PDHonline Course C708 (2 PDH)

An Introduction to Activated Sludge Wastewater Treatment Plants

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An Introduction to Activated Sludge Wastewater Treatment Plants

J. Paul Guyer, P.E., R.A.

CONTENTS

1. INTRODUCTION
2. ACTIVATED SLUDGE PROCESSES
3. CLOSED-LOOP REACTOR DESIGN CRITERIA
4. EXAMPLE CALCULATIONS
5. REFERENCES
6. BIBLIOGRAPHY

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(Figures, tables and formulas in this publication may at times be a little difficult to read, but they are the best available. **DO NOT PURCHASE THIS PUBLICATION IF THIS LIMITATION IS NOT ACCEPTABLE TO YOU.**)

1. GENERAL CONSIDERATIONS. The activated sludge process has been employed extensively throughout the world in its conventional form and modified forms, all of which are capable of meeting secondary treatment effluent limits. This publication presents the different modifications of the conventional activated sludge process, including general bases for design, methods of aeration, and design factors for aeration tanks, final sedimentation units and sludge handling systems. Figures 1 through 4 are schematic diagrams of the conventional and modified processes. The characteristics and obtainable removal efficiencies for these processes are listed in table 3. All designed processes will include preliminary treatment consisting of bar screen as a minimum and, as needed, comminutor, grit chamber, and oil and grease removal units.

2. ACTIVATED SLUDGE PROCESSES

2.1 CONVENTIONAL ACTIVATED SLUDGE. In a conventional (plug-flow) activated sludge plant (fig 1), the primary-treated wastewater and acclimated micro-organisms (activated sludge or biomass) are aerated in a basin or tank. After a sufficient aeration period, the flocculent activated sludge solids are separated from the wastewater in a secondary clarifier. The clarified wastewater flows forward for further treatment or discharge. A portion of the clarifier underflow sludge is returned to the aeration basin for mixing with the primary- treated influent to the basin and the remaining sludge is wasted to the sludge handling portion of the treatment plant. The portion recirculated is determined on the basis of the ratio of mixed liquor volatile suspended solids (MLVSS) to influent wastewater biochemical oxygen demand which will produce the maximum removal of organic material from the wastewater. Recirculation varies from 25 to 50 percent of the raw wastewater flow, depending on treatment conditions and wastewater characteristics.

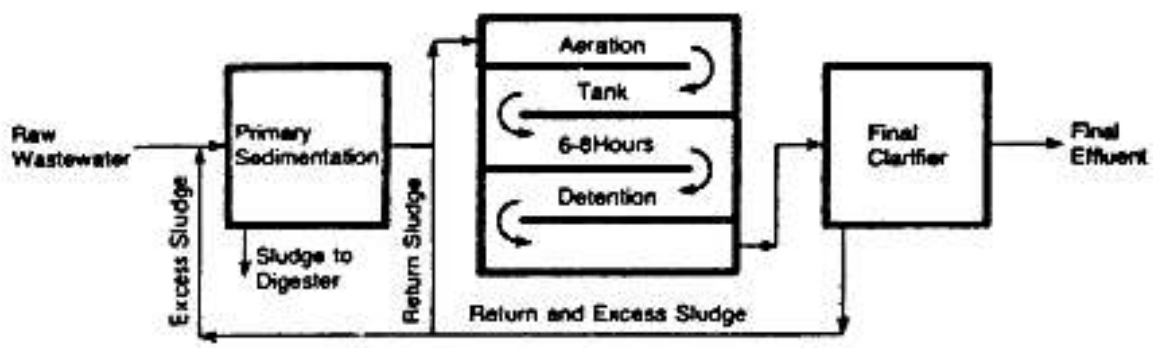


Figure 1

Conventional plug flow activated sludge flow diagram

2.2 STEP AERATION. In this process (fig 2), the influent wastewater is introduced at various points along the length of the aeration tank. Sludge return varies between 25 and 50 percent. Aeration or the oxygen requirement during step aeration (3 to 7 hours) is about half that required for the conventional process. This results from a more effective biomass utilization in the aeration basin, allowing organic loadings of 30 to 50 pounds biochemical oxygen demand per 1,000 cubic feet per day as compared to loadings of 30 to 40 pounds biochemical oxygen demand per 1,000 cubic feet per day permitted for conventional systems.

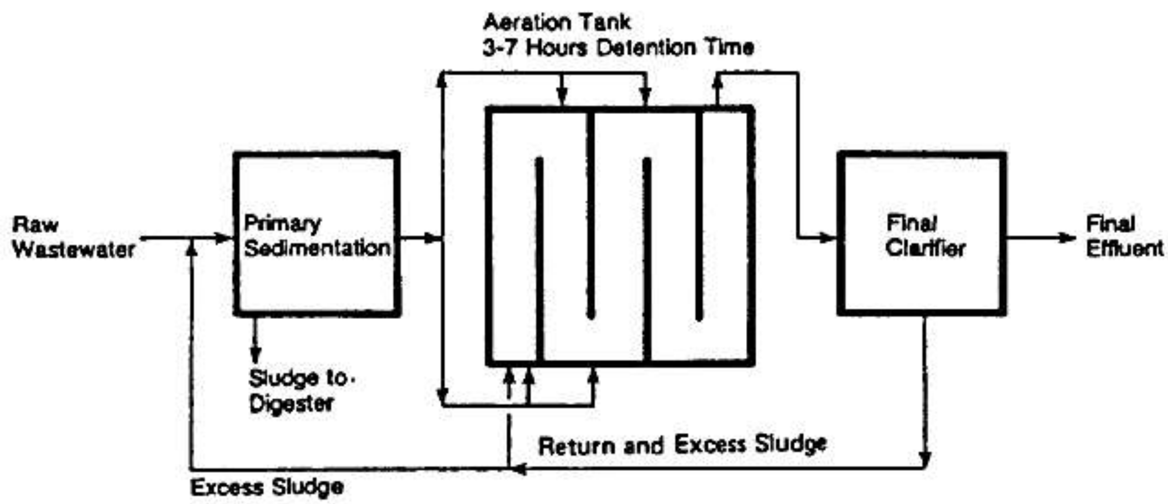


Figure 2

Step aeration flow diagram

2.3 CONTACT STABILIZATION. The contact stabilization activated sludge process (fig 3) is characterized by a two-step aeration system. Aeration of short duration (½ to 2 hours) is provided in the contact tank where raw or primary-settled wastewater is mixed with the activated sludge in the contact tank. The effluent from the contact tank is then settled in a final settling tank. The settled activated sludge to be recycled from the final clarifier is drawn to a separate re-aeration in a stabilization basin for 3 to 8 hours of aeration time. It is then returned to the contact aeration basin for mixing with the incoming raw wastewater or primary settled effluent. In addition to a shorter wastewater aeration time, the contact stabilization process has the advantage of being able to handle greater shock and toxic loadings than conventional systems because of the buffering capacity of the biomass in the stabilization tank. During these times of abnormal loadings, most of the activated sludge is isolated from the main stream of the plant flow. Contact stabilization plants will not be used where daily variations in hydraulic or organic loadings routinely exceed a ratio of 3:1 on consecutive days or for plants with average flows less than 0.1 million gallons per day without prior approval of the owner.

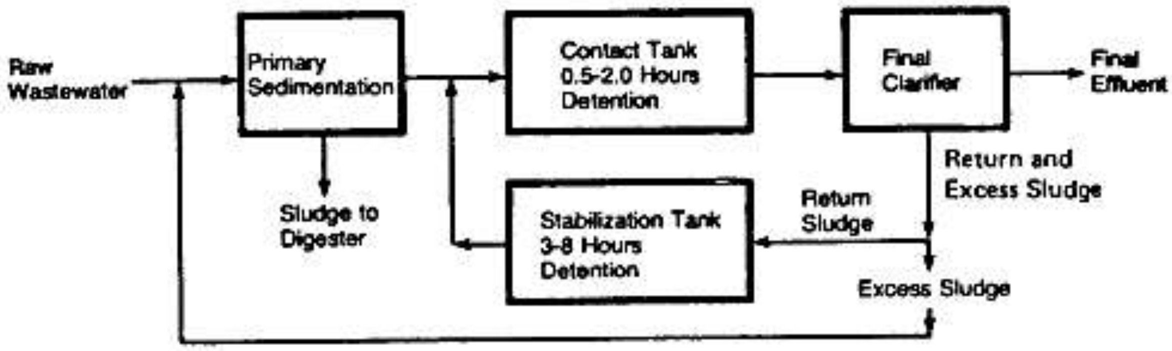


Table 3

Contact stabilization flow diagram

2.4 COMPLETELY-MIXED ACTIVATED SLUDGE. In the completely-mixed process (fig 4), influent wastewater and the recycled sludge are introduced uniformly through the aeration tank. This allows for uniform oxygen demand throughout the aeration tank and adds operational stability when treating shock loads. Aeration time ranges between 3 and 6 hours. Recirculation ratios in a completely-mixed system will range from 50 to 150 percent.

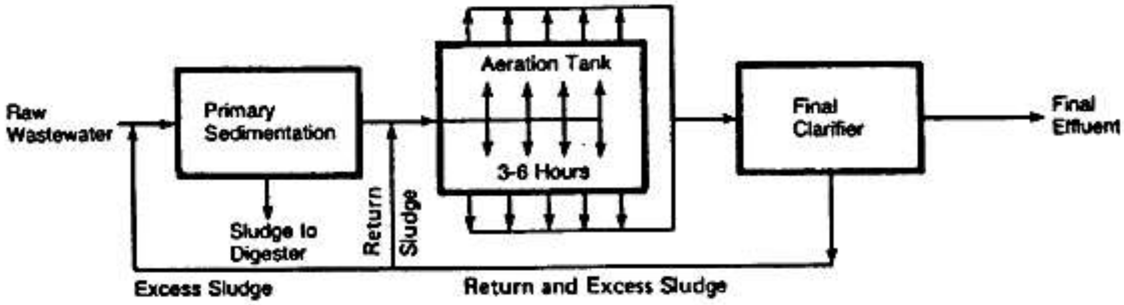


Figure 4

Completely-mixed process flow diagam

2.5 EXTENDED AERATION. Extended aeration activated sludge plants are designed to provide a 24-hour aeration period for low organic loadings of less than 20 pounds biochemical oxygen demand per 1,000 cubic feet of aeration tank volume. This approach, which can be used for treatment plants of less than 0.1 million gallons per day capacity, reduces the amount of sludge being wasted for disposal.

2.6 OXIDATION DITCH. The closed-loop reactor, also known as an oxidation ditch (fig 5), is a form of the extended aeration process. The wastewater is propelled around an oval racetrack-configured basin by mechanical aerator/mixing devices located at one or more points along the basin. These devices can be either brush aerators, surface aerators or jet aerators. The velocity in the basin is designed to be between 0.8 and 1.2 feet per second. The closed-loop reactor is the preferred type of activated sludge system for many installations.

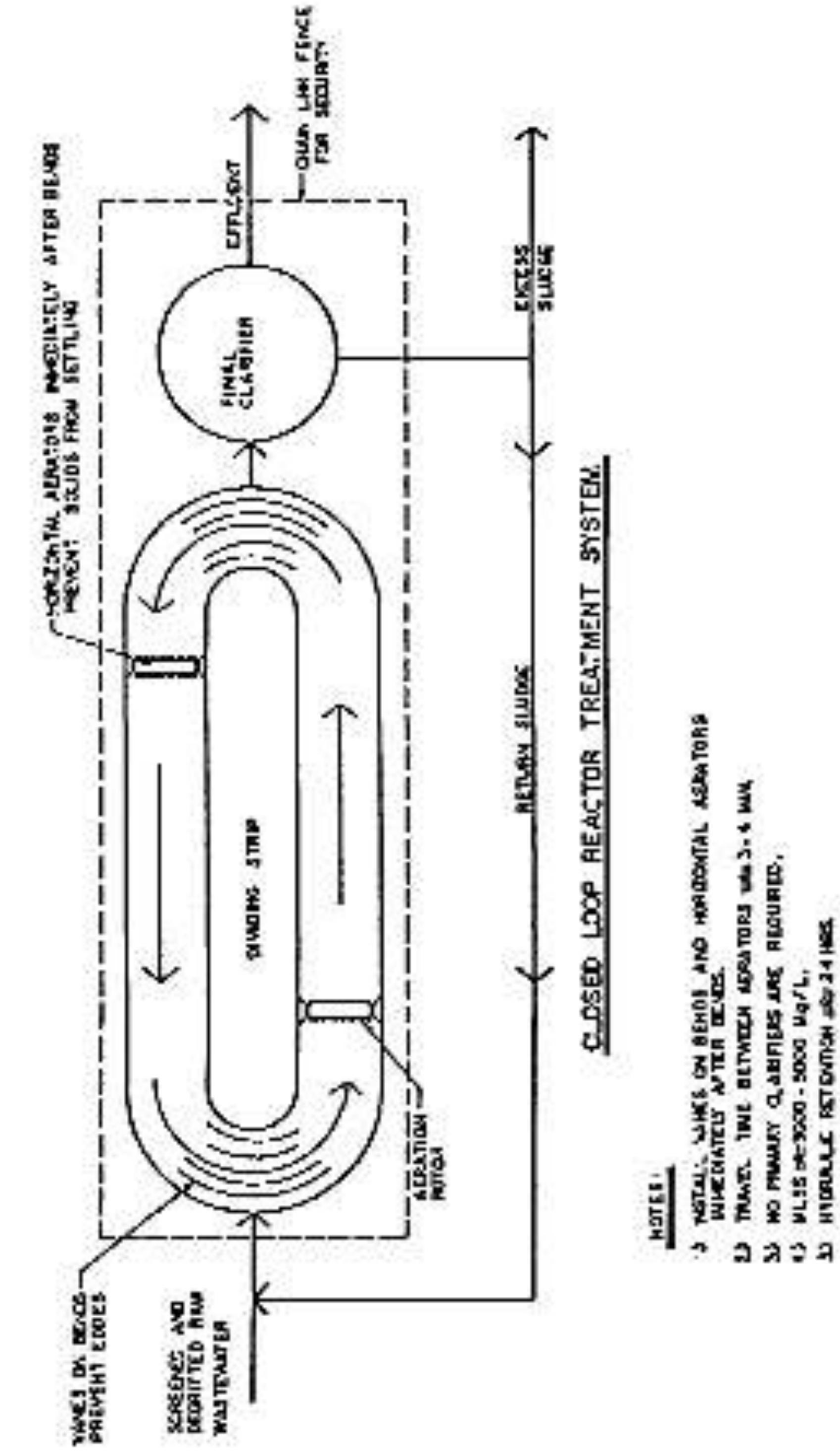


Figure 5

Closed-loop reactor treatment system

3. CLOSED-LOOP REACTOR DESIGN CRITERIA

3.1 GENERAL. Table 1 presents the design criteria to be used for the design of a closed-loop reactor plant.

Parameter	Value
Primary clarifier	None required
Hydraulic retention time	18–24 hrs
Sludge retention time	20–30 days
Secondary clarifier	
Overflow rate	450 gpd/sq ft
Solids loading rate	15 lb/sq ft/day

Table 1

Closed-loop reactor design criteria

3.2 AERATION TANK DESIGN. All oxidation ditch plants use looped channels or ditches. A looped channel with a partition in the middle may be shaped like an oval or a concentric ring. The design engineer may adopt a specific channel configuration and flow scheme recommended by the equipment manufacturer or supplier.

3.2.1 CHANNEL DEPTH. The number of loops and the channel depth are dependent upon the size of the plant. A shallow channel, less than 14 feet deep, is used for smaller plants with unlimited land area available. A deep channel, greater than 20 feet, should be designed for larger plants or to conserve heat.

3.2.2 NUMBER OF CHANNELS. Multiple-channel or multiple-loop is the preferable design so that part of the plant can be shut down for repair and maintenance.

3.2.3 DRAINAGE. A drain should be provided for each channel. This provision allows mixed liquor or accumulated grit material to be drained from the channel without expensive pumping. Many oxidation ditch plants do not have drains in their channels and are having maintenance problems.

3.2.4 CHANNEL LINING. Deep channels are to be built exclusively with reinforced concrete. A concrete liner can be placed against the earth backing in shallow channels by pouring concrete or gunite (shotcrete) to a thickness of 3 to 4 inches. The concrete or gunite should provide a minimum compressive strength of 3,000 pounds per square inch in 28 days.

3.3 AERATION. Depending on the width and depth of the channel, various types of aerators can meet the oxygenation and mixing requirements.

3.3.1 ROTOR AERATOR. A rotor aerator is a horizontal shaft with protruding blades which rotates, thereby transferring oxygen into the wastewater and propelling it around the ditch. Figure 6 illustrates a typical horizontal-shaft aerator. The minimum length of shaft is 3 feet; the maximum length of shaft is 30 feet. This type of aerator is suitable for shallow channels.

3.3.2 INDUCTION AERATOR. This type of aerator, which is available in various sizes, draws the mixed liquor and air down a U-tube and discharges it for a distance downstream in the channel. Compressed air at low pressure can be injected near the top of the down-draft tube to enhance oxygenation. A bulkhead (which should be partially opened at the bottom) is required to separate the channel to maintain the flow circulation. This type of aerator is suitable for shallow to moderately deep channels.

3.3.3 JET AERATION. Jet aeration is specifically designed for deep channels. Both air and the mixed liquor are pressurized (by aspirator pumping) into a mixing chamber from where the mixture is discharged as a jet stream into the surrounding channel liquid. Deep channels are used to take advantage of better oxygen transfer. Figure 7 illustrates a jet aerator, among various other types of aerators.

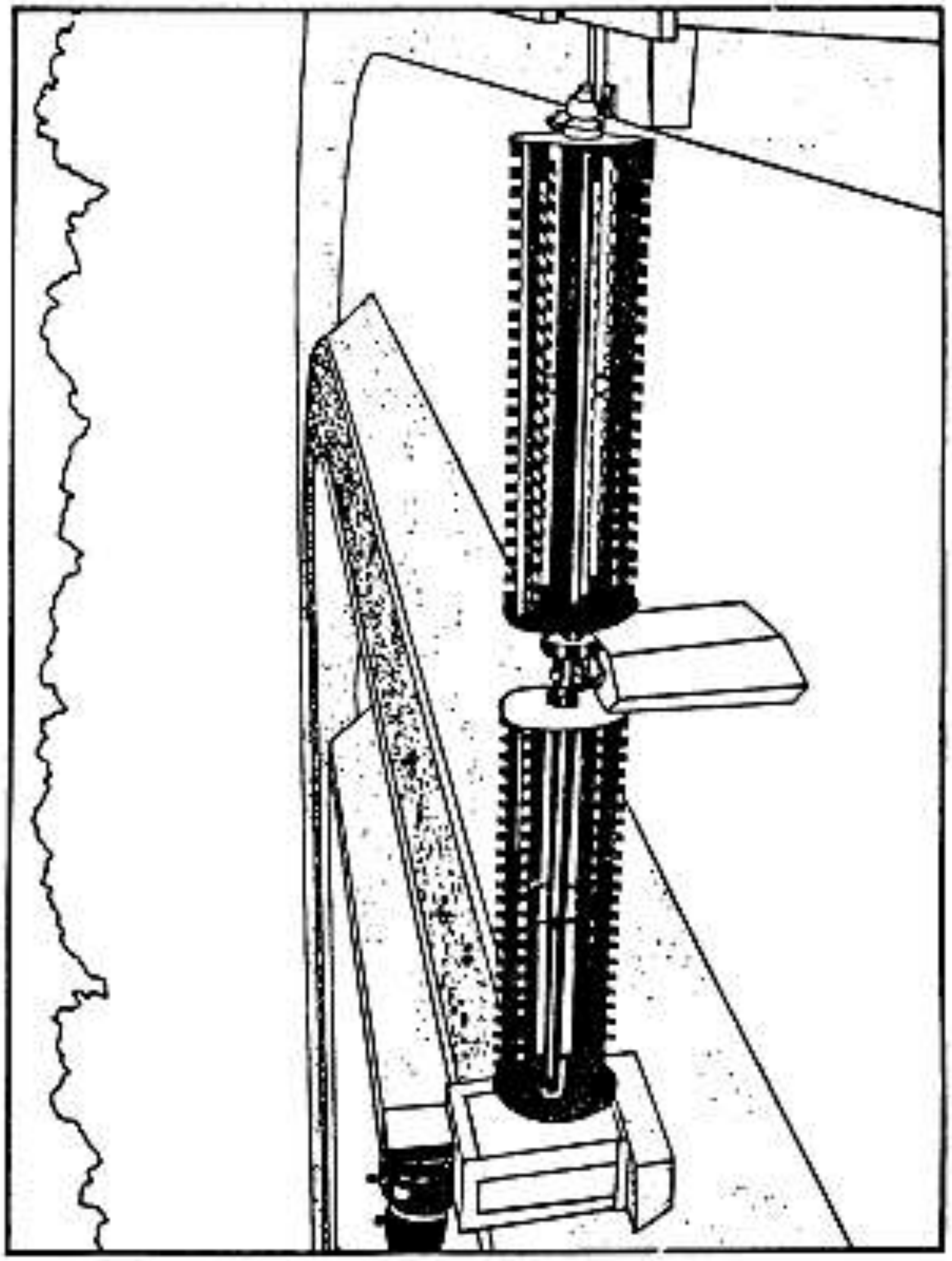


Figure 6

Horizontal shaft aerator

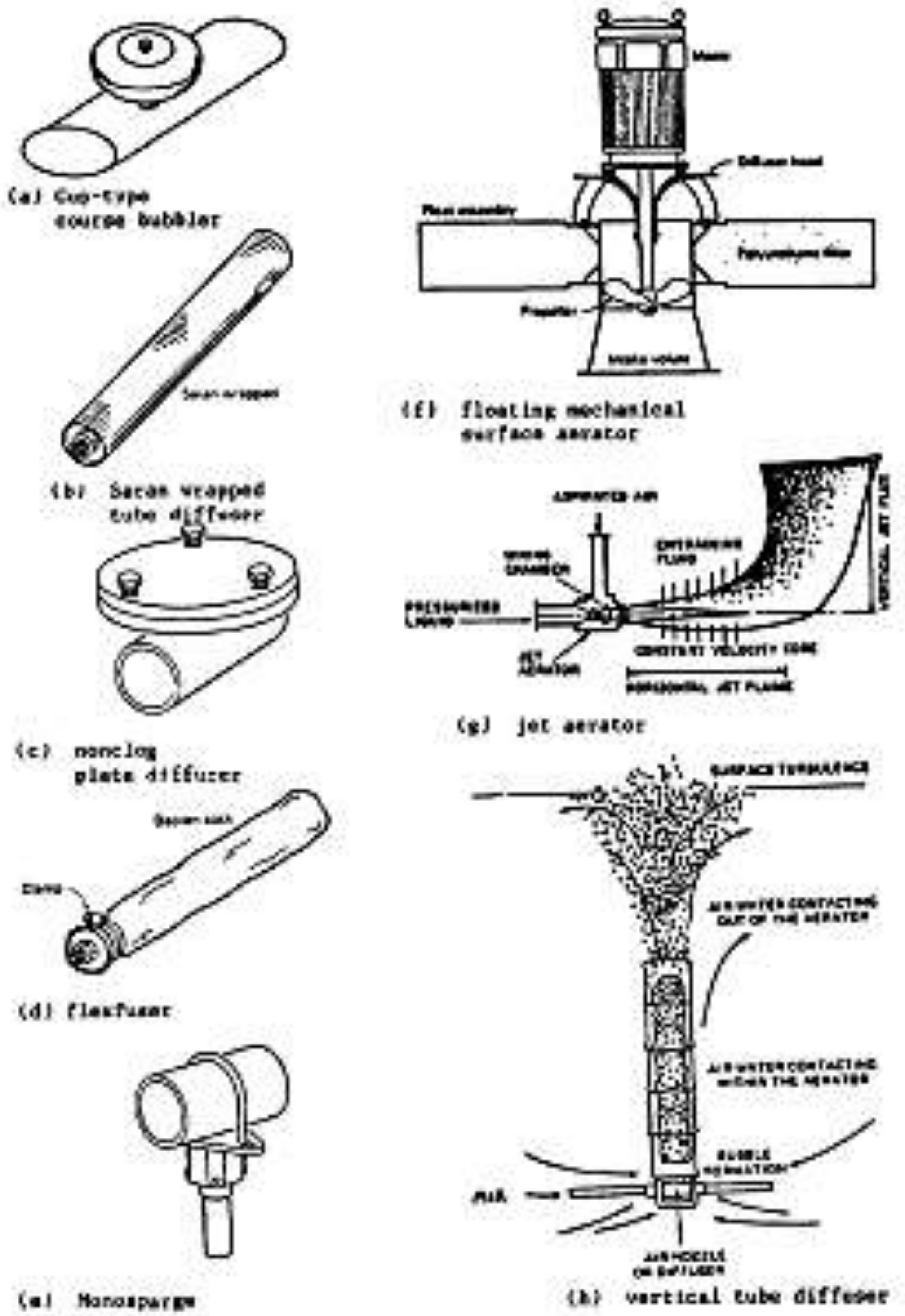


Figure 7

Aerators

3.3.4 DIFFUSED AERATION PLUS SLOW MIXER. This type of aeration is more suitable for deep channels. Air bubbles are introduced into the mixed liquor through a pipe grid system with diffusers to provide oxygenation while a slow propeller mixer provides the flow circulation and mixing.

3.3.5 AERATOR SIZING. Aerators should be sized to provide adequate mixing and oxygenation. However, the same size rotor provides different levels of mixing and oxygenation depending on the degree of its submergence. First, the oxygen requirement must be calculated for a level that will satisfy the carbonaceous biochemical oxygen demand removal as well as nitrification-denitrification (if needed). Oxidation ditch equipment manufacturers provide tables or charts for selecting the aerator size for any given speed and submergence (immersion) based on the calculated oxygen requirement. The aerator size should also be checked against the mixing requirement set by the manufacturers. Preferably, more than one aerator should be used per channel; they should be placed at different locations so that if one breaks down, the channel will still function. The procedure for selecting the jet aerator size is similar except there is no submergence factor. The sizing of the induction aerator and the diffused air plus slow mixer units is not precise. Design data for these new aeration systems are not yet available. One reason for this is that the amount of energy required for mixing relative to the energy required for oxygenation is uncertain since it depends a great deal on the channel geometry, which varies among plants. More testing data must be collected before a design criterion can be established.

3.4 SLUDGE DEWATERING AND DISPOSAL. Sludge from oxidation ditch plants operating in the extended aeration mode (sludge retention time of 20 to 30 days) can be wasted directly to open drying beds. It can also be wasted directly to tank trucks which spread the liquid sludge on the plant grounds or on adjacent land. The degree of sludge stabilization in the oxidation ditch is equivalent to that of a conventional activated sludge plant operated at a 10-day sludge retention time followed by aerobic digestion of the sludge for 7 to 15 days. In most climates, 1.0 square foot of drying bed surface area per population equivalent (0.17 pound biochemical oxygen demand per capita per day) should be used. This capacity can accept 2.2 cubic feet of wasted sludge per 100 capita per day, which is typical for domestic wastewater treatment. Double units of drying beds should be used so that half of them can be taken out of service for maintenance.

3.5 COLD CLIMATE. In moderately cold areas, ice buildup on clarifier scum collection boxes can cause problems and eventually jam the skimmer mechanisms. Therefore, final clarifiers should be covered. In cold areas, the spray from surface aerators will freeze on adjacent structures, bearings, gear reducers, etc., making maintenance difficult. Drive components and walkways near the aerators should be covered to shield them from spray, or mounted in isolated compartments. In very cold areas, heated covers for surface aerators should be provided. Ice fences should be installed across the channel upstream of brush-type aerators to prevent chunks of ice from breaking the brushes.

4. EXAMPLE CALCULATIONS

Activated sludge, closed-loop reactor.

a. The following design criteria are for an oxidation ditch with horizontal-shaft rotor aerators, using a single- channel, oval configuration and miltiple units in parallel, operated in an extended aeration mode with nitrification.

Influent requirements:

Q, avg daily flow	= 1.0 mgd;
Q, peak flow	= 2.0 mgd;
BOD_5	= 250 mg/b, typical of Army installations;
TSS	= 250 mg/b, typical of Army installations;
VSS	= 200 mg/b;
TKN	= 25 mg/b (all NH3 N);
P	= 8mg/b;
pH	= 6.5 to 8.5;
Minimum temp	= 10°C;
Maximum temp	= 25°C.

Effluent requirements:

BOD_5	≤ 10 mg/b;
TKN	≤ 5 mg/b;
NH_3^{-N}	≤ mg/b;
TSS	≤ 20 mg/b.

Waste characteristics:

Y = 0.8 lb solids produced/lb BOD_5 removed;
= total sludge produced;
k_d = 0.05 day^{-1}.

Clarifier characteristics:

Overflow rate ≤ 450 gpd/sq ft @ Q_{avg};
Solid loading ≤ 30 lb/sq ft-day.

Flow divided equally into two units in parallel, or Q = 0.5 mgd average daily flow each; the calculation shown below applies to either unit:

Q_{avg} = 0.5 mgd = 347.2 gpm;

Organic load = 0.5 mgd × 250 mg/b × 8.34 = 1042.5 lb BOD/day.

b. Calculation of oxidation ditch volume.

Use 20 lb BOD/1,000 cu ft—day;

$$V = \frac{1042.5 \text{ lb/day}}{20\text{lb}/1{,}000 \text{ cu ft-day}} = 52{,}125 \text{ cu ft;}$$

$$\text{Hydraulic detention time} = \frac{52{,}125 \times 7.48 \times 24}{0.5 \times 10^6} = 18.75 \text{ hr.}$$

c. Calculation of rotor requirements.

Rotor mixing requirement = 16,000 gal/ft of rotor; recommended by bakeside Equipment Corporation for maintaining a channel velocity of 1.0 fps.

$$\text{Length of rotor} = \frac{52{,}125 \quad 7.48}{16{,}000} = 24.4\text{; use 25 ft.}$$

Oxygenation requirement = 2.35 lb O_2/lb BOD-recommended for domestic sewage. Assume the following operating conditions:

RPM = 72;
Immersion = 8 in.

Oxygenation = 3.75 lb O_2/hr-ft (from fig C-2), using a 42-in diameter MAGNA rotor.

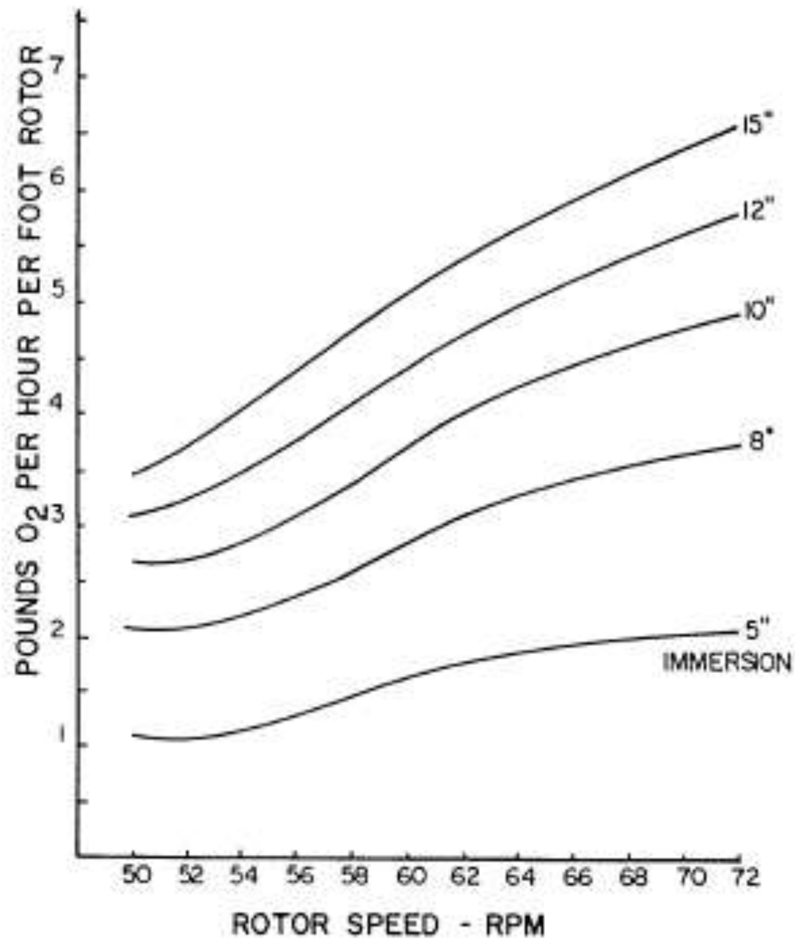


Figure C-2. Rotor aerator oxygenation capacity curve.

$$\text{Length of rotor} = \frac{1042.5 \times 2.35}{24 \times 3.75} = 27.2\text{; use 28 ft.}$$

Use two rotors per each unit oxidation ditch or 2- × 14-ft length rotor.

Theoretical oxygen transfer requirement:

$$\text{lb } O_2/\text{hr-ft} = \frac{1042.5 \times 2.35}{24 \times 28} = 3.65$$

Actual immersion = 8 in.

Power requirement = 0.84 kW/ft of rotor (from fig C-3), using 42-in diameter MAGNA rotor.

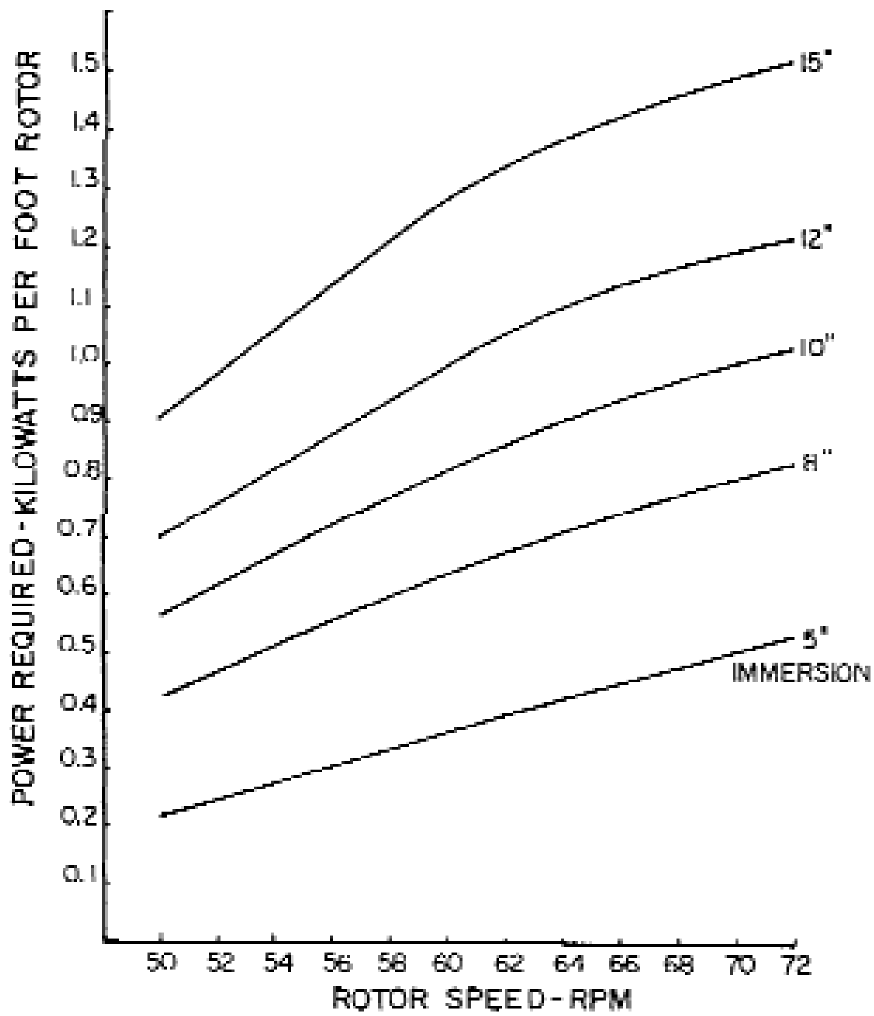


Figure C-3. Rotor aerator power requirements curve.

Aerator brake horsepower requirement = $1.34 \times 0.84 \times 14 = 15.75$ BHP for each rotor.

d. Calculation of motor horsepower. Size all motors to allow at least 1½ inch above the 8-in actual immersion to allow for peak flows.

Motor horsepower required at $(8 + 1.5) = 9.5$ in;

$$= \frac{0.99 \text{ BHP} \times 14 \times 1.34}{0.95} = 19.6 \text{ hp};$$

Use standard horsepower, or 20 hp each.

e. Channel sizing (fig C-4).

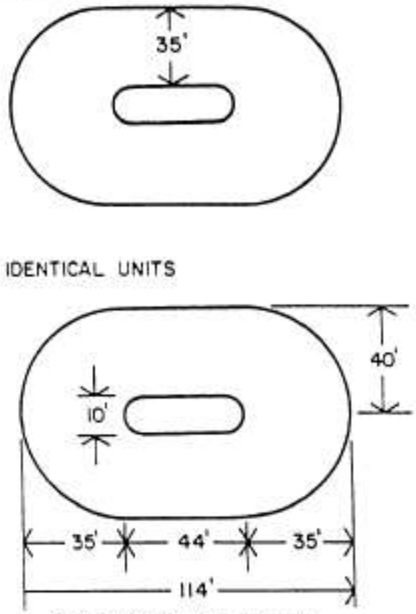


Figure C-4. Closed-loop reactor channel sizing

Ditch liquid volume = 52,125 cu ft.

Choose:

Depth: 10 ft;
Side wall slope: 45 degrees;
Median strip width: 10 ft;
Ditch flat bottom width = rotor length + 1.0 ft = 14 + 1 = 15 ft;
Ditch width at water surface = 2 x 10 + 15 = 35 ft;

$$\text{Ditch cross-section area} = \left(\frac{15 + 35}{2}\right) \times 10 = 250 \text{ sq ft};$$

Curvature volume = (2)(3.142)(22.5)(250) = 35,325 cu ft (by Theorem of Pappus);
Ditch straight wall volume = 52,125 - 35,325 = 16,800 cu ft;

$$\text{Total length of the ditch at the waterline} = \frac{16{,}800}{2 \times 250} = 33.6, \text{ or } 34 \text{ ft};$$

Ditch width = 2(35) + 10 = 80 ft;
Total ditch length = 34 + 80 = 114 ft;
Overall ditch dimensions = 114 ft x 80 ft x 10 ft deep.

f. Final clarifier.

One clarifier required for each ditch unit (Spiraflow clarifier).

Overflow rate $\leq$ 450 gpd/sq ft;

Detention time = 3 hr;

Area required = $\dfrac{.05 \times 10^6}{1 \times 450}$ 1,111sq ft;

Diameter $= \left[\dfrac{1{,}111}{0.785}\right]^{0.5} = 37.6$, use 38 ft;

Actual area = $(38)^2 \times 0.785 = 1133.5$ sq ft;

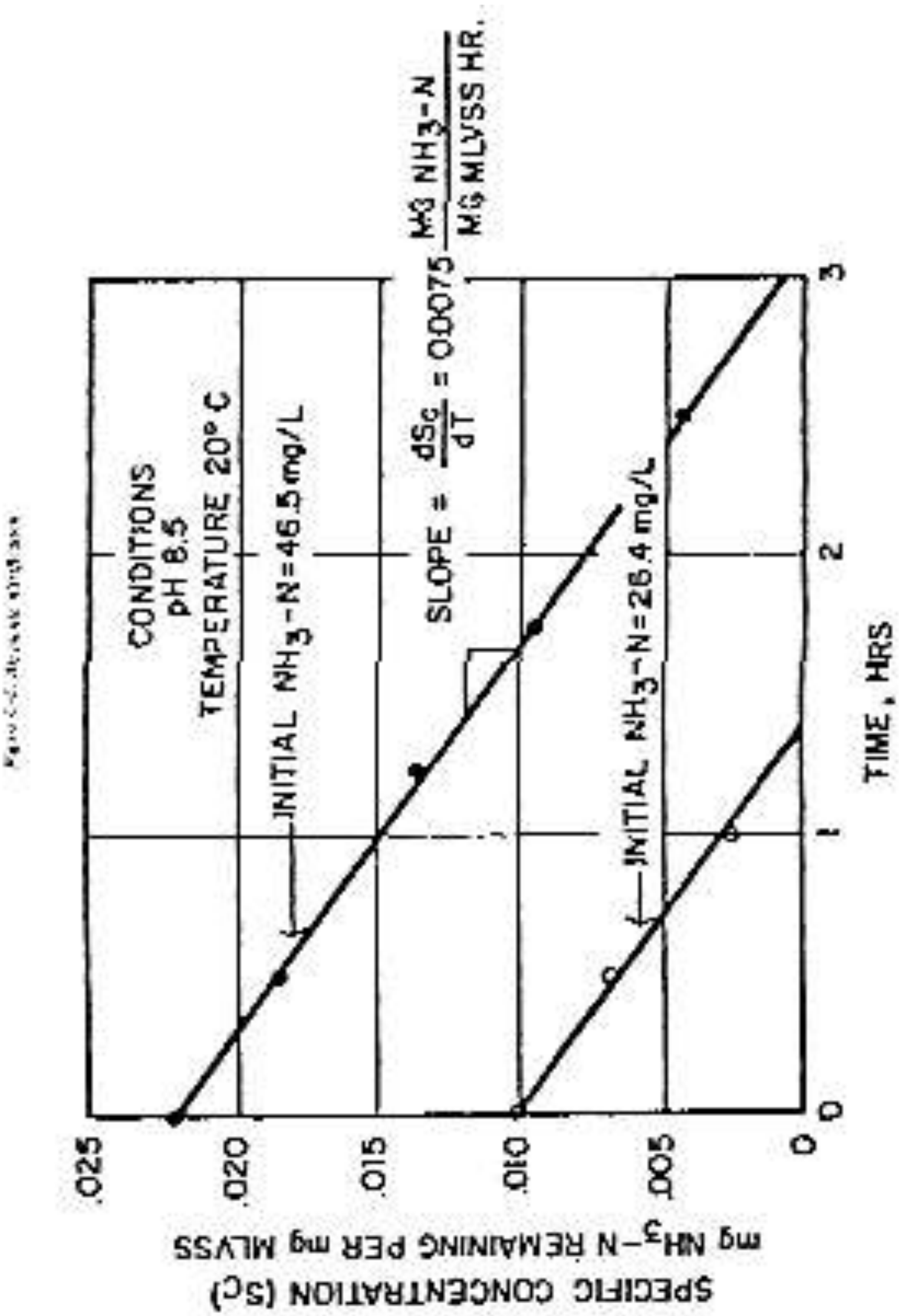
Actual overflow rate = $\dfrac{0.5 \times 10^6}{1133.5} = 441$ gpd/sq ft;

Volume = $\dfrac{0.5 \times 10^6 \times 3}{24 \times 7.48 \times 1} = 8356.5$ cu ft;

Straight wall dept = $\left(\dfrac{8356.5}{1133.5}\right) = 7.4$ ft, use 8 ft;

Actual detention time = $8 \times 1133.5 \times 24 \times 7.48/0.5 \times 10^6 = 3.25$ hr.

g. Check for nitrificaiton in oxidation ditch. Find MLVSS concentration required at standard conditions (20°C, pH8.5) to completely nitrify ammounia shown in figure C-5.


CONDITIONS
pH 8.5
TEMPERATURE 20° C
INITIAL NH_3-N=46.5 mg/L
SLOPE = $\frac{dS_c}{dT}$ = .00075 $\frac{MG\ NH_3-N}{MG\ MLVSS\ HR.}$
INITIAL NH_3-N=26.4 mg/L
.025
.020
.015
.010
.005
0
0
1
2
3
TIME, HRS
SPECIFIC CONCENTRATION (S_c)
mg NH_3-N REMAINING PER mg MLVSS

$$MLVSS = \frac{1}{0.0075} \times \frac{25\ mg/L\ NH_3^{-N}}{18.75\ hr.} = 177.8\ mg/L,$$

where 0.0075 mg NH_3^{-N}/mg MLVSS-hr is the rate of nitrification given in figure C-5.

Change MLVSS concentration at standard conditions to design conditions (10°C, pH 6.5 to 8.5[assume pH is 7.0]), using figures C-6 and C-7 for temperature and pH corrections.

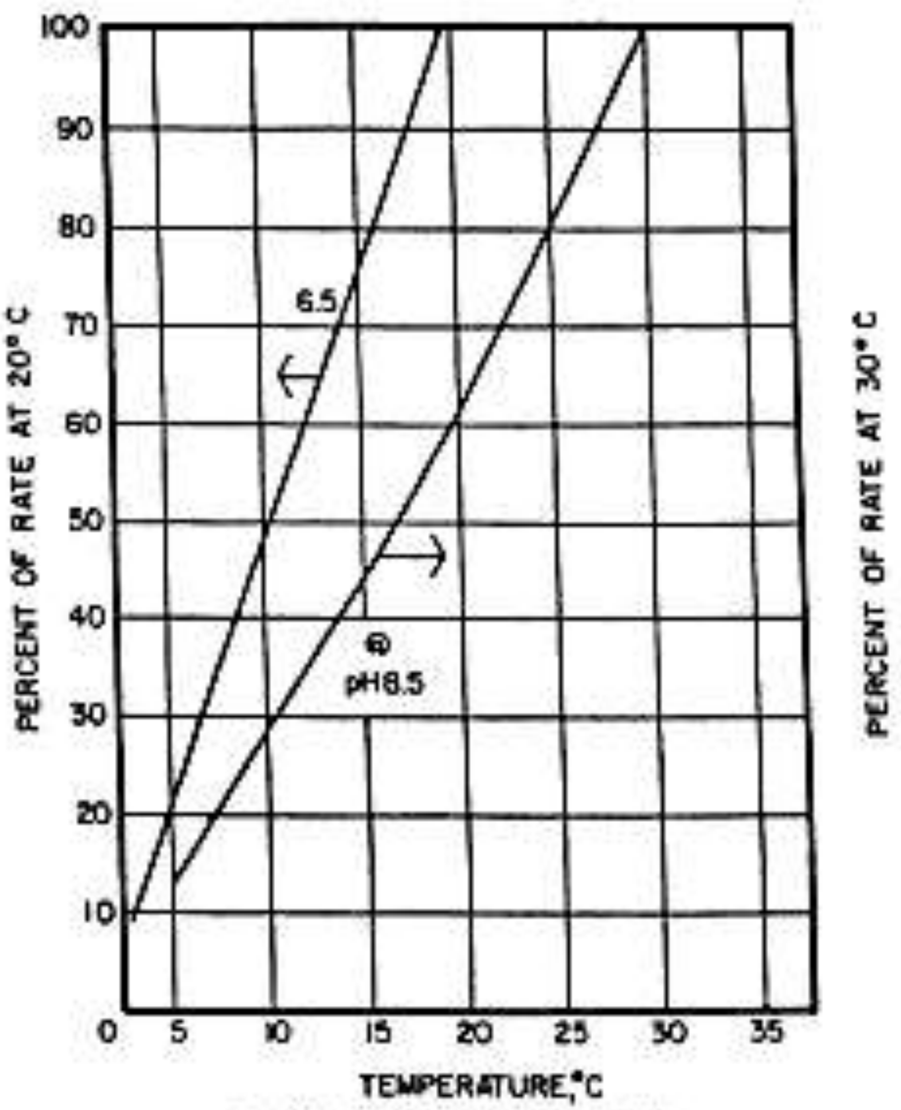
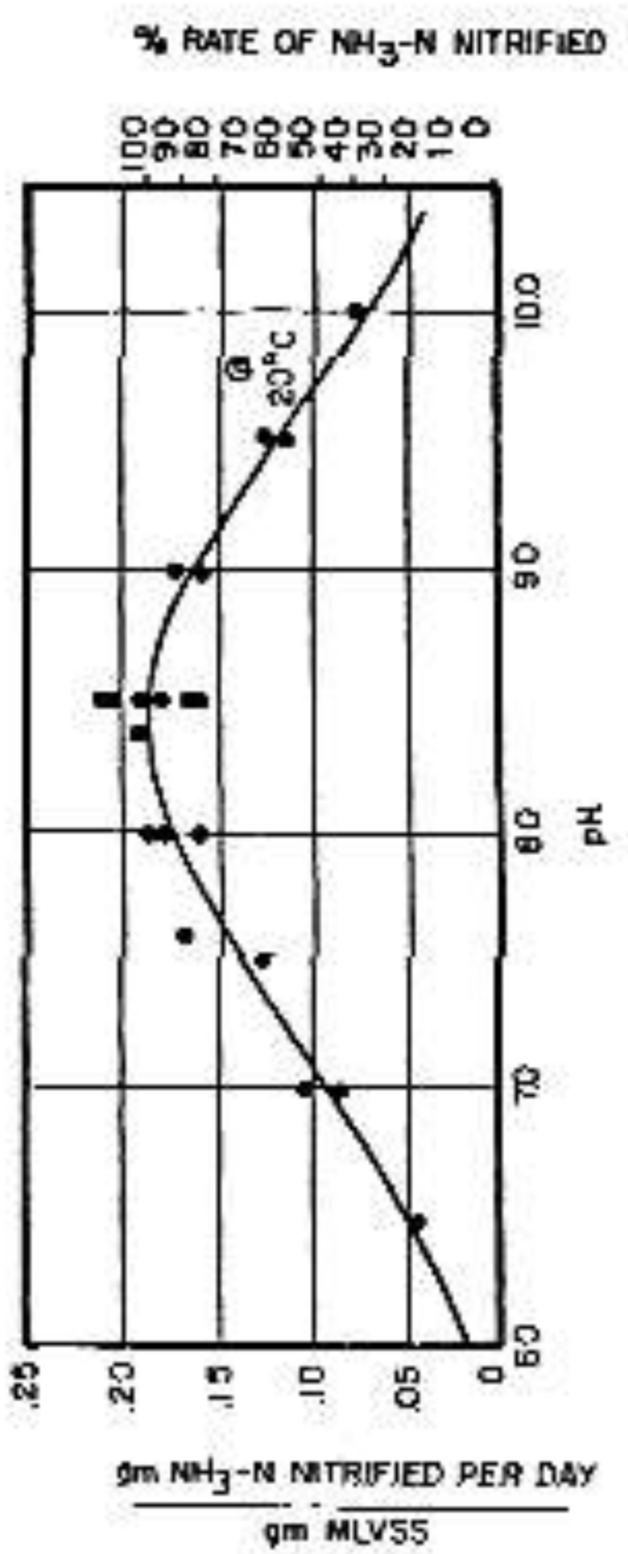


Figure C-6. Nitrification temperature corrections.


% RATE OF NH3-N NITRIFIED
100 90 80 70 60 50 40 30 20 10 0
@ 20°C
6.0 7.0 8.0 9.0 10.0
pH
.25 .20 .15 .10 .05 0
gm NH3-N NITRIFIED PER DAY
gm MLVSS

Temperature correction = 0.46 (from 20°C to 10°C),
pH correction = 0.50 (pH 8.5 to 7.0),

$$\text{MLVSS concentration} = \frac{177.8 \text{ mg/L}}{0.46 \times 0.50} = 774 \text{ mg/L}.$$

This concentration is low, the sludge age would be too short to give a stable performance. To obtain a stabilized sludge.
Use 3200 MLVSS,

$$\text{MLVSS} = \frac{3200}{0.8} = 4000 \text{ mg/L}.$$

Check sludge age (with negligible sludge wasting):

$$c = \frac{\text{MLVSS}}{Y(\text{BOD})} = \frac{3200}{0.8(250)} = 16 \text{ days}.$$

This sludge age is considered adequate.

h. Check size of final clarifier with solids loading.
Spiraflow clarifier allows 30 lb solids/sq ft-day.

$$\text{Actual solids loading} = \frac{(441 \text{ gpd/sq ft})(8.34)(4000)}{500{,}000} = 29.4 \text{ lb/sq ft-day} < 30.$$

i. Adjustable oxidation ditch weir. A weir on each oxidation ditch is provided and sized so that the head differential over the weir between the maximum and minimum flow rates is less than 1.25 in.

Maximum flow rate = Q_{peak} + maximum recirculation rate;
Minimum flow rate = Q_{min} + recirculation rate.
For average daily flows between 200,000 gal/day to 1 mgd, the length (L) of the overflow weir is:

$$L = \frac{3.5 \times Q_{avg} \text{ gpm}}{102} = \frac{3.5 \times 347.2}{102} = 11.9, \text{ use } 12 \text{ ft}.$$

j. Calculate the return sludge flow rate, Q_R:

The equipment manufacturer recommends the following:
Minimum pumping capacity = $0.25 \times Q_{avg}$ = 0.25×347.2 gpm = 87 gpm.
Maximum pumping capacity = $1.0 \times Q_{avg}$ = 1.0×347.2 gpm = 347 gpm.
2 pumps, each with 44 to 174 gpm capacity.

k. Calculate amount of sludge for disposal. The amount of sludge wasted is the same as in the vertical-shaft aerator example since the influent and effluent characteristics and the MLVSS concentrations are identical.
Px = 683.2 lb/day of VSS from 2 oxidation ditches.
Total sludge 933.4 lb/day dry solids from 2 oxidation ditches.
2 pumps, each with a capacity of 120 gpm for a 52-min/day wasting schedule as in the vertical-shaft aerator example.

l. Sizing of sludge drying beds.

$$\text{BOD population equivalent} = \frac{1 \times 250 \ 8.34}{0.17} = 12{,}265 \text{ persons};$$

Drying bed area = 12,265 × 1 sq ft/cap = 12,265 sq ft.
Use 10 drying beds, each at 35 ft × 35 ft.
Total area = 12,250 sq ft.

Table C-4 summarizes the design criteria for the horizontal-shaft aerators and multiple-ditch units.

Table C–4. Summary of closed–loop reactor design.

Quantity	Unit	Size
2	Oval-shaped oxidation ditch with 10-ft wide median strip; 45 degree slope. Lining using gunite or shotcrete method of construction.	114 ft × 80 ft, 10 ft deep
4	Horizontal–shaft rotor aerator; 2 ea in oxidation ditch	14 ft long ea; 16 BHP ea (motor 20 hp)
2	Secondary clarifier.	38 ft dia × 8 ft SWD (Spiraflo clarifier)
2	Return sludge pump.	0.125 to 0.5 mgd ea (41 to 174 gpm)
2	Sludge wasting pump.	120 gpm ea
2	Adjustable oxidation ditch weir.	12 ft ea
10	Sludge drying bed	35 ft × 35 ft ea

5. REFERENCES

5.1 GOVERNMENT PUBLICATIONS

PL 92-500 Federal Water Pollution Control Act

5.1.1 DEPARTMENT OF DEFENSE

AFM 88-15 Air Force Design Manual-Criteria and Standards for Air Force Construction
AFP 19-5 Environmental Quality Control Handbook: Industrial Wastes
AFR 19-1 Pollution Abatement and Environmental Quality
AR 200-1 Environmental Protection and Enhancement
TM 5-813-5/AFM 88-10, Vol.5 Water Supply Water Distribution Systems
TM 5-814-1/AFM 88-11, Vol.1 Sanitary and Industrial Waste Sewers
TM 5-814-2/AFM 88-11, Vol.2 Sanitary and Industrial Wastewater Collection—Pumping Stations and Force Mains
TM 5-814-6 Industrial Wastes
TM 5-814-8 Evaluation Criteria Guide for Water Pollution: Prevention, Control, and Abatement
TM 5-852-1/AFR 88-19, Vol.1 Arctic and Subarctic Construction: General Provisions
TM 5-852-4/AFM 88-19, Chap. 4 Arctic and Subarctic Construction: Building Foundations
TM 5-852-5/AFR 88-19, Vol.5 Arctic and Subarctic Construction: Utilities

5.1.2 ENVIRONMENTAL PROTECTION AGENCY (EPA)

R-2-73-199 Application of Plastic Media Trickling Filters for Biological Nitrification Systems
625/1-74-006 Process Design Manual for Sludge Treatment and Disposal (Oct 74)
625/1-75-003a Process Design Manual for Suspended Solids Removal (Jan 75)
625/1-76-001a Process Design Manual For Phosphorus Removal (Oct 71)
625/1-80-012 Process Design Manual for Onsite Wastewater Treatment and Disposal Systems (Oct 80)

625/1-81-013 Process Design Manual for Land Treatment of Municipal Wastewater (Oct 81)

625/1-82-014 Process Design Manual for Dewatering Municipal Wastewater Sludges (Oct 82)

625/1-83-015 Process Design Manual for Municipal Wastewater Stabilization Ponds (Oct 83)

Process Design Manual for Carbon Absorbtion (Oct 73)

Process Design Manual for Nitrogen Control (Oct 75)

Process Design Manual for Upgrading Exist-Wastewater Treatment Plants (Oct 75)

Handbook for Monitoring Industrial Wastewater (Aug 73)

5.2 NON-GOVERNMENT PUBLICATIONS

5.2.1 AMERICAN WATERWORKS ASSOCIATION (AWWA)

6666 West Quincey Avenue, Denver CO 80235

Standard Methods for the Examination of Water and Wastewater

16th Edition, Franson, M.A. (ed), APHA, WPCF (1984)

Safety Practices for Water Utilities

5.2.2 WATER POLLUTION CONTROL FEDERATION (WPCF)

Manual of Practice No.1 Safety and Health in Wastewater Works (1983)

Manual of Practice No.8 Wastewater Treatment Plant Design (1977)

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