1 - 1 inch diameter columns were used

2 - to 0.5 ppm breakpoint from third column section

Determination of the Freundlich Isotherm for this carbon yields the following results:

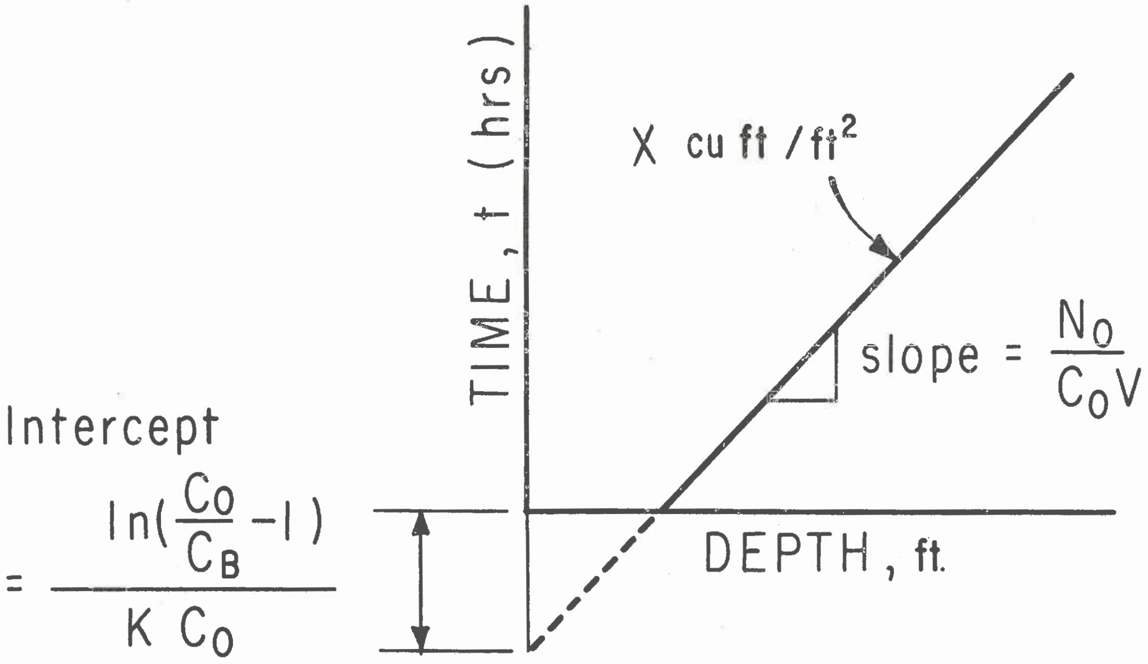


FIG. (a)

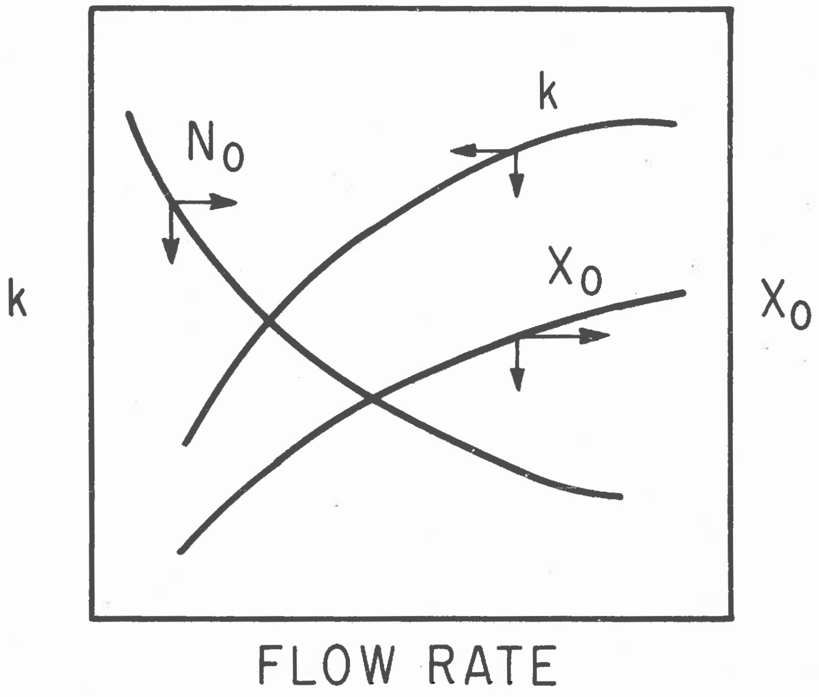


FIG. (b)

Figure 33

At ABS 11.6 mg/l = C_o

x mg ABS/mg C	C, mg/500 ml
.15	.2
.18	1.3
.20	2.8

The coefficients in the Bohart-Adams equation for this carbon may be evaluated graphically (Fig. 33, 33.1). A plot of t vs. X results in a straight line as shown in Fig. 33.2.

The adsoprtive capacity N_o can be calculated from the slope of the line

$$N_o = (\text{slope})\ C_o\ V$$

For these data

$$C_o = \left(\frac{10\ \text{mg/l}}{10^6}\right)\left(\frac{62.4}{\text{ft}^3}\right) = 6.24 \times 10^{-4}\ \text{lb/ft}^3$$

The rate constant K can be calculated from the intercept

$$K = \frac{\ln\left(C_o/C_B - 1\right)}{C_o\ (\text{intercept})}$$

For these data

$$\ln\left(\frac{10}{0.5} - 1\right) = 2.95$$

$$C_o = 6.24 \times 10^{-4}\ \text{lb/ft}^3$$

The critical bed depth can be calculated from the expression

$$X_o = \frac{V}{KN_o} \ln\left(\frac{C_o}{C_B} - 1\right)$$

The values for these coefficients are summarized in the following table:

Flow Rate gpm/ft^2	Slope	N_o lb/ft^3	Intercept b	K $ft^3/lb\text{-}hr$	X_o ft
2.5	435	5.4	-630	7.5	1.46
5.0	170	4.2	-370	12.7	2.18
10.0	73	3.6	-250	18.8	3.48

This carbon will be used to treat water which contains 10 mg/l ABS to a residual of 0.5 mg/l at a rate of 100,000 gallons per week. The carbon bed is two feet in diameter and five feet deep. (Area = 3.14 ft^2).

a. Calculate the annual carbon requirement.

b. Calculate the adsorption efficiency.

Solution

a. Assume a 5-day work week; therefore the flow rate is

$$\frac{100,000}{5(1440)} = 13.9 \text{ gpm or } 4.4 \text{ gpm/ft}^2$$

From the chart at this flow rate

$$K = 11.7 \text{ ft}^3/\text{lb-hr}$$

$$N_o = 4.5 \text{ lb/ft}^3$$

$$V = \frac{4.4 \text{ gpm}}{\text{ft}^2}\left(\frac{60 \text{ min}}{\text{hr}}\right)\left(\frac{\text{ft}^3}{7.48 \text{ gal}}\right) = 35.4 \text{ ft/hr}$$

$$t = \frac{4.5}{0.000624\ (35.4)}\left[5 - \frac{35.4}{11.7(4.5)}\ (2.95)\right]$$

$$= 615 \text{ hrs.}$$

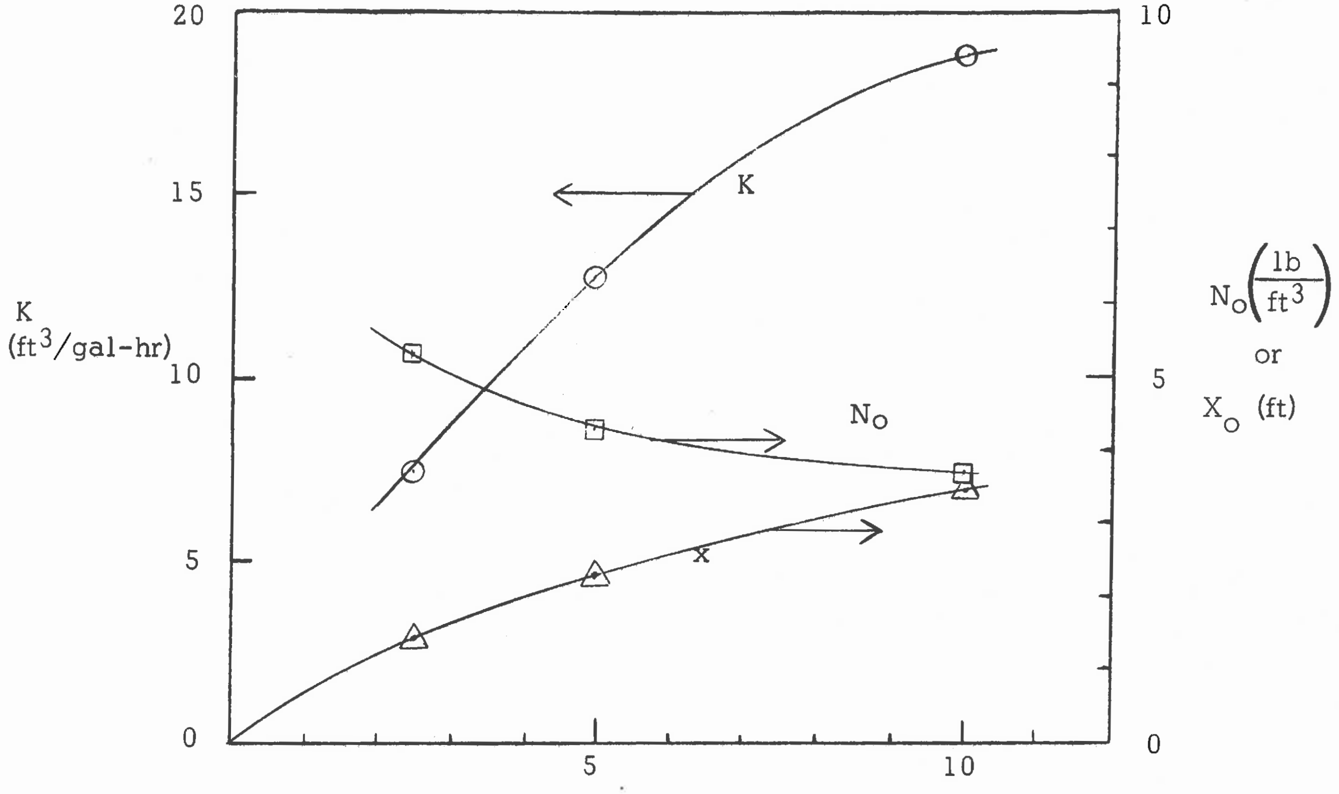


Figure 33.1. Flow Rate (gpm/ft^2)

The effect of flow on the value of K, N_o and X_o (Bohart-Adams relationship)

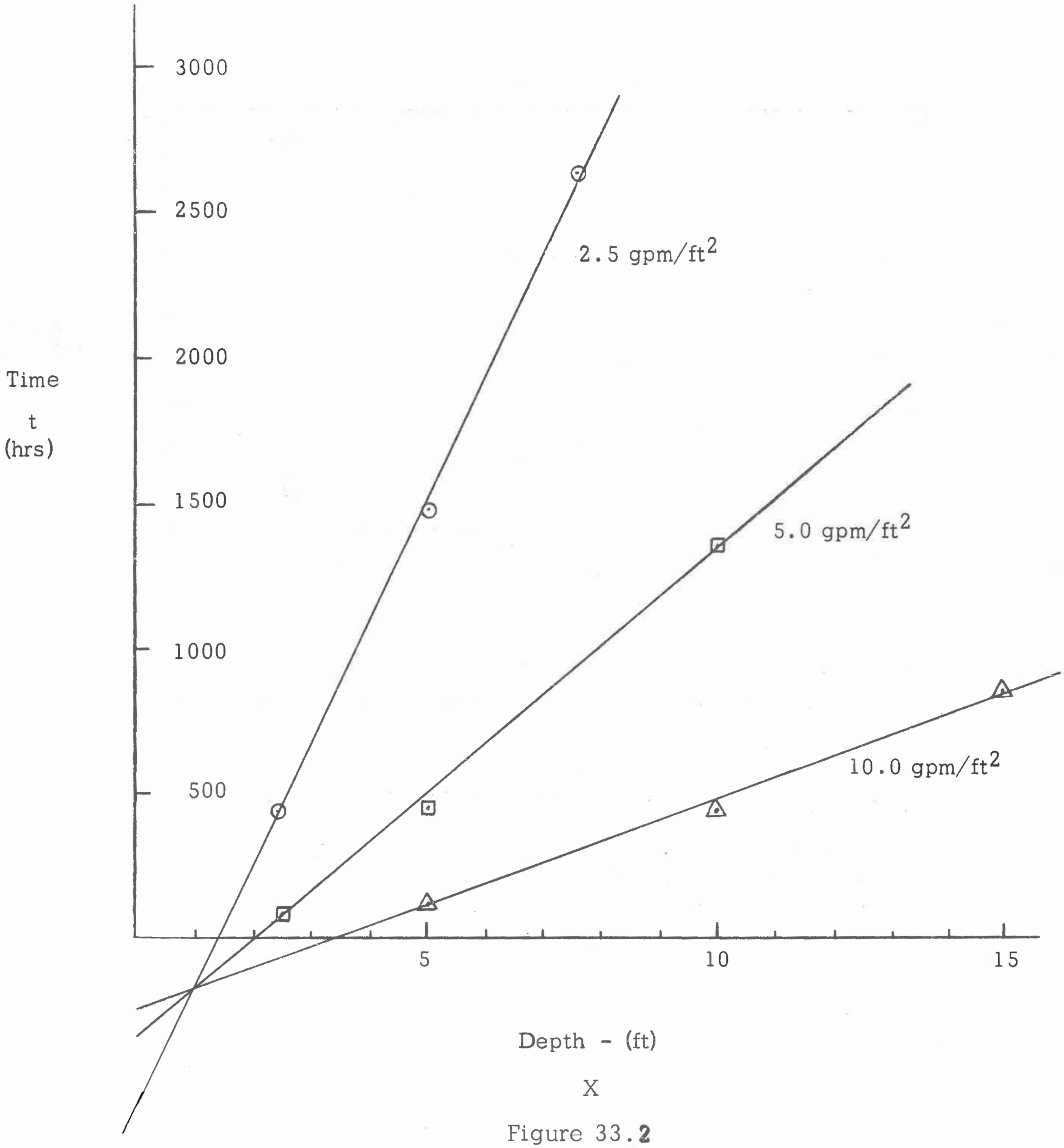

3000
2500
2000
1500
1000
500
Time
t
(hrs)
2.5 gpm/ft^2
5.0 gpm/ft^2
10.0 gpm/ft^2
5
10
15
Depth - (ft)
X

Figure 33.2

Total Volume = 13.9 (60) (615) = 514,000 gal

Annual Volume = 5,200,000 gal

Number of carbon changes = $\frac{5,200,000}{514,000}$ = 10.2

Volume of carbon per change = 3.15 x 5 = 15.8 ft^3

Annual carbon volume = 15.8 (10.2) = 160 ft^3

From isotherm

$x = \frac{21 \text{ lb ABS}}{100 \text{ lb C}}$

Total ABS removed = 9.5 (5.2) (8.34) = 410 lb

$\frac{410 \text{ lb}}{21 \text{ lb}/100 \text{ lb C}}$ = 1950 lb/yr carbon required

b. Efficiency of carbon beds

Total ABS adsorbed = 410 lb

Total capacity = $(4.5 \text{ lb/ft}^3)(160 \text{ ft}^3) = 720$

$$\text{Efficiency} = \frac{410\ (100)}{720} = 57\%$$

Computed from X_o

X_o @ 4.4 gpm/ft^2 = 2.1

$$\text{Efficiency} = \frac{X - X_o}{X}\ (100) = \frac{5 - 2.1}{5}\ (100) = 58\%$$

Chapter XI.

THICKENING

Procedure for the Design of a Gravity Thickening Unit

A. Data to be Obtained

1. Solids to be thickened; average daily lbs/day; variation in quantity; solids concentration

2. Variation in solids characteristics

B. Design Procedure

1. Develop a series of three batch settling tests for the range of influent solids concentrations which might be expected in full-scale operation as indicated in 2 above and as shown in Step 1 - Figure 34.

2. The unit area for each interfacial settling velocity is calculated from the relationship

$$UA = \frac{1/C_i - 1/C_u}{U_i}$$

and as shown in Step 2 - Figure 34.

U_i is the settling velocity as determined from the tangent to the curve and C_i the interfacial concentration at the point of tangency. H_i is the height the sludge would occupy if it all were at the interfacial concentration C_i.

3. The maximum unit area for each feed concentration and selected underflow concentrations is determined for several interfacial settling velocities over the compression portion of the settling curve as shown in Step 3 - Figure 34. It is necessary to select the desired underflow concentrations. The ultimate compaction of the

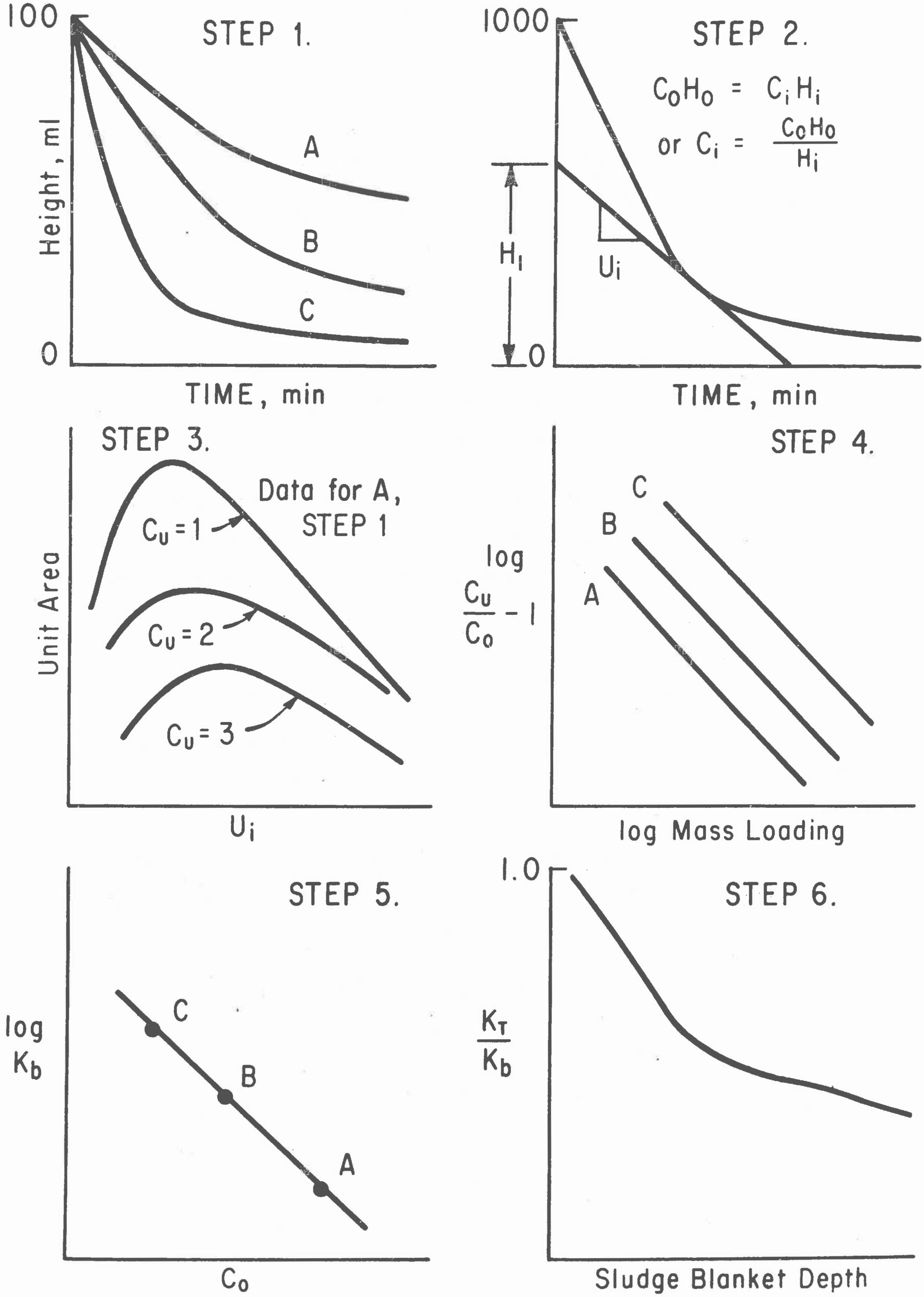

STEP 1.
100
0
Height, ml
TIME, min
A
B
C
STEP 2.
1000
0
$C_oH_o = C_iH_i$
or $C_i = \frac{C_oH_o}{H_i}$
H_i
U_i
TIME, min
STEP 3.
Unit Area
U_i
Data for A, STEP 1
$C_u = 1$
$C_u = 2$
$C_u = 3$
STEP 4.
log $\frac{C_u}{C_o} - 1$
log Mass Loading
A
B
C
STEP 5.
log K_b
C_o
A
B
C
STEP 6.
1.0
$\frac{K_T}{K_b}$
Sludge Blanket Depth

Figure 34

sludge can reasonably be estimated by extrapolation of the settling curves in Step 1 (or by a plot of log $[H - H_\infty]$ vs time in which H_∞ is the height occupied by the sludge at infinite time). The underflow concentration C_u is usually selected as $0.7 - 0.9\, C_\infty$. (It should be noted that $C_o H_o = C_\infty H_\infty$).

4. Using the maximum unit area values obtained in Step 3 for different feed and underflow concentrations a plot is made of $\log\left(\frac{C_u}{C_o} - 1\right)$ vs the log of the mass loading (1/UA) as shown in Step 4 - Figure 34. The coefficients K_b and n are obtained from this plot; n is the slope of the curves and is characteristic of the type of sludge while K_b is the intercept of the plot and is related to the initial concentration of the suspension and the height of the test cylinder.

5. The intercept, K_b, is related to C_o as shown in Step 5 - Figure 34, by plotting log K_b vs C_o. From this plot the appropriate coefficient K_b is selected corresponding to the feed concentration C_o.

6. The scale-up factor to a continuous thickener is selected from Step 6 - Figure 34 which related K_T/K_b to the sludge blanket depth in the continuous thickener.

7. Using the appropriate value for K_T, the allowable mass loading for selected values of feed concentration, and underflow concentration, the mass loading can be computed. The cross-sectional area of thickener can then be computed.

Example--Gravity Thickening

An industrial sludge gave the following thickening properties in 1-liter cylinder studies.

C_o = 1310 mg/l		C_o = 2640 mg/l		C_o = 5300 mg/l	
Time min.	Ht. ml	Time, min.	Ht. ml	Time, min.	Ht. ml
0	1000	0	1000	0	1000
1	970	1	980	3	990
2	820	2	920	6	955
3	700	3	870	9	925
4	550	4	800	12	820
5	400	5	735	16	740
6	290	6	680	20	670
7	245	7	620	24	600
8	220	8	570	30	530
9	200	9	525	34	500
10	190	10	480	43	455
13	170	13	400	50	430
17	150	17	330	57	410
27	120	27	255	73	370
34	110	34	240		
46	90	46	210		

Design a thickner to produce an underflow concentration of 8000 mg/l from an initial concentration of 3200 mg/l and a flow of 1.0 MGD. Use a blanket depth of 6 ft. in the continuous thickener.

C_o = 1310 mg/l

		U_i	Unit Area ft^2/lb/day		
H_i, ml	C_i, gm/l	ft/hr	C_u=5000mg/l	C_u=7000mg/l	C_u=8000mg/l
1000	1.31	8.62	0.044	.0482	.0497
565	2.32	3.26	.047	.059	.0626
360	3.64	1.24	.0405	.0715	.08
230	5.70	0.33	---	.0645	.10
200	6.55	0.212	---	---	.0885

C_o = 2640 mg/l

H_i, ml	C_i, gm/l	U_i ft/hr	Unit Area ft^2/lb/day		
			C_u=8000mg/l	C_u=10,000mg/l	C_u=12,000mg/l
1000	2.64	4.06	0.0292	.0335	.036
770	3.43	1.93	0.058	.073	.081
660	4.0	1.38	0.0606	.0885	.11
455	5.8	0.545	0.058	.096	.13
390	6.77	0.336	0.046	.0665	.0724

C_o = 5300 mg/l

H_i, ml	C_i, gm/l	U_i ft/hr	Unit Area ft^2/lb/day		
			C_u=8000mg/l	C_u=10,000mg/l	C_u=12,000mg/l
1000	5.30	1.15	---	.0506	.0605
960	5.58	1.09	.0338	.0485	.059
950	5.65	1.01	.0346	.0515	.063
800	6.70	0.65	.0233	.0475	.064
500	10.6	0.384	---	---	.02

C_o, mg/l	C_u, mg/l	$\frac{C_u}{C_o}$	$\frac{C_u}{C_o} - 1$	UA (max) ft^2/lb/day	Mass Leading lb/ft^2/day
1310	5000	3.82	2.82	.047	21.3
	7000	5.35	4.35	.0715	14.0
	8000	6.10	5.10	.10	10.0
2640	8000	3.04	2.04	.0606	16.5
	10000	3.80	2.80	.096	10.4
	12000	4.55	3.55	.13	8.7
5300	8000	1.51	0.51	.0346	28.9
	10000	1.89	0.89	.0515	19.4
	12000	2.26	1.26	.064	15.6

For an initial concentration of 3.2 gm/l, K_b = 16.5 (Plot 5, Fig. 34 and Fig. 39).

Applying a correction factor of 0.7 for scaling up to the field; $K_{tank} = (16.5)(0.7) = 11.55$.

From plot 4, Fig. 34, n = 0.72 (See Fig. 38)

Design is for C_u = 8000 mg/l and C_o = 3200 mg/l.

$$\frac{C_u}{C_o} - 1 = \frac{K}{(ML)^n}$$

$$(ML)^n = \frac{K}{\frac{C_u}{C_o} - 1} = \frac{11.55}{1.5} = 7.7$$

$$n \log ML = 0.886$$

$$\log ML = \frac{0.886}{0.72} = 1.23$$

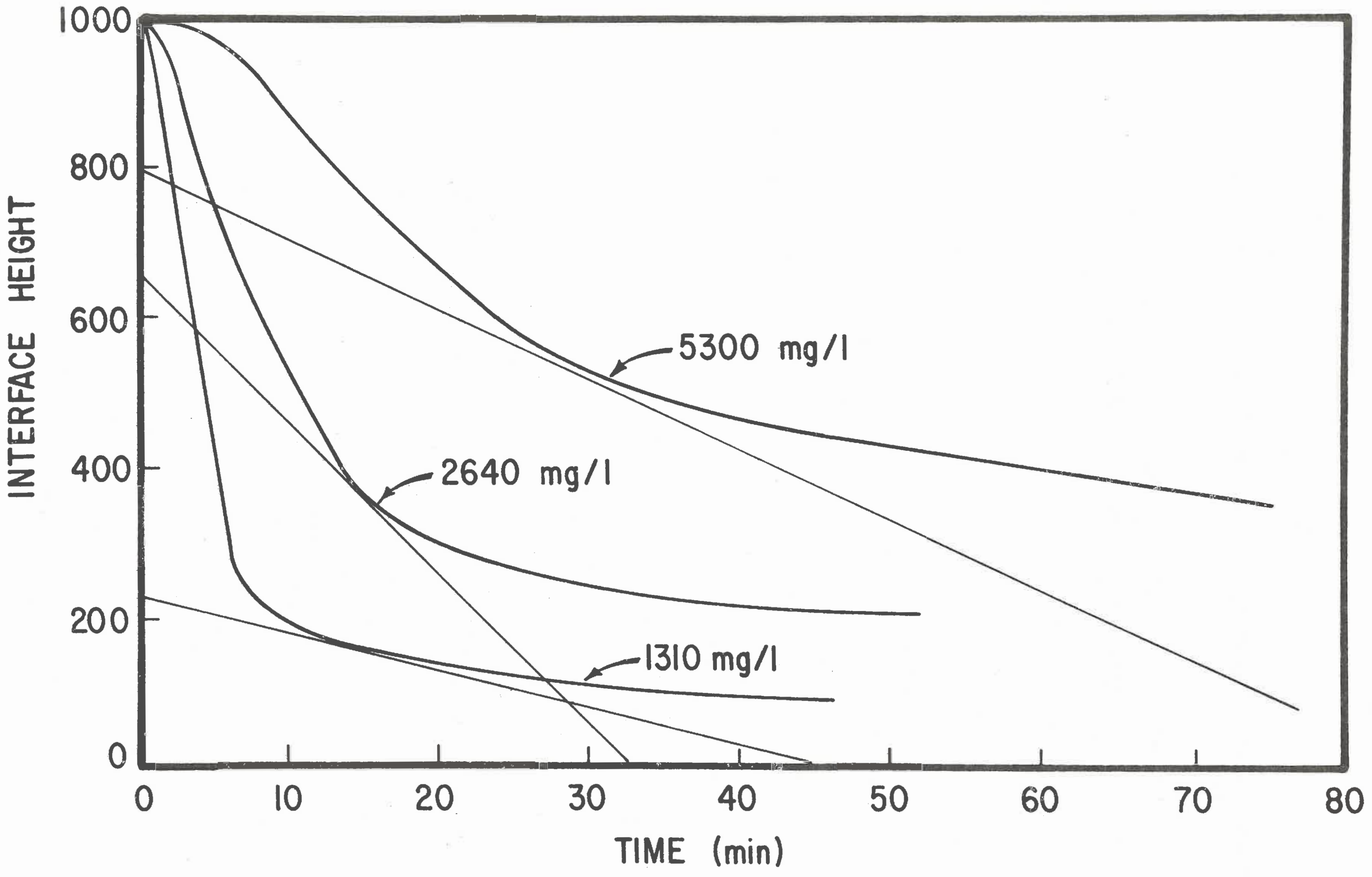
$$\therefore ML = 17 \text{ lbs/ft}^2\text{/day}$$

Total sludge for 1 MGD flow at 3200 mg/l = 3200 x 8.34

= 26,800 lbs

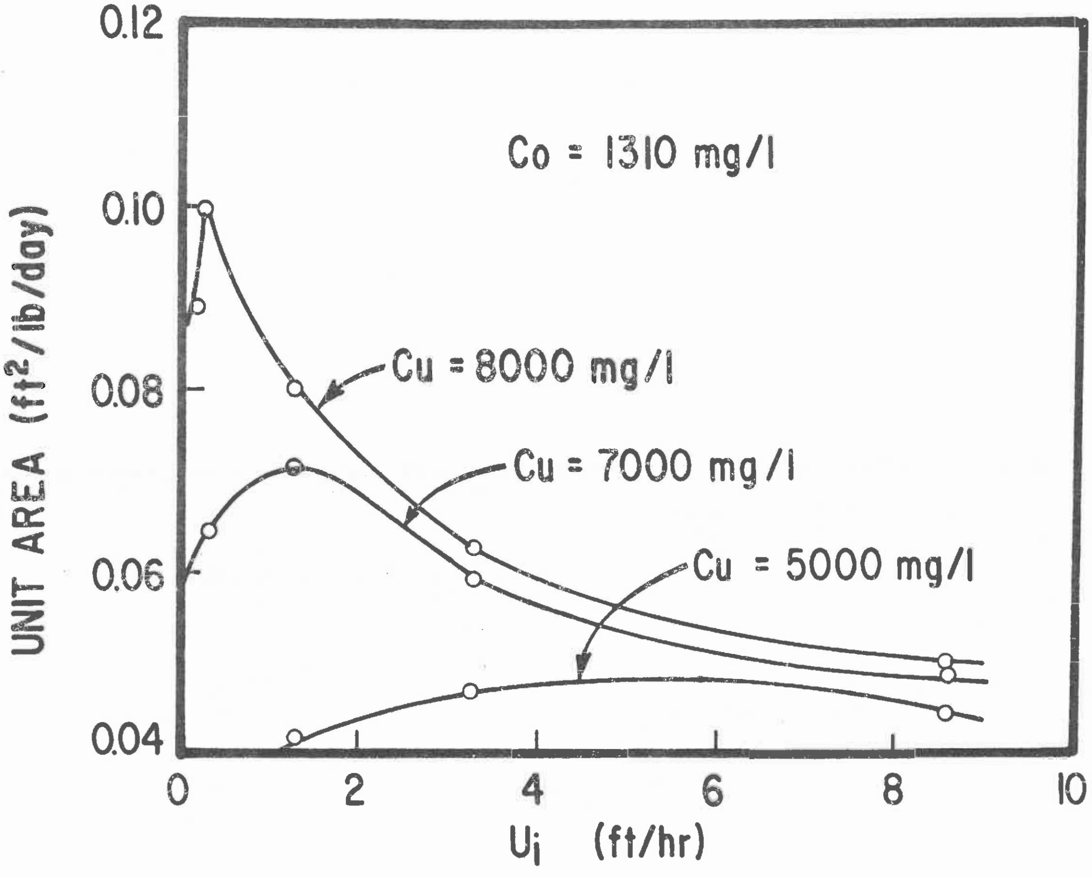
Area required for thickener $= \frac{26,800}{17} = 1580 \text{ ft}^2$

Diameter of the thickener $= \sqrt{\frac{1580}{0.785}} = \sqrt{2020} = 45$ ft.



1000
800
600
400
200
0
INTERFACE HEIGHT
0
10
20
30
40
50
60
70
80
TIME (min)
5300 mg/l
2640 mg/l
1310 mg/l

FIG. 35


Co = 1310 mg/l
Cu = 8000 mg/l
Cu = 7000 mg/l
Cu = 5000 mg/l
UNIT AREA (ft^2/lb/day)
0.12
0.10
0.08
0.06
0.04
0
2
4
6
8
10
U_i (ft/hr)

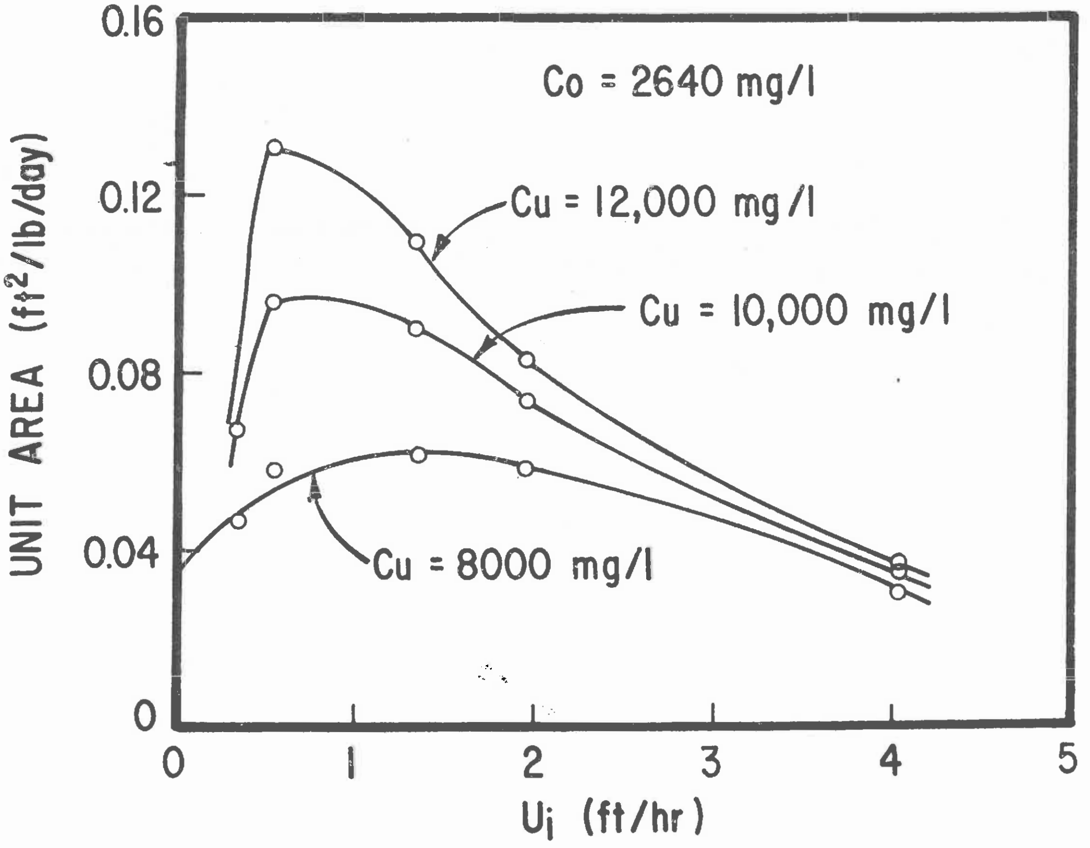

Co = 2640 mg/l
Cu = 12,000 mg/l
Cu = 10,000 mg/l
Cu = 8000 mg/l
UNIT AREA (ft^2/lb/day)
0.16
0.12
0.08
0.04
0
0
1
2
3
4
5
U_i (ft/hr)

FIG. 36

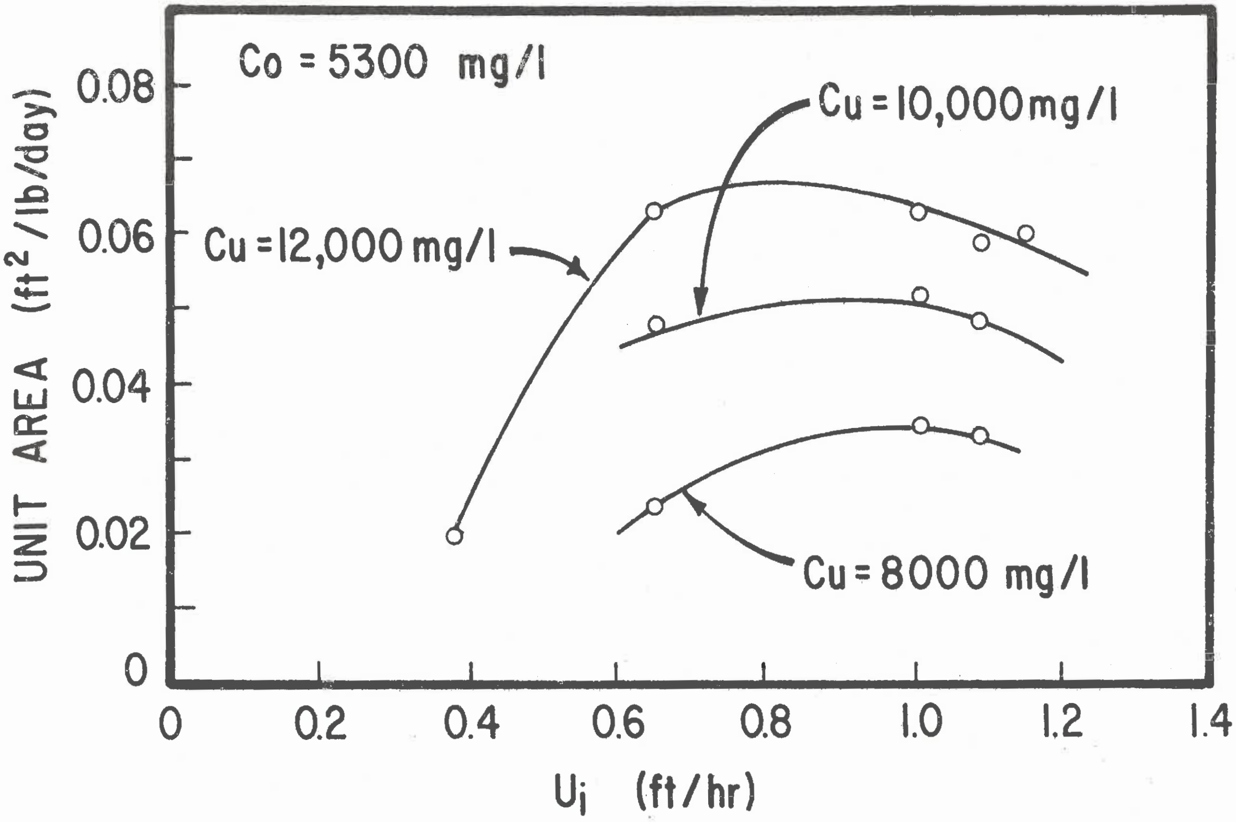
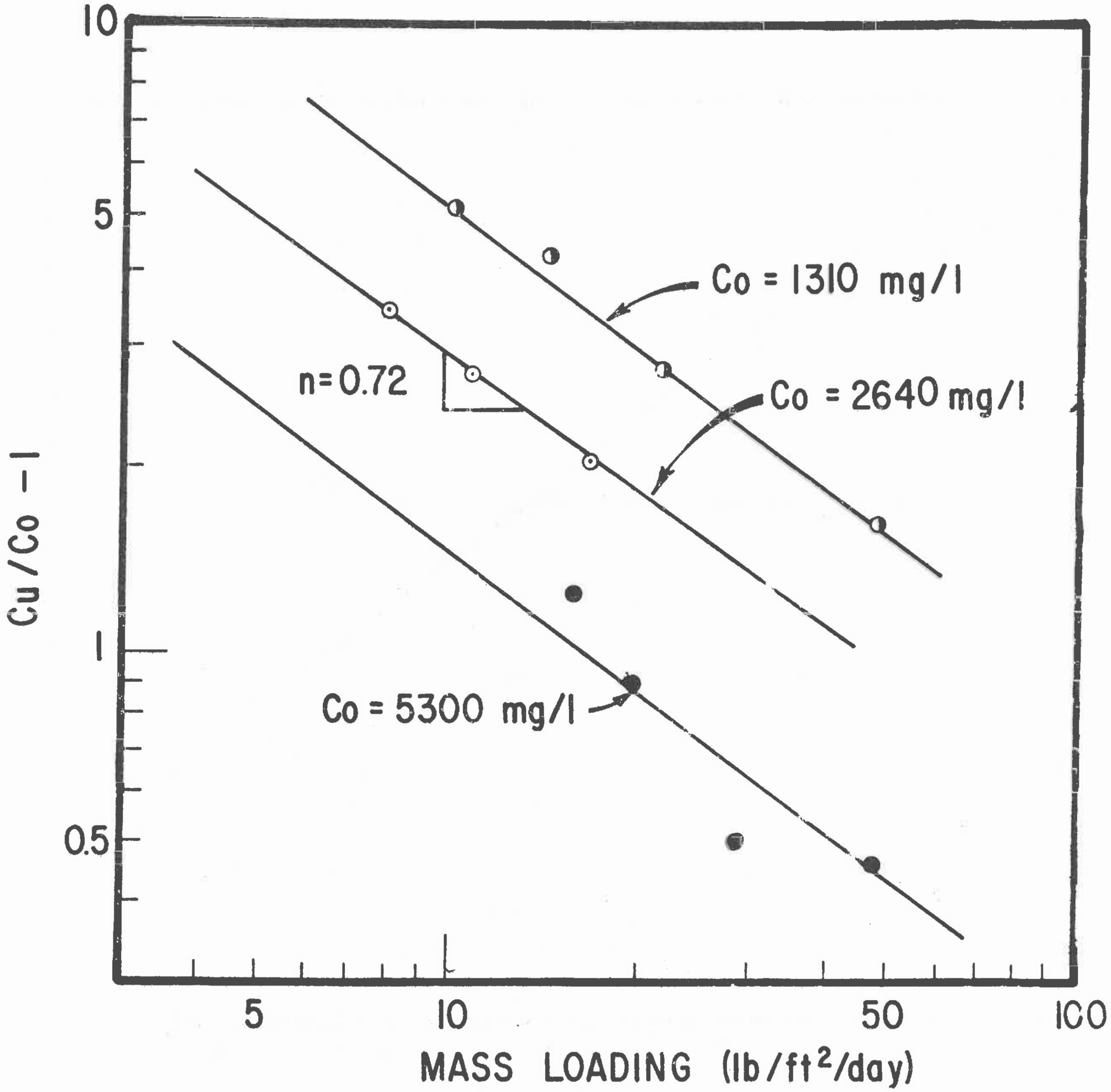

Co = 5300 mg/l
Cu = 10,000 mg/l
Cu = 12,000 mg/l
Cu = 8000 mg/l
UNIT AREA (ft^2/lb/day)
0.08
0.06
0.04
0.02
0
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
U_i (ft/hr)

FIG. 37



10
5
1
0.5
Cu/Co − 1
5
10
50
100
MASS LOADING (lb/ft²/day)
Co = 1310 mg/l
Co = 2640 mg/l
Co = 5300 mg/l
n = 0.72

FIG. 38

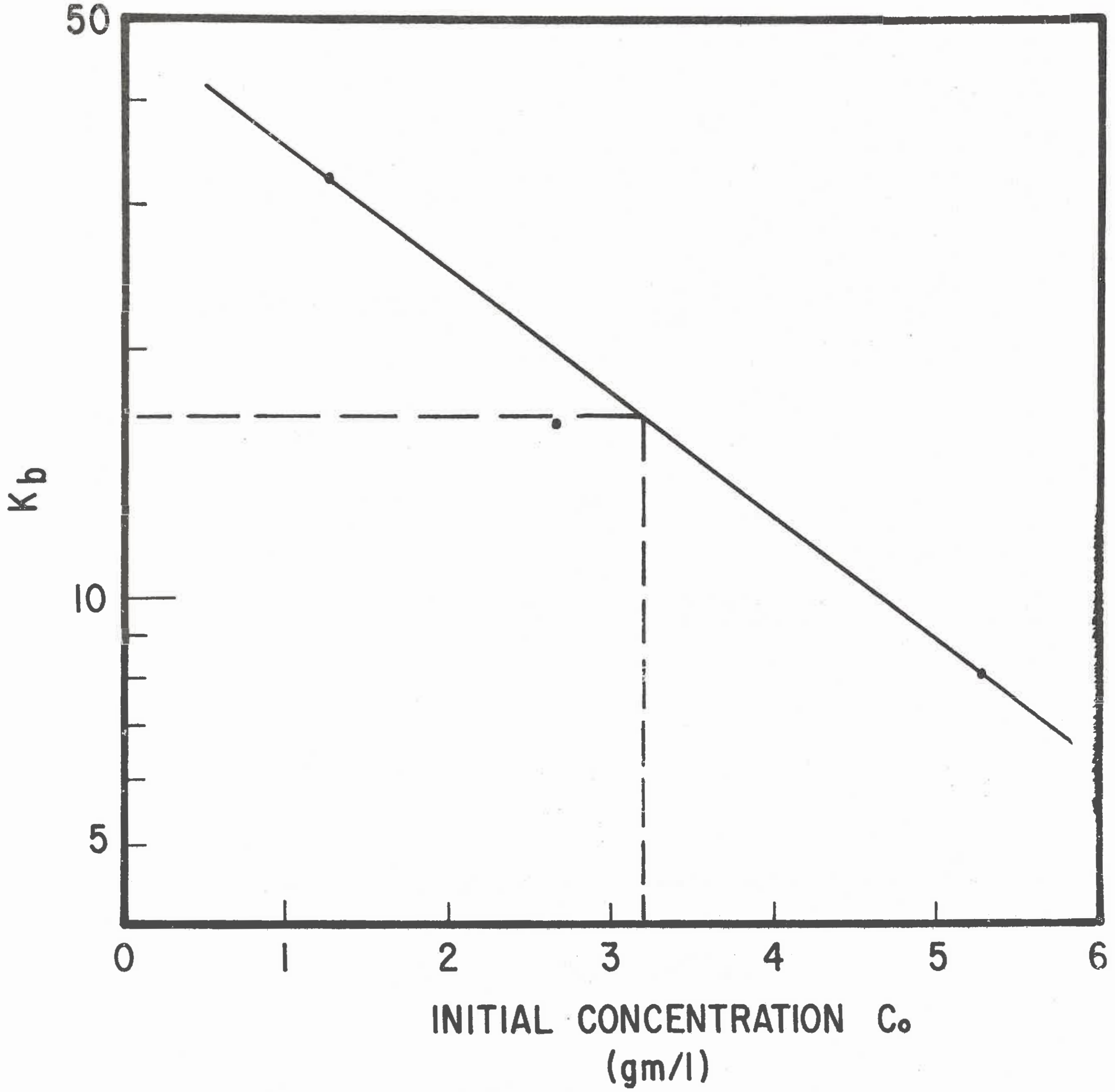


FIG. 39

Chapter XII.

VACUUM FILTRATION

Procedure for the Design of a Vacuum Filtration

A. Data to be Obtained

1. Volume and characteristics of sludge; variation in sludge quantity.
2. Pretreatment (i.e., thickening) and resulting concentrations of sludge to be dewatered.

B. Design Procedure

1. The optimum coagulant dosage is determined from a series of Büchner funnel tests and plotted as shown in Figure 40 (a). It is important to note that the coagulant dosage must be expressed as a % by weight if the solids content of the feed to the filter is variable.
2. The design relationship is:

$$L_f = 35.7 \left[\frac{P^{(1-s)}}{\mu R_o} \right]^{1/2} \cdot \frac{C^m}{t_f^{\,n}} \qquad (1)$$

The specific resistance, R_o, is obtained from a correlation of the leaf test data. The exponents n and m are obtained from the slopes of the plots 40 (b) and 40 (d). The compressibility coefficient, s, is obtained from the slope of plot 40 (c).

3. In general, the maximum initial solids concentration, C, is used which can be dependably expected from prethickening unit.

EXPERIMENTAL PLOTS FOR VACUUM FILTRATION DESIGN

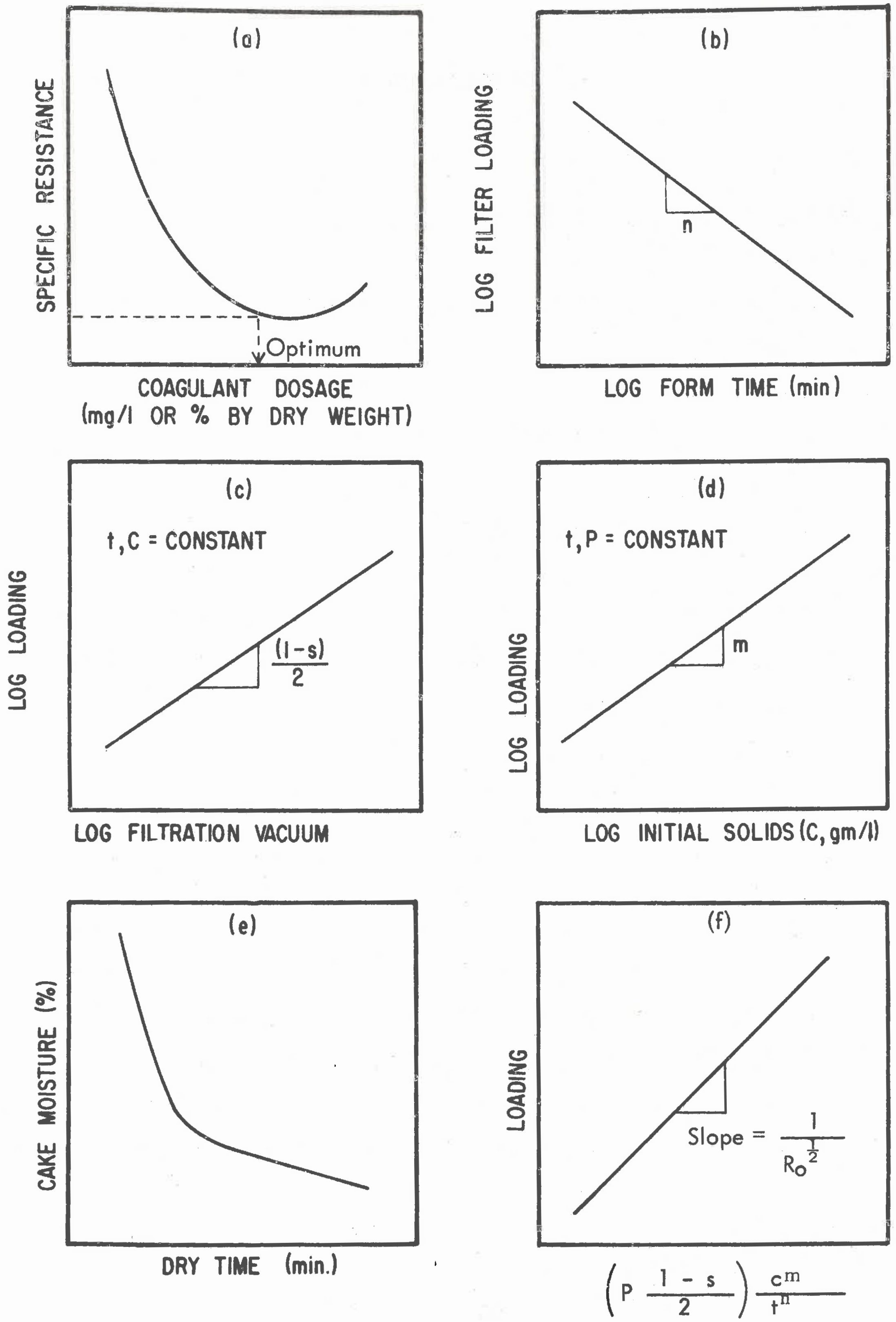


Figure 40

4. Chemical conditioning feed equipment is sized for the optimum dosage (on a % of the dry weight solids fed to the filter).

5. It is next necessary to select a cycle time, t_c. Total cycle time on a filter may vary from 1 to 6 minutes. Submergence of the drum may vary from 10 to 60% resulting in a maximum spread of form time 0.1 to 3.5 minutes. This also yields a maximum spread in dry time of 2.5 to 4.5 minutes.

(a) From plot 40 (e) a dry time is selected to give a maximum cake solids. A porus cake will crack quickly, yielding a maximum cake solids in a short dry time; while other cakes will require a long dry time to achieve a maximum cake solids.

(b) A form time is next selected to give a maximum cake pickup within the cycle time limitations established above.

6. The operating form vacuum is then selected. In general the filter yield from a highly compressible cake will be relatively unaffected by increases in form vacuum. Filters will operate with form vacuum varying from 12 to 17 inches Hg for most economical operation, the more compressible cakes will operate at the lower form vacuum. Dry vacuum may be expected to vary from 12 inches to 17 inches Hg.

7. Having calculated or selected the appropriate coefficients, the filter loading based on form time is computed from Equation (1).

8. The form loading is converted to cycle time from the relationship:

$$L_c = L_f \cdot \frac{\%\ \text{submergence}}{100} \cdot 0.8$$

The factor 0.8 compensates for that area of the filter drum where the cake is removed and the media washed.

9. The filter drum is sized:

$$\text{Drum area required} = \frac{\text{lbs solids/hr fed (dry weight)}}{L(\text{lbs/ft}^2/\text{hr})}$$

It should be noted that if the filter is to be operated less than 24 hours a day, the solids to be fed to the filter must be adjusted for the operating schedule.

Problem -- Vacuum Filtration

The following data was obtained from leaf tests conducted on a pulp and paper mill primary sludge. Design a vacuum filter to dewater (25,000 lbs) per day of sludge (dry weight) at 3% solids. Filter operation will be 98 hours per week.

% Solids in Feed	Vacuum in Hg	Cycle form	time, min. dry	Cake % Solids	Thick-ness, inch.	Loading lb/ft^{2}hr	$t_{d/w}$
2.18	20	1.5	2.0	20.0	5/8	21.8	1.27
2.43	20	0.5	1.0	19.0	1/2	54.5	0.79
2.55	20	0.25	0.5	17.4	3/8	84.0	0.53
2.43	20	0.25	1.0	19.2	3/8	84.3	1.05
2.37	20	0.13	0.3	16.6	5/16	126.7	0.384
1.85	20	0.25	0.5	16.6	1/4	70.5	0.80
1.87	20	0.50	0.5	17.2	3/8	48.5	0.53
1.85	20	1.0	1.25	18.9	1/2	29.2	0.98
1.87	10	0.5	0.75	18.2	3/8	44.2	0.79
1.90	5	0.5	0.75	18.7	3/8	45.2	0.79
1.89	5	0.5	0.75	16.0	3/8	39.2	0.79
1.89	5	0.5	0.75	16.6	3/8	39.5	0.79
1.89	15	0.5	0.75	19.5	3/8	46.0	0.79
1.88	15	0.5	0.75	20.0	3/8	46.0	0.79
5.63	20	0.5	0.75	15.6	7/8	156.0	0.33

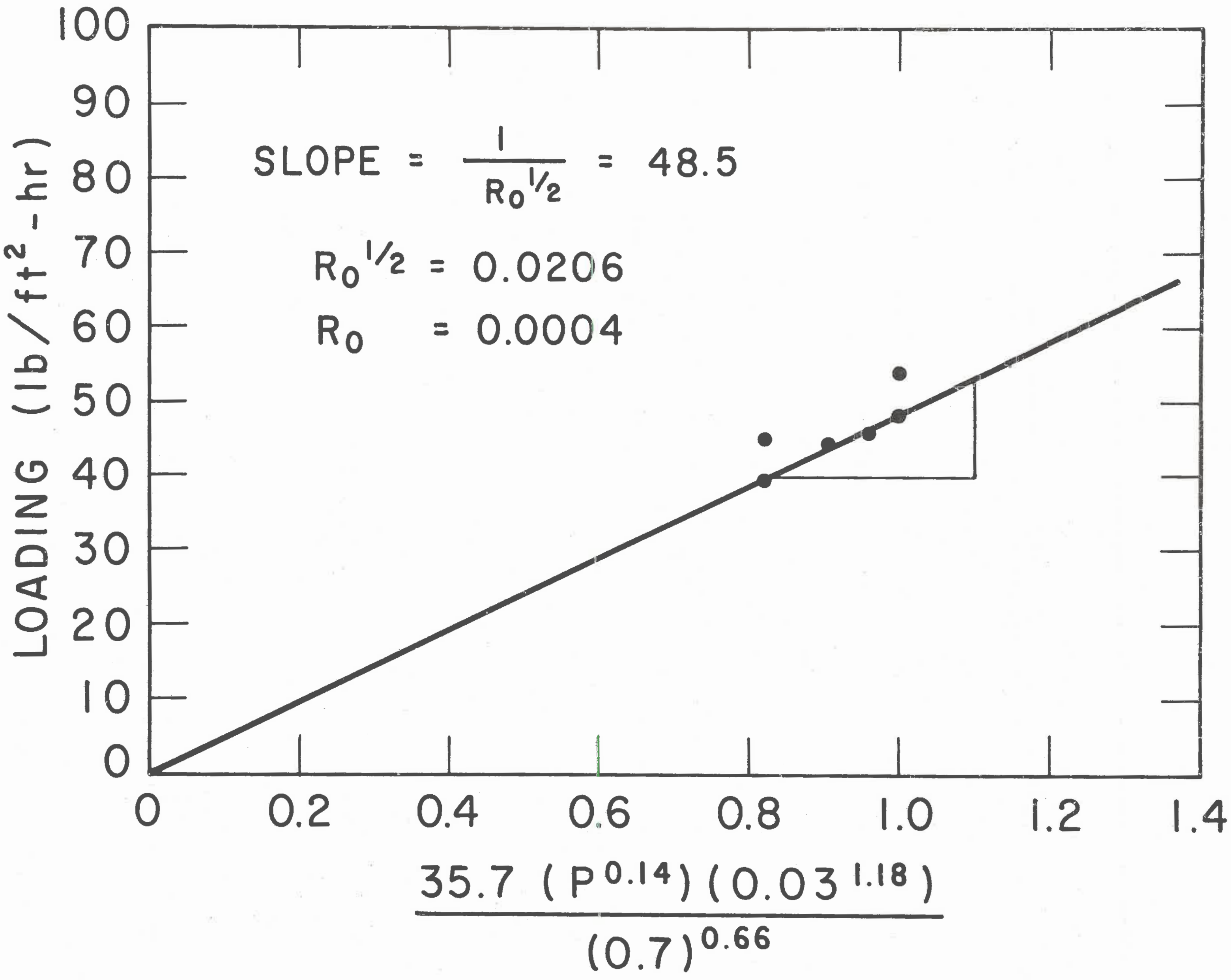
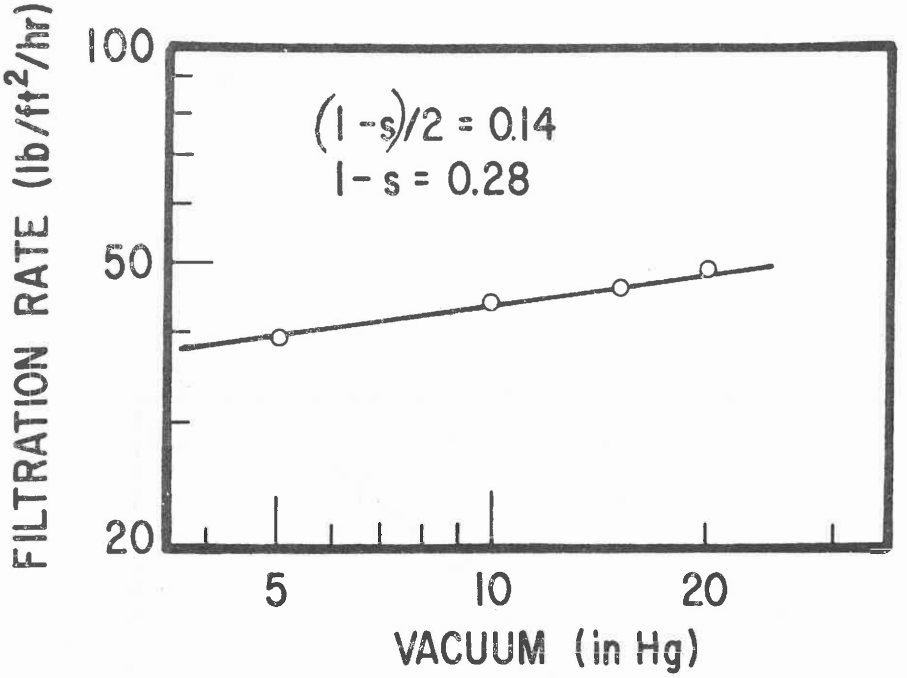

SLOPE = $\frac{1}{R_0^{1/2}}$ = 48.5
$R_0^{1/2}$ = 0.0206
R_0 = 0.0004
LOADING (lb/ft^2-hr)
100
90
80
70
60
50
40
30
20
10
0
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
$\frac{35.7\,(P^{0.14})(0.03^{1.18})}{(0.7)^{0.66}}$

FIGURE 41


FILTRATION RATE (lb/ft^2/hr)
100
50
20
$(1-s)/2 = 0.14$
$1-s = 0.28$
5
10
20
VACUUM (in Hg)

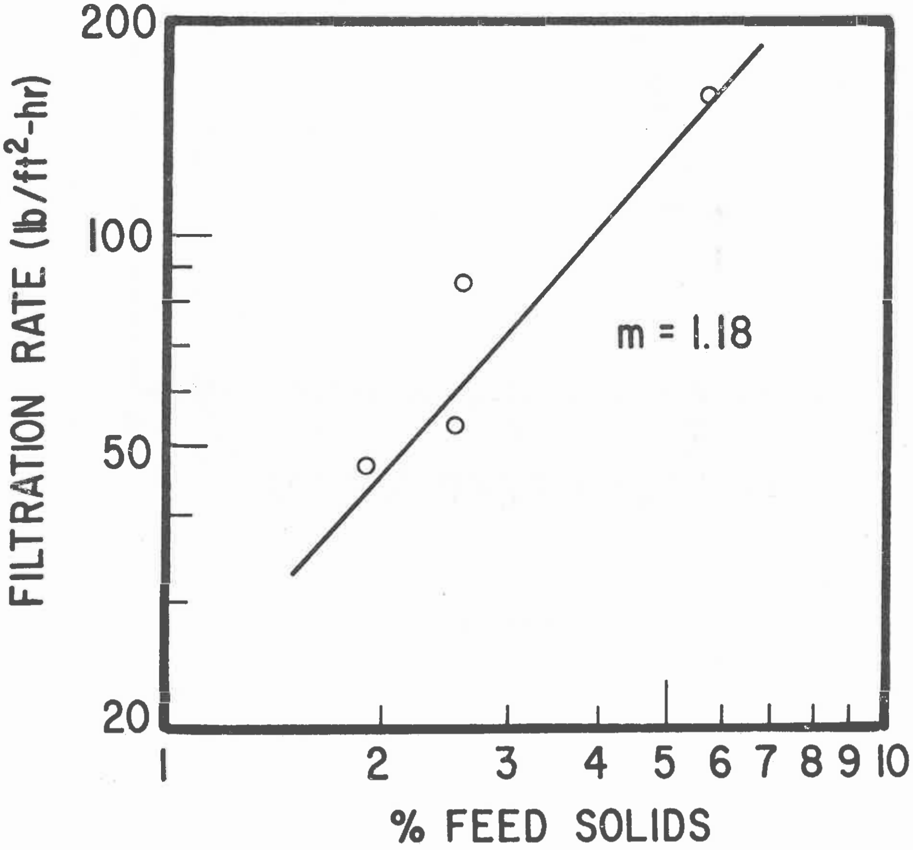

FILTRATION RATE (lb/ft^2-hr)
200
100
50
20
m = 1.18
1
2
3
4
5
6
7
8
9
10
% FEED SOLIDS

FIG. 42

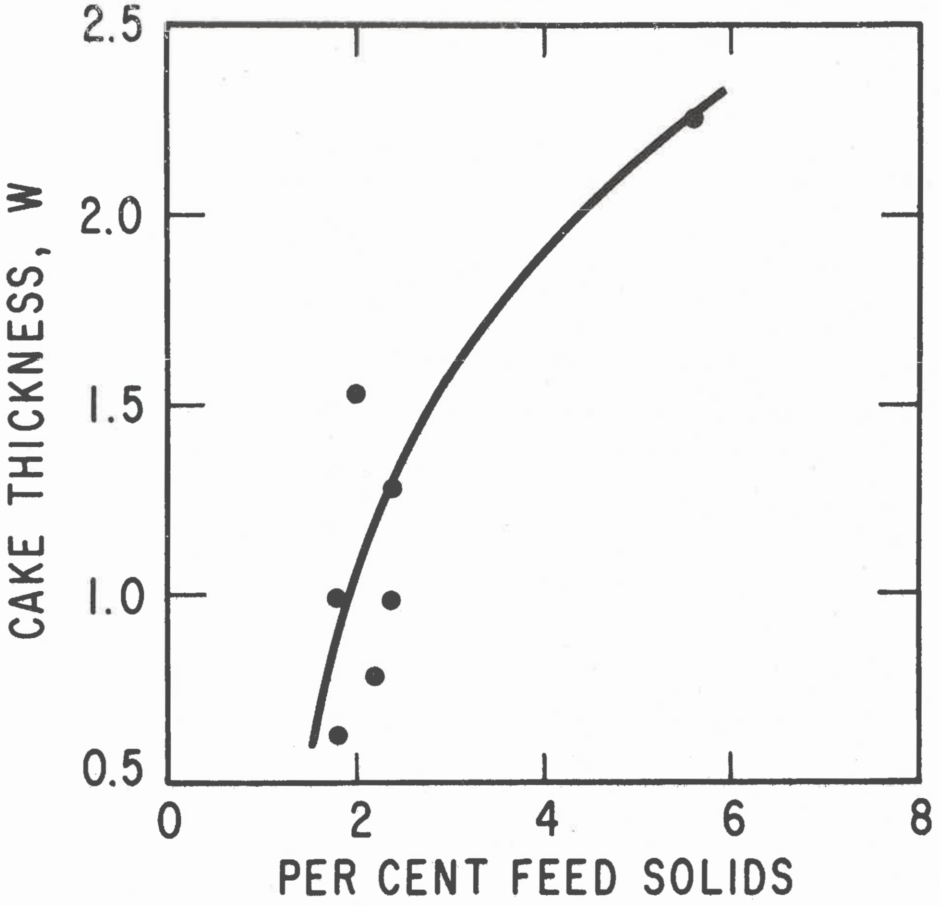
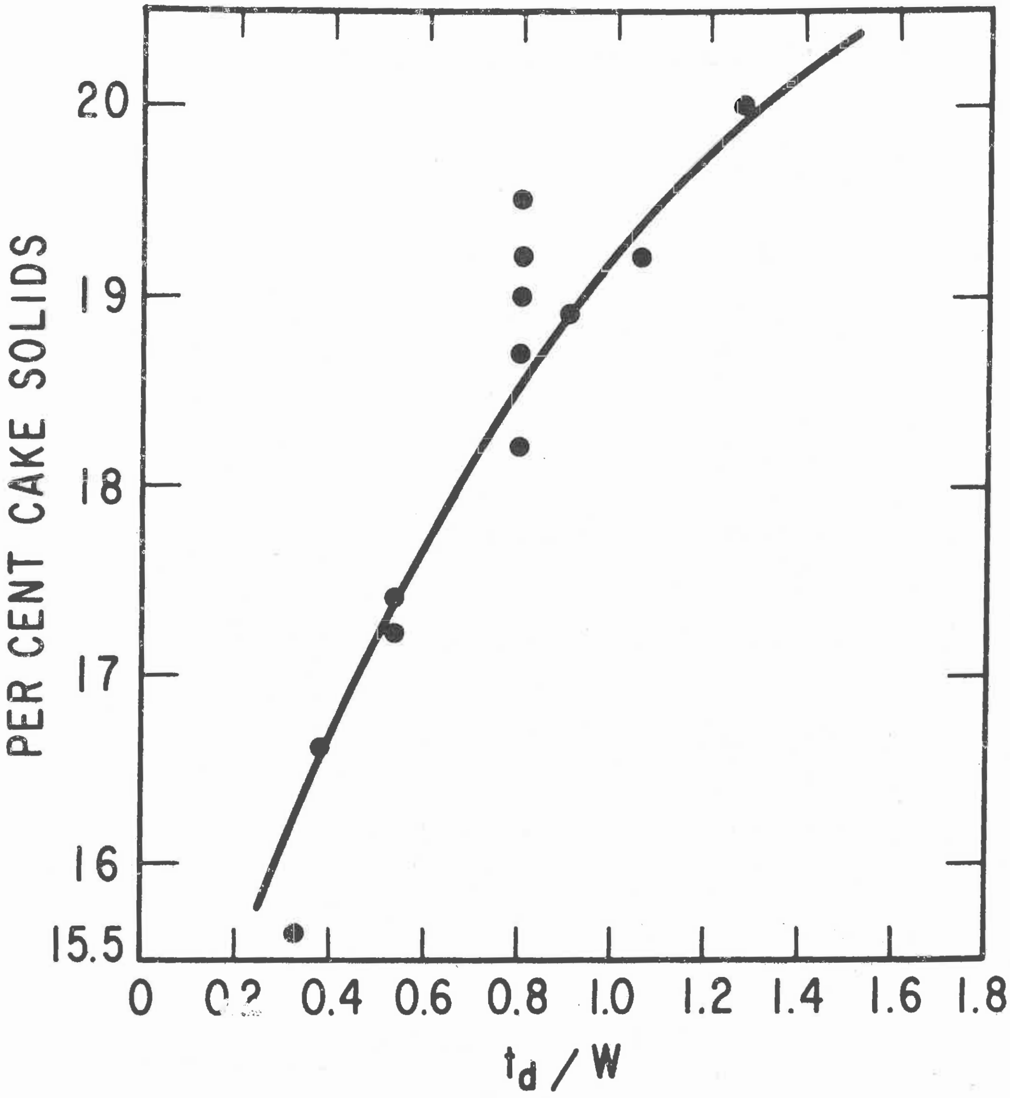


Figure 43



PER CENT CAKE SOLIDS
20
19
18
17
16
15.5
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
t_d / W

Figure 44

Design -- Vacuum Filtration

The operating equation for the form loading

$$L_f = 35.7\left(\frac{P^1 - S}{\mu R_o}\right)^{1/2} \frac{C^m}{t^n}$$

From the plots, $1 - s = 0.28$

$m = 1.18$

$n = 0.66$

$R_o = 0.0004$

Selecting an operating vacuum of 10 in Hg and a dry time of 1.5 miuntes and a submergence of 30% , t_f is calculated to be 0.7 min. The total cycle time is 2.2 minutes.

$$L = 35.7\left[\frac{4.9^{0.28}}{(1.0)\ 0.0004}\right]^{1/2}\left[\frac{.03^{1.18}}{0.7^{0.66}}\right]$$

$= 44.5$ lbs/ft^2/hr based on form time

$= 14.2$ lbs/ft^2/hr based on total cycle time

Using a scale to factor 0.8:

The design loading is:

$= 0.8 \times 14.2 = 11.3$ lbs/ft^2/hr

Solids to be dewatered $= \frac{25{,}000 \text{ lbs/day} \times 7 \text{ days/wk}}{98 \text{ hrs/wk operation}}$

$= 1780$ lbs/hr

Filter area $= \frac{1780 \text{ lbs/hr}}{11.3 \text{ lbs/ft}^2\text{/hr}} = 157 \text{ ft}^2$

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