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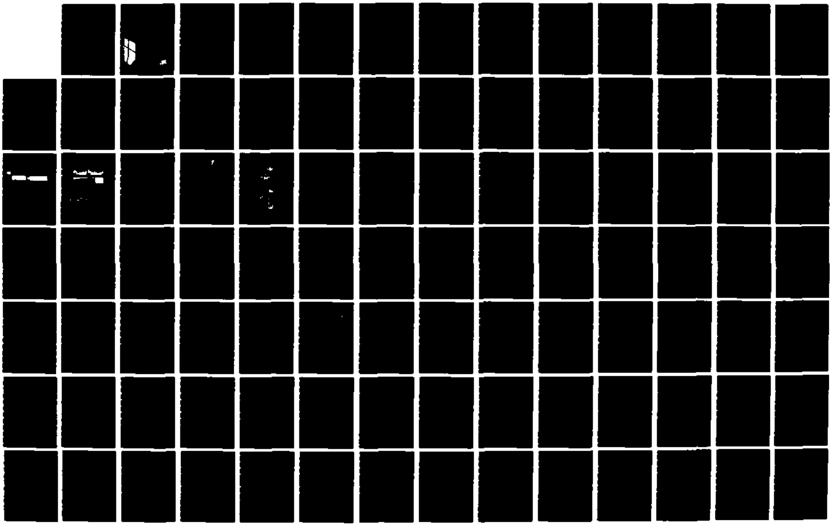

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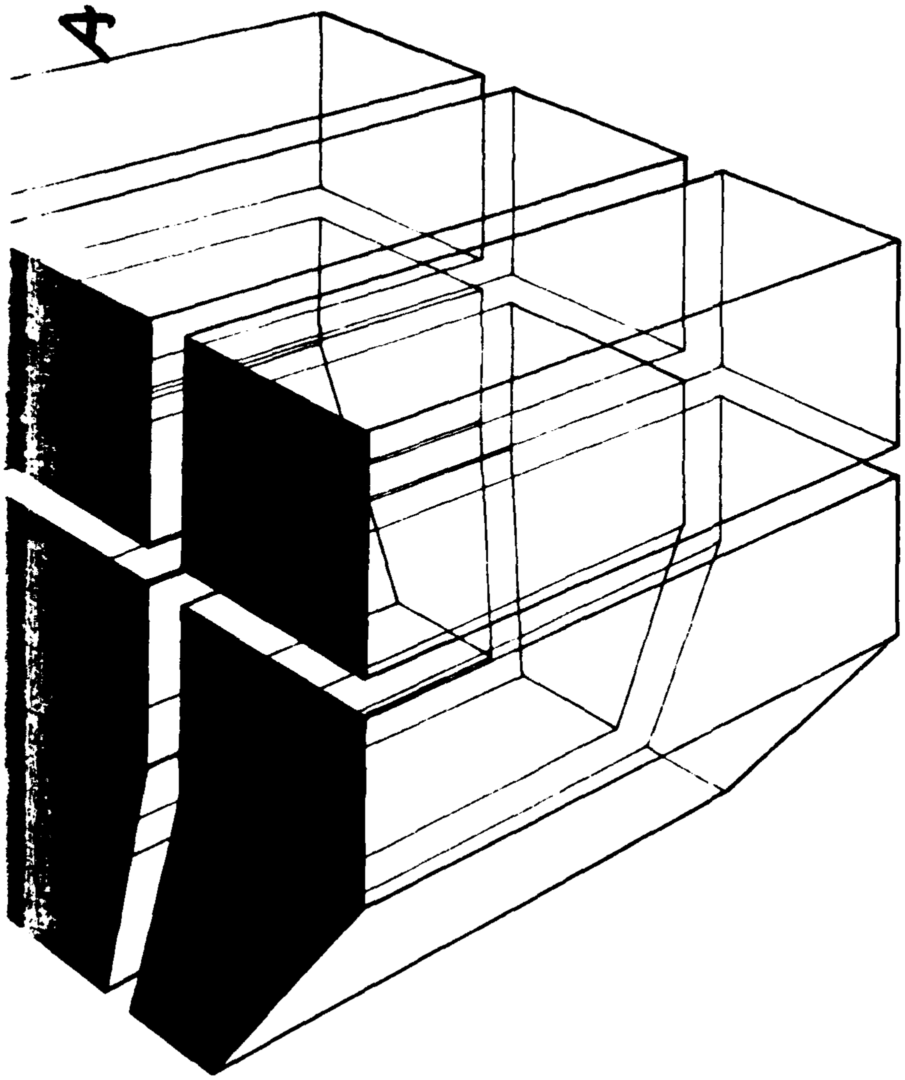
Technical Report N-151
August 1983
Design and Operation for Upgrading Wastewater Treatment Plants

AD-A133 140

OXIDATION DITCH TECHNOLOGY FOR UPGRADING ARMY SEWAGE TREATMENT FACILITIES



by
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Army installations in the United States own and operate more than 100 wastewater treatment plants. The treatment capacities of most of these plants are adequate and can meet the National Pollutant Discharge Elimination System treatment standards. However, many others need upgrading.

Oxidation ditches are a new treatment technology which is better than more traditional processes for upgrading Army wastewater treatment plants or

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered) for building new ones. Within the flow range of 0.15 to 10 mgd, oxidation ditches are more cost-effective than competing biological processes. They are easy to obtain and install, simple to operate and maintain, and require relatively little equipment. They are flexible and are adaptable for both nitrification and denitrification. They can stabilize sludge without the use of expensive and unreliable anaerobic digestion. Because of these advantages, oxidation ditch technology should be considered when planning wastewater treatment plant facilities at Army installations and when planning mobilization requirements.

This report provides guidelines for oxidation ditch selection. Three design examples are presented: one for the Carrousel system with vertical shaft aerators, one for oval ditches with horizontal shaft aerators, and one for deep channel ditches with jet aeration. All systems have the same capacity or design flow (1.0 mgd) and can provide both BOD removal and nitrification. The deep channel oxidation ditch design also provides for denitrification. Cost estimates are given for the three systems. Operational and maintenance problems are discussed, and recommended solutions are provided. Suggestions for an operator training program are also given.

FOREWORD

This study was conducted for the Directorate of Engineering and Construction, Office of the Chief of Engineers (OCE), under Project 4A762720A986, "Environmental Quality Technology"; Task B, "Source Reduction Control and Treatment"; Work Unit 043, "Design and Operation for Upgrading Wastewater Treatment Plants." The work was performed by the Environmental Division (EN) of the U.S. Army Construction Engineering Research Laboratory (CERL). The applicable QCR is 6.27.20A. The OCE Technical Monitor was Walter Medding, DAEN-ECE-G. Dr. R. K. Jain is Chief of EN.

COL Paul J. Theuer, PE, is Commander and Director of CERL, and Dr. L. R. Shaffer is Technical Director.

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OXIDATION DITCH TECHNOLOGY FOR UPGRADING ARMY SEWAGE TREATMENT FACILITIES

1 INTRODUCTION

Background

The Department of the Army owns and operates more than 100 wastewater treatment plants. Most of these plants have adequate treatment capacity and can meet the requirements of their National Pollutant Discharge Elimination System (NPDES) permits. However, others now or may in the future require upgrading. In addition, it has always been hard for the Army to recruit and keep enough highly skilled operating personnel for wastewater treatment plants. Since the way wastewater treatment plants are operated critically affects their performance, the Army prefers simple-to-operate treatment technologies.

Oxidation ditches -- a new treatment technology -- meet these criteria. In the early 1960s, there were about 10 oxidation ditch plant installations in the United States. By 1975 there were 90, and by 1980, there were 650 shallow ditches in municipal, industrial, and institutional installation applications throughout the United States and Canada. Many shallow and deep oxidation ditch systems are used in Europe. Over the last two decades, the use of oxidation ditches in this country has been successful in both warm and cold regions. There has been sufficient evidence to show several advantages of this process over other alternatives.

1. Within the range of 0.15 to 10 mgd* capacity (which includes 75 percent of the Army's plants), the construction costs are equal to or less than costs for competitive treatment processes.

2. The process requires a minimum of mechanical equipment.

3. The process gives higher performance reliability, even with minimum operator attention or lower operator competence, primarily due to conservative design.

4. Waste sludge is relatively nuisance-free and is readily disposed of at most plants (drying bed followed by land disposal).

5. The process does not generate odors, even under poor operating conditions.

To use this innovative technology, the Army needs guidance on the selection, design, and operation of oxidation ditches.

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* SI unit conversion factors are given on p 108.

Objective

The objectives of this study were to (1) evaluate oxidation ditches as an appropriate means of wastewater treatment for the Army, and (2) provide guidance on selecting, designing, and operating oxidation ditch technology for installation Facilities Engineering personnel and Corps of Engineers District design engineers and Architect/Engineers.

Approach

Army wastewater treatment practice was studied (Chapter 2), and special requirements which might influence the Army's selection of wastewater treatment technologies were identified (Chapter 3). Oxidation ditch technology was then evaluated in terms of the Army's requirements (Chapter 4). Guidelines for oxidation ditch selection were developed and design and costing procedures were generated (Chapter 5).

Scope

The information in this report will be useful primarily to installations which produce wastewater similar to typical domestic sewage. The information will be useful for upgrading, mobilization, and new construction activities.

Mode of Technology Transfer

It is recommended that information from this study be incorporated into Technical Manual (TM) 5-814-3, Domestic Wastewater Treatment; TM 5-814-8, Evaluation Criteria Guide for Water Pollution Prevention Control and Abatement Programs; and TM 5-665, O&M of Wastewater Treatment Facilities (January 1982). An Engineer Technical Note based on this information will also be published.

2 ARMY INSTALLATION SCENARIO

Most Army installations in the United States occupy from thousands to more than 1 million acres of land. The population at these installations ranges from several thousand to tens of thousands. Although regular water supply and sewerage services are generally unavailable to remote installation sites (e.g., firing ranges, distant gates and guard stations, ceremonial grounds, seasonally used parks and recreational areas), water is piped to all cantonment areas and areas with sufficient population and activities.

Treatment Plant Scenario

In 1979, the Office of the Chief of Engineers surveyed existing Army wastewater treatment facilities in the United States. Each facility submitted information on its treatment facility capacity, plant performance, unit processes, personnel inventory and training, and energy consumption. These data have been categorized and tabulated for this study (Tables 1 through 7). Analysis of the data was helpful in drawing up the scenario of wastewater treatment in Army installations.

The summarized data in Table 1 show that 114 plants (91.2 percent of the total) have primary and secondary treatment, while only 17 (13.6 percent) have tertiary treatment. Fifty-six percent of the secondary treatment plants use trickling filters, while activated sludge plants account for 22.4 percent. Recently, several new technologies have been applied at Army installations: rotating biological contactors, the oxidation ditch (one installed in 1981-1982), and land treatment. However, less than 4 percent are using new technology.

Table 2 shows that half of all the Army's secondary treatment plants were built between 1940 and 1950. Another 34 percent were built between 1950 and 1970. Only 27 plants, or 24 percent, were built between 1970 and 1980. Although most plants' flows are lower than their design flows (Table 3), their performance does not meet the NPDES permits. Table 4 shows that only 63.7 percent of all plants meet their NPDES permits completely. Others either do not meet the biochemical oxygen demand (BOD) standards or suspended solids (SS), fecal coliform, or nitrogen and phosphorus standards. Table 4 shows that 27 plants using biological processes report an immediate need to upgrade treatment performance. Many more report that some upgrading is required. The 40 facilities listed in Table 6, which deal specifically with industrial wastes, use physical-chemical treatment technology since biological treatment would be ineffective. Thus, upgrading with biological treatment processes does not apply to this group.

Tables 5 and 7 identify the problems often reported by the Army treatment facilities and help explain why many plants cannot meet their NPDES permits. The two major, readily identifiable problems in Army treatment facilities are (1) inadequate training of personnel, and (2) insufficient manpower. Twenty-seven plants, or 19.4 percent, report inadequate training of personnel. Other related problems are inadequate training to handle industrial wastes, insufficient funding for training, lack of opportunity to upgrade operators, as well as inability to obtain or to retain trained personnel. When these are

Table 1

Type of Treatment

Types of Treatment Facilities	Number of Plants	Percentage of Plants
Plants with primary and secondary treatment	114	91.2%
Plants with primary, secondary, and tertiary treatment	17	13.6
Plants with Imhoff tanks	29	23.2
Plants with evaporation lagoons	17	13.6
Plants with trickling filters	70	56.0
Plants with activated sludge processes	28	22.4
Plants with rotating biological contactors	2	1.6
Plants with oxidation lagoons	2	1.6
Plants with land treatment	1	0.8
Plants with anaerobic sludge digestors	64	51.2
Plants with sludge-drying beds	83	66.4
Plants with surface discharge of effluent	108	86.4
Plants with subsurface discharge	17	13.6
Plants with carbon adsorption (as tertiary treatment)	2	1.6
Plants with dual-media filters (as tertiary treatment)	2	1.6

*Many installations have more than one wastewater treatment plant.

Table 2

Age of Treatment Facility

Plant Installation	Number of Plants	Percentage of Plants
Primary Treatment		
Before 1940	2	3
1940-1950	44	61
1950-1960	11	15
1960-1970	9	13
1970-1980	6	8
After 1980	0	0
Secondary Treatment		
Before 1940	0	0
1940-1950	49	45
1950-1960	12	11
1960-1970	22	20
1970-1980	27	24
After 1980	0	0
Tertiary Treatment		
Before 1940	0	0
1940-1950	1	6
1950-1960	0	0
1960-1970	0	0
1970-1980	14	88
After 1980	1	6

Table 3

Flow Characteristics

Flow Characteristics	Number of Systems	Percentage of Systems
Design Flow (mgd)		
0.001 - 0.01	7	5.9
0.01 - 0.1	23	18.4
0.1 - 0.5	34	28.6
0.5 - 1.0	17	14.3
1.0 - 2.5	20	16.8
2.5 - 5.0	15	12.6
5.0 - 10.0	3	2.5
Present Flow (mgd)		
0.001 - 0.01	25	21.9
0.01 - 0.1	26	22.8
0.1 - 0.5	31	27.2
0.5 - 1.0	8	7.6
1.0 - 2.5	20	17.5
2.5 - 5.0	3	2.6
5.0 - 10.0	1	0.9
Domestic Flow		
100%	51	40.8
85+	52	41.6
50 - 85	12	9.6
10 - 50	5	4.0
1 - 10	1	0.8
0*	4	3.2
(Balance of flow is industrial wastewater)		
Stormwater handled separately	117	

*Industrial flow treated by a biological process.

Table 4

Treatment Performance

Treatment Performance	Number of Plants	Percentage of Plants
Plants with NPDES permits	102	87.2
Plants meeting NPDES permits completely	65	63.7
Plants not meeting NPDES permit for biochemical oxygen demand (BOD) concentration	17	16.7
Plants not meeting NPDES permit for percentage BOD removed	12	11.8
Plants not meeting NPDES permit for suspended solids (SS) concentration	9	8.8
Plants not meeting NPDES permits for percentage SS removed	9	8.8
Plants not meeting NPDES permit for fecal coliform	8	7.8
Plants not meeting NPDES permit for nitrogen and/or phosphorous concentration	12	11.8
Plants that do not require NPDES permit*	15	12.0
Plant's performance needs upgrading	27	22.0

*No surface discharge of plant effluent.

Table 5

Treatment Plant Operation and Maintenance Requirements*

		Manhours/Week		Electricity**	Percent of Personnel Certified			
Plant Size (mgd)		Operation	Maintenance	kW/Month	Supervision	Operation	Lab Tech	Maintenance
0.001-0.01	Avg.	21.9	18.9	830	100	83.0	0	40.0
	Range	10-40	3-40	15-2,000	—	0-100	—	0-100
0.01-0.1		25.8	9.5	11 00	55.5	61.6	42.5	57.1
		2.5-64	2.5-30	0-2520	0-100	0-100	0-100	0-100
0.1-0.5		57.1	16.8	6,717	89.0	73.4	27.7	45.5
		5-238	0-120	0-36,000	0-100	0-100	0-100	0-100
0.5-1.0		86.2	24.0	16,476	83.3	57.1	66.6	0
		0-240	2-80	0-115,000	0-100	0-100	0-100	—
1.0-2.5		145.7	52.5	23,652	55.5	67.2	33.3	33.3
		23-340	0-200	0-71,000	0-100	0-100	0-100	0-100
2.5-5.0		155.8	73.7	207,000	70.0	70.0	66.6	75.0
		64-225	51-140	30-34,000	0-100	0-100	0-100	0-100
5.0-10.0		248.3	138	15,750	66.6	66.6	100	50.0
		175-305	74-215	1,000-45,000	0-100	0-100	000	0-100

* Both average value and a range of values are reported for each plant size. The wide range of values reported for manhours and electricity consumption indicate the various treatment processes used, with tertiary plants having a great demand on both.

**Besides the electrical rqeuirement, many installations require additional fuel sources, such as gasoline, propane, No. 2 fuel oil, and natural gas; however, the amounts of such fuel used are insignificant.

Table 6

Industrial Facilities

Industrial Facilities Using Physical-Chemical Treatment

Type of Facility	Number of Facilities	Percentage of Facilities
Depot	6	15.0
Ammunition manufacturing	22	55.0
Acid production	1	2.5
Arsenal	5	12.5
Armament research	1	2.5
Plating	1	2.5
Laboratory	3	7.5
Missile plant	1	2.5

considered, 31.7 percent of all plants have the problem of improperly trained personnel. Many plants have operators who service several sewage treatment plants and a water treatment plant on a split shift. The manhour/wk data from Table 5 illustrate the problem of insufficient manpower. Table 8 converts these data to man-shift data.

The figures in Table 8 show that the average Army facility is understaffed -- some critically so. Full attendance during the day and some attendance at night or on weekends is impossible for small plants. Laboratory work or efforts to monitor performance at the larger plants would be extremely limited, if possible at all. The statistics in Table 5 also reveal that a relatively high percentage (30 to 66 percent) of personnel at both small and larger plants have not been certified to do their jobs. A worse problem is that regardless of plant size, key personnel, such as the supervisor or the chief operator, are not certified. The training program appears to be falling behind the demand for more highly technical personnel required to meet State or Federal effluent standards.

Many other problems at 16 percent of the plants are equipment-related (Table 7). These, along with insufficient manpower and inadequate training, are the major pitfalls in the operation and maintenance of Army treatment facilities. Between 1979 and May 1982, some plants were upgraded, but many others still need improvement.

Table 7

Problems Reported by Plant
Personnel in Plant Operation and Maintenance*

Identified Problems	Number of Plants Reporting	Percentage
Insufficient personnel (manpower)	23	16.5%
Inadequate training of personnel	27	19.4
Inadequate training of personnel because of problems with industrial wastes	3	2.2
Inadequate funding for training	4	2.9
Lack of opportunity to upgrade operators**	5	3.6
Inability to get or keep trained personnel	5	3.6
Insufficient equipment	5	3.6
Obsolete equipment	3	2.2
Improper functioning of equipment	5	3.6
Lack of proper equipment maintenance program	3	2.2
Lack of laboratory facility	3	2.2
Equipment is inferior	3	2.2
Excessive infiltration of ground water into sewer line	2	1.4
None	48	34.5

*Many plants report more than one of the problems identified above.
**Training program in the region not available, or work schedule does not allow operators to attend longer training courses.

When selecting a treatment system either to replace a facility or to expand its capacity, plant managers should keep in mind that current personnel-related problems will remain in the immediate future. This suggests that a treatment alternative which is easy to operate and maintain will have an advantage over others if it is cost-effective and reliable.

A General Accounting Office report[1] indicates that, between 1978 and 1979, 84 percent of 242 municipal wastewater treatment plants violated their NPDES permits (31 percent were serious violations) and that 56 percent of the violating plants exceeded their discharge permit limits for more than half the year. Among the various reasons for plant noncompliance in 9 out of 15 cases were O&M deficiencies (insufficient or underqualified staff, inadequate budgets, and the lack of operator training programs). Unless some personnel become certified soon, the Army will have to choose a treatment system which requires less operation and maintenance effort, lower operator competence, and less control equipment. If these limitations occur, oxidation ditch technology would be an excellent alternative for Army installations.

Oxidation Ditch Technology

An oxidation ditch is an activated sludge biological treatment process commonly operated in an extended aeration mode, although conventional activated sludge operation is also possible. A continuous loop reactor provides a bio-reactor recirculating mixed liquor continuously. Horizontal velocity is supplied by a directionally controlled aeration and mixing device. Typical oxidation ditch treatment systems (Figure 1) consist of a single closed-loop channel, 4 to 6 ft deep, with 45-degree sloping side walls. However, there are many modifications to this configuration.

Some form of preliminary treatment, such as screening, comminution, or grit removal, normally precedes the oxidation ditch process; however, primary clarification is usually not done. Single or multiple mechanical aerators are mounted across the channel in a fixed or semi-fixed floating position. Horizontal brush, cage, or disc-type aerators, or vertical shaft aerators designed for oxidation ditch applications are normally used. Aerating jets

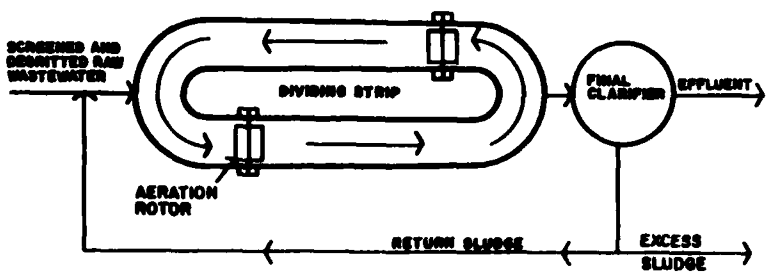

Figure 1. Typical oxidation ditch treatment system.

[1] Costly Wastewater Treatment Plants Fail to Perform as Expected (General Accounting Office, 1980).

Table 8

Manhours/Week Values

Plant size (mgd)	Total manhours/wk* (operation and maintenance)	Equivalent man-shift**
0.001-0.01	40.8	1.00
0.01-0.1	35.3	0.88
0.1-0.5	73.9	1.85
0.5-1.0	110.2	2.76
1.0-2.5	198.2	5.00
2.5-5.0	229.5	5.74
5.0-10.0	386.3	9.66

* Total manhours/wk is the sum of the average manhours/wk for operation and the average manhours/wk for maintenance for each plant size.
**Man-shift is 40 manhours/wk.

are also used. Figures 2 and 3 show a typical horizontal-shaft aerator and a vertical-shaft aerator, respectively. The aerators provide mixing and circulation in the ditch, as well as sufficient oxygen transfer. The process can provide BOD removal, as well as a high degree of nitrification without special modification because of the long detention time (normally 16 to 24 hours) and the long solids retention time (10 to 50 days). Secondary clarification is provided in a separate clarifier, although a new modification is being developed which uses intrachannel clarification. Denitrification can be incorporated into the design and operation of oxidation ditches by providing an anoxic zone in the channel, using an elaborate dissolved oxygen concentration control system. Raw wastewater is usually the carbon source for denitrification; thus, an external carbon source is not needed.

Ditches may be constructed of various materials, including concrete, gunite, asphalt, or impervious membranes. Concrete is the most common. The ditch loop may be oval or circular. "Ell" and "horseshoe" configurations have also been built to maximize land use. The following paragraphs describe the various types of oxidation ditch systems and their features.

Single-Loop Plants Using Horizontal-Rotor Aerators

Small plants (less than 0.25 mgd) use horizontal rotors of 27- to 30-in. diameters. These aerators are available from:

Lakeside Equipment Corp.
1022 E. Devon Avenue
Bartlett, IL 60103

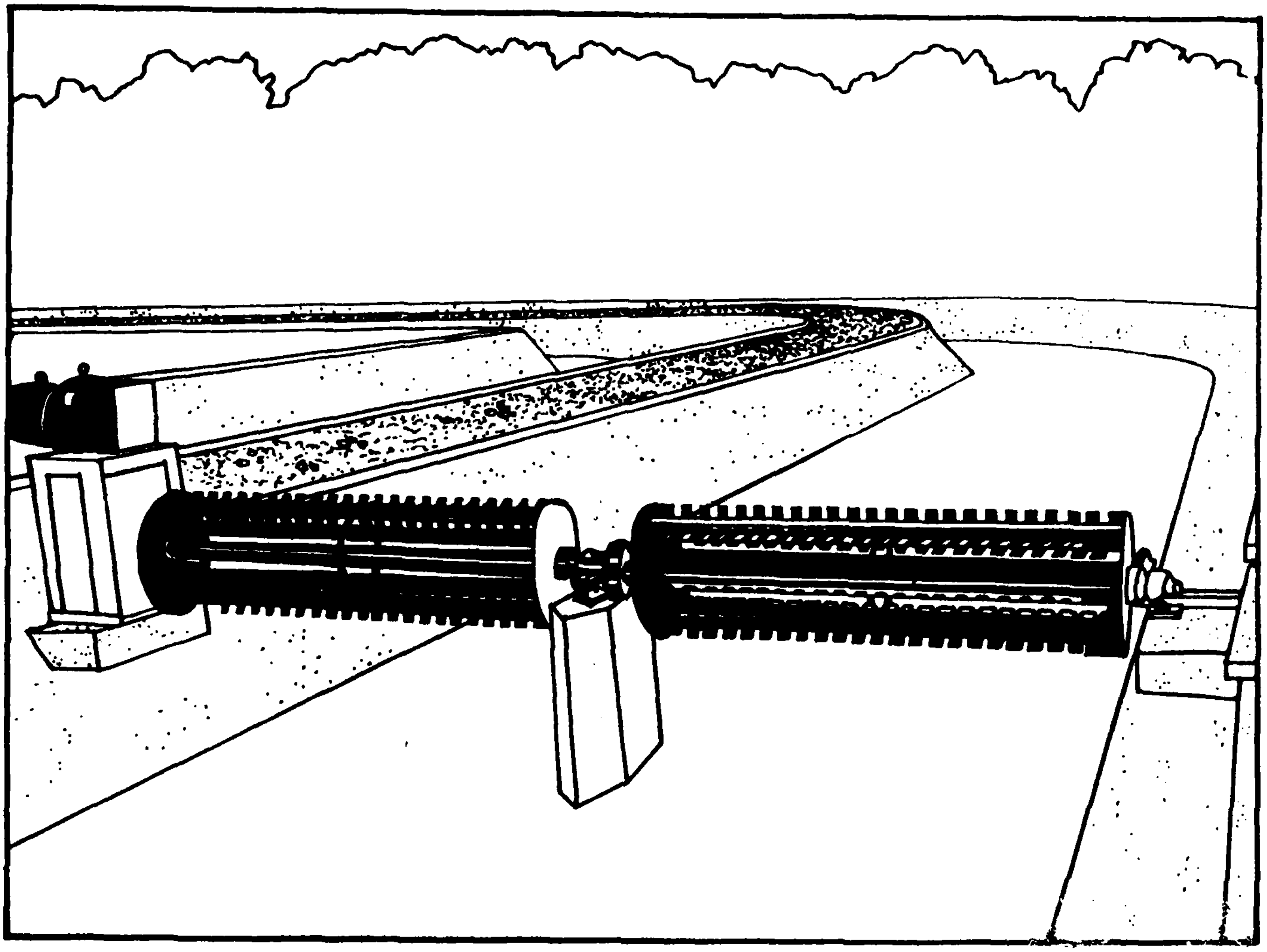

Figure 2. Typical horizontal-shaft aerator.

Passavant Corp.
P.O. Box 2503
Birmingham, AL 35201

Most small oxidation ditch plants use 45-degree slope side walls with an oval-shaped configuration. Larger plants (0.25 mgd or more) use horizontal brush-type rotors of 42-in. diameter or above. This equipment is available from the manufacturers listed above. The 45-degree slope side walls may be used in up to a 10-ft liquid depth. Vertical walls are used for deeper channels.

Carrousel Plants

These plants use a folded design of looped channels, as shown in Figure 4. Most plants using Carrousel systems are large. The smallest plant flow to which the Carrousel system can be adapted is 100,000 gal/day, but there is no upper limit on the size of Carrousels. Large, vertical-shaft aerators must be used for deep channels. These are slow-speed aerators placed at every other 180-degree turn of the channel. At the turning points not occupied by aerators, semicircular guide walls reduce turbulence losses in the channel. The Carrousel plants are available from:

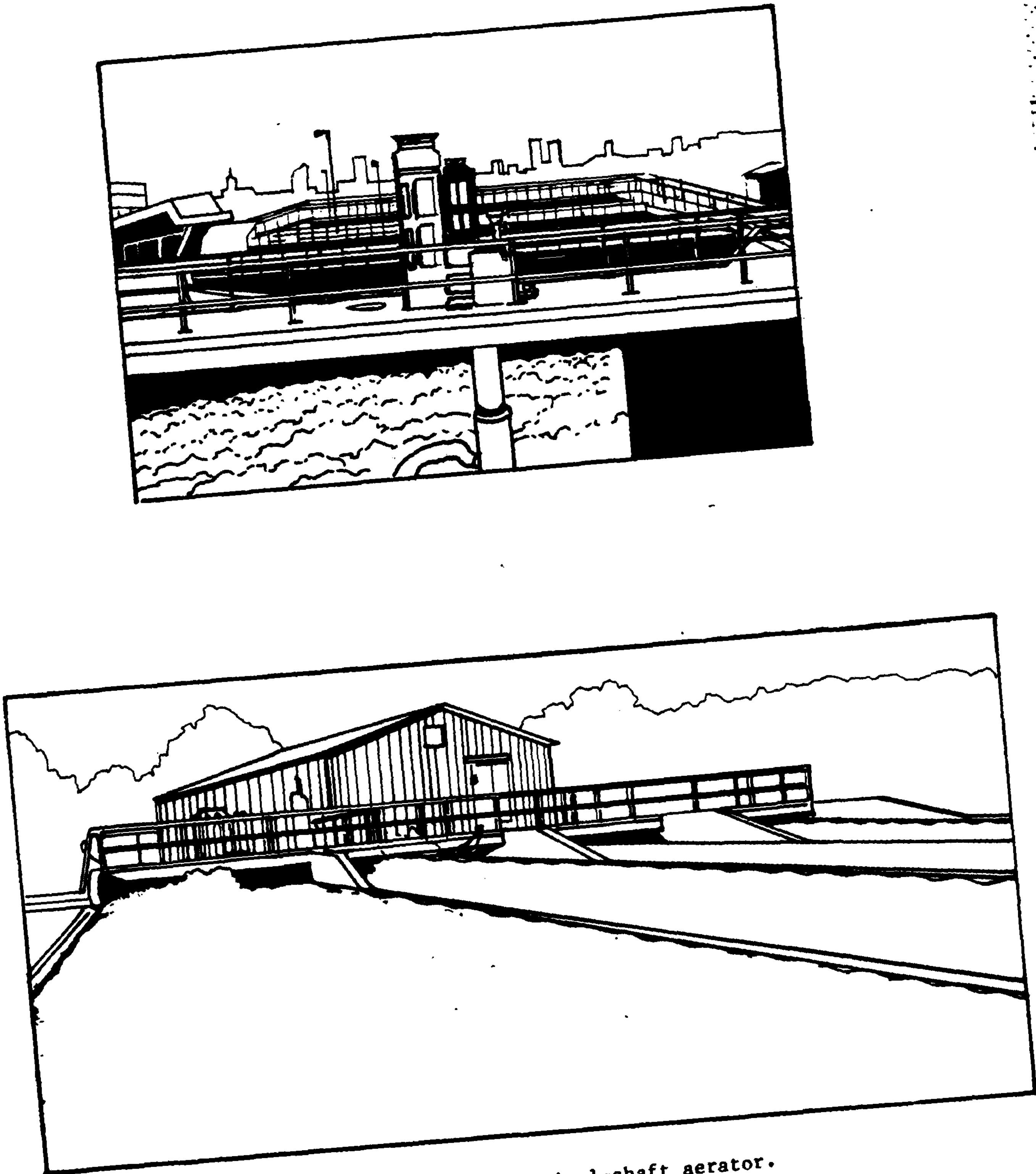

Figure 3. Typical vertical-shaft aerator.

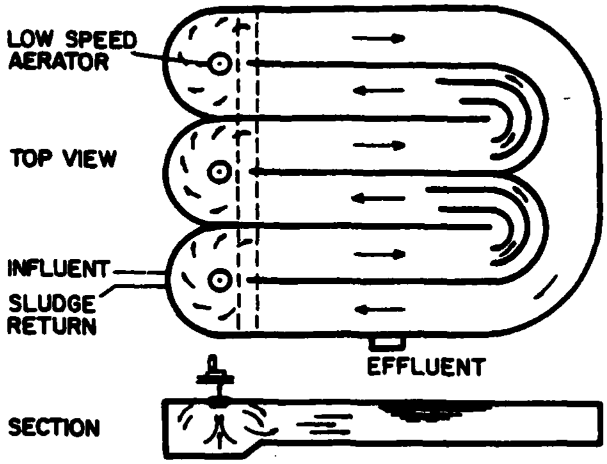

Figure 4. Typical Carrousel plant.

Envirotech Eimco Process Machinery Division
669 W. Second South
P.O. Box 300
Salt Lake City, UT 84110

Envirotech also has a dissolved oxygen (DO) control system which uses strategically located probes to determine the DO depletion rate in the channel; assuming BOD has been removed, this defines the precise location of the denitrification and nitrification zones.

Jet-Aerated Channels

In comparison with other looped channels, the jet aerator system can use very deep channels and is independent of channel width. Both a floating surface aspirator and a surface aerator of fixed design are available. The wastewater below the surface is pumped through a specially designed nozzle to aspirate air into the mixing chamber; from here, the mixture is discharged below the surface. As shown in Figure 5, the wastewater flow increases in velocity as it passes through the nozzle. Air is drawn into the low-pressure zone in the aspirator and mixes with the liquid, forcing oxygen to transfer into solution. The jet plume then discharges through the liquid into the ditch and mixes the oxygenated liquid in the plume with the bulk of the ditch liquid.

Figure 6 shows various models and capacities of the floating surface aspirator and the surface aspirator of fixed design. Use of submerged jets in deep channels avoids the adverse cooling effect of surface aerators. These aspirators are available from:

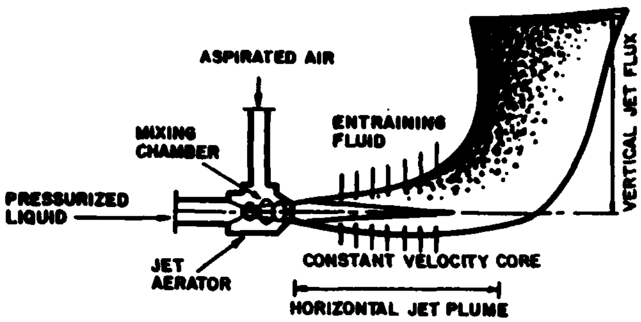

Figure 5. Horizontal jet plume action, providing mixing force along with extended high-pressure bubble residence time. (From Aspirating Aerators Technical Bulletin [Pentech Houdaille].)

Clevepak, Aerocleve Division
Fall River Industrial Park
1075 Airport Road
Fall River, MA 02720
(formerly Pentech Houdaille JAC Oxyditch, Iowa)

Orbal Plants

The design of orbal plants (Figure 7) is somewhat like the folding design of the looped channel in the Carrousel system, but is a concentric configuration. The number of aeration disks per channel decreases from channel to channel, proceeding inward. The maximum economic plant size is 6 mgd. The plant can be obtained from:

Envirex Inc. (A. Rexnord Co.)
1901 S. Prairie Ave.
Wanesa, WI 53186

In 1975, 34 orbal plants were in operation in the United States; half of these are in Texas. Presently, there are 70 in operation in the United States, ranging from 50,000 gpd to 9.6 mgd.

Influent waste progresses in series from the outside channels inward through submerged transfer ports, each channel being a complete mix reactor with an endless flow circuit. This arrangement permits complete mix and plug flow benefits. Different process modes may be used, depending on treatment goals.

Several additional closed loop reactor type systems have emerged recently using rotating diffusers or fixed diffusers with separate propulsion devices to create circulation. A short discussion and additional details on the previous modifications are available in a publication by Mandt and Bell.[2]

[2] M. G. Mandt and B. A. Bell, Oxidation Ditches in Wastewater Treatment (Ann Arbor Science, 1982).

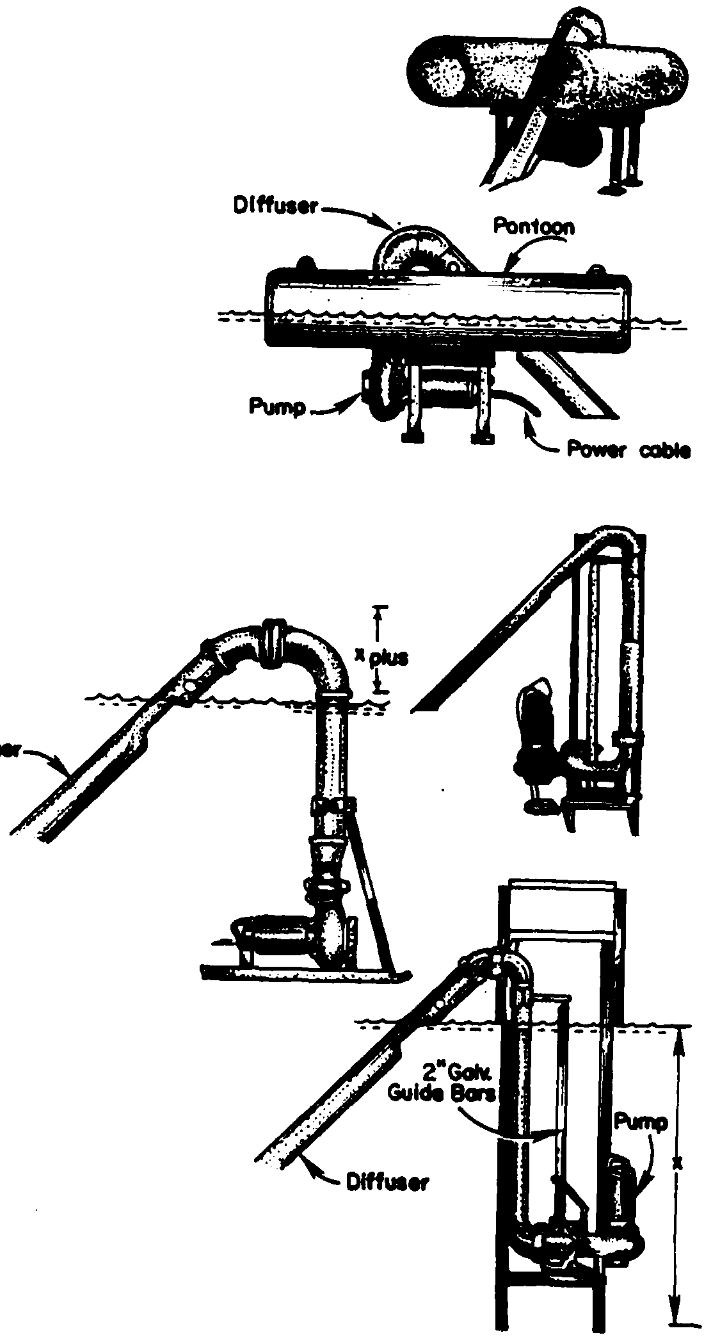

Figure 6. Various models of floating surface and fixed-design aspirators. (From Aspirating Aerators Technical Bulletin [Pentech Houdaille].)

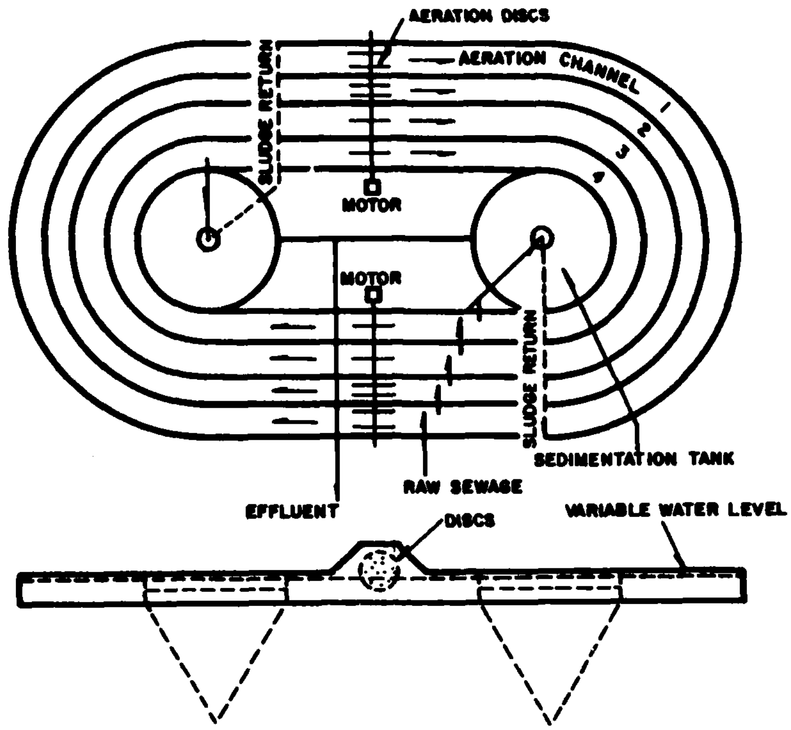

Figure 7. Diagram of orbal plant layout. (From R. J. L. C., et al, "The Orbal Extended Aeration Activated Sludge Plant." J.W.P.C.F., Vol 44. No. 2 (1972), pp 221-231.)

3 QUESTIONS AND ANSWERS ON OXIDATION DITCH TECHNOLOGY

Since the use of oxidation ditches is relatively new at Army installations, comparison of oxidation ditch technology with other available processes will raise many questions. This report attempts to provide answers to these questions. For quick reference, the following text lists the most commonly asked questions and provides short answers to them. References direct the reader to parts of this report which give more detailed answers.

Questions and Short Answers About the Application of Oxidation Ditch Technology for Army Installations

Question	Short Answer	Where Detailed Information Can Be Found in This Report
1. What is the treatment capability of an oxidation ditch process?	Oxidation ditch is a modified activated sludge biological treatment process operated in the extended aeration mode. It can be designed and operated for BOD removal, nitrification, and denitrification in the same channel. Therefore, the technology can apply to a new treatment plant (secondary or tertiary treatment) or can be used to retrofit existing treatment plants to meet BOD, suspended solids, and nitrification-denitrification requirements.	Chapter 2: Oxidation Ditch Technology Appendix A: Literature Review Chapter 5 TM 5-665
2. When should and should not oxidation ditch technology be chosen?	Conditions favorable for selecting oxidation ditch over other alternatives: 0.15 to 10.0 mgd capacity range (less costly within this range for comparable processes). High performance reliability (better than other competing biological treatment plants in winter and summer). Operator competence level need not be high (compared to other biological processes). Suitable for Army mobilization construction. Conditions unfavorable for selecting oxidation ditch:	Chapter 4: Guidelines for Oxidation Ditch Selection

Question	Short Answer	Where Detailed Information Can Be Found in This Report
2. Continued	Outside the 0.15 to 10.0 mgd capacity ranges. Regulating agency requires further treatment of oxidation ditch sludge before sand filtering. Limited land space available on the plant site.	
3. What are process advantages/disadvantages?	Advantages: Simple O&M. Higher performance reliability (summer and winter). Flexibility in design, BOD-nitrification-denitrification. Flow equalization provided. Lower operator competence requirement. Stabilized sludge eliminates the need for further sludge treatment (e.g., anaerobic digestion can be eliminated). Does not need primary clarifier. Few moving parts in system; therefore less subject to failure and conducive to easy repair. Suitable for Army mobilization construction. Disadvantages: Has larger land requirement than most biological processes except lagoon and land treatment. More expensive than RBC technology when plant capacity is less than 0.15 mgd. More expensive than most competing biological processes when plant capacity exceeds 10.0 mgd because of the extensive concrete work.	Chapter 4

Question	Short Answer	Where Detailed Information Can Be Found in This Report
4. What are the appropriate design criteria?	Q_{design} = 1.6 avg daily flow for small plants = 1.3 avg. daily flow for 0.5 to 1.0 mgd plants. = avg. daily flow for plants > 1.0 mgd. Hydraulic detention time: 18 to 24 hours. Mean sludge retention time: 20 to 30 days. Aerator size sufficient to meet both oxygenation and mixing requirements. Manufacturers provide charts or tables for selecting appropriate aerator size. Final clarifiers 450 gpd/sq ft; solid loading 15 lb/sq ft-day. Sludge drying beds -- 1 sq ft/population equivalent or 1 sq ft/0.17 lb BOD/day.	Chapter 5: Design Features and Guidelines
5. What are the system land requirements?	More land is required for the oxidation ditch process than other competing biological processes except lagoon and land treatment. For 1.0 mgd plant (ditch component only): Carrousel: 12,000 sq ft (10 ft deep) Dual oval channels: 20,000 sq ft (10 ft deep) Deep channel: 10,000 sq ft (20 ft deep) (including final clarifier)	Chapter 4: Guidelines for Oxidation Ditch Selection
6. How much does it cost?	System cost: $0.25 million/0.25 mgd. $0.33 million/0.5 mgd. $0.48 million/1.0 mgd. $1.8 million/5.0 mgd. $2.9 million/10.0 mgd. Includes nitrification-denitrification cost, but excludes chlorination, pretreatment, control panel,	Chapter 4: System and O&M Costs

Question	Short Answer	Where Detailed Information Can Be Found in This Report
6. Continued	and building costs. Less expensive than all competing biological processes.	
7. Is the oxidation ditch easy to install and start up?	Yes. Suitable for Army mobilization construction. Start-up is easy, but it takes time (weeks--as in any biological process using suspended cultures--to months) to build up the biological solids in the channel to the desirable high level (4000 to 8000 mg/L).	Chapter 4 TM 5-665
8. Can equipment be obtained and installed in a tight compliance schedule?	Not different from other competing biological processes. Easier for small facilities than for larger ones.	
9. What are the skill and manpower requirements?	Lower than other competing biological processes.	Chapter 4 Appendix A: EPA Report
10. What are the operational and maintenance problems?	Minor problems in brush, disc, and cage rotor aerators in the winter. Air lift sludge return pumps often have problems. Problems minimal compared to other competing biological processes.	Chapters 4 and 5: Operation and Maintenance TM 5-665
11. What are the limitations and restraints of system application?	Similar to all biological treatment processes; unable to remove toxic and non-biodegradable chemicals. Shock toxic load causes processes upset, but less often because of the built-in equalization capacity.	
12. Can an oxidation ditch remove phosphorus?	Oxidation ditch can be modified to biologically remove phosphorus using the Bardenpho process without adding chemicals. No full-scale operation in the U.S.	Chapter 4: Design Flexibility

Question	Short Answer	Where Detailed Information Can Be Found in This Report
13. What is the system's energy requirement?	Primarily for oxygenation and mixing: 4.5×10^4 kWh/yr for 0.1 mgd capacity. 4.5×10^5 kWh/yr for 1.0 mgd capacity. 5.0×10^6 kWh/yr for 10 mgd capacity. Nitrification requires an estimated additional 30 percent power consumption. More energy is needed than in activated sludge processes except for extended aeration. However, including nitrification, oxidation ditch requires less energy than activated sludge processes.	Chapter 5: EPA I&S Technology Assessment Manual
14. What are the clarifier design criteria?	No primary clarifier is required. Conservative design for final clarifiers: 450 gpd/sq ft for hydraulic loading. 15 lb/sq ft-day for solid loading.	Chapter 5
15. What are the new developments in oxidation ditch technology?	Intrachannel clarifier is being demonstrated in an EPA project which further reduces mechanical equipment, making the technology simpler. However, the sludge problem has to be overcome, and more full-scale demonstration is required.	Appendix A: Literature Review; Burns & McDonnell Process
16. What are the sludge characteristics, and what is the potential need for sludge treatment?	Stabilized sludge is obtained which is allowed in most plants, depending on responsible health authorities; or sludge can be applied directly on sand drying beds without applying aerobic or anaerobic treatment beforehand.	Chapter 4

Question	Short Answer	Where Detailed Information Can Be Found in This Report
17. What are the opinions of oxidation ditch operators?	Favorable. Most feel that an oxidation ditch is easier to operate and maintain than other competing biologoical processes.	Appendix A: Literature Review; and Oxidation Ditch Plant Visits
18. What is the life expectancy of major components?	The warranty period of the mechanical equipment, such as aerators and pumps, runs from 1 to 5 years. With a good service and maintenance program, all equipment should have a life expectancy longer than 20 years. With the exception of very small plants, duplicate aerators are installed for each channel so that breakdown of one aerator does not significantly affect or interrupt plant operation.	Manufacturers' Information
19. What are the O&M training requirements?	Hands-on operational training is preferred, with supervision by existing plant operators.	Chapter 5
20. Do any U.S. Army installations have an oxidation ditch plant?	There is one plant at Blue Grass Army Depot, Lexington, KY. Design flow = 17,000 gpd. It was put on line in early 1982.	Appendix A: Oxidation Ditch Plant Visits

4 APPLICABILITY OF OXIDATION DITCH TECHNOLOGY TO U.S. ARMY INSTALLATIONS

Many alternatives can be considered if an Army installation's wastewater treatment capacity needs upgrading beyond primary treatment or beyond some existing but outdated secondary treatment. Most of the Army's treatment facilities are trickling filter plants. A few installations use activated sludge, aerated lagoon, rotating biological contactor, and oxidation ditch processes.

This chapter discusses the applicability of the oxidation ditch technology to the treating of wastewater at Army installations. The operational and maintenance requirements, performance reliability, design flexibility, and cost are compared to other prevalent technologies, and guidelines for oxidation ditch selection are given.

Operation and Maintenance

The important operational requirements of an oxidation ditch plant are adequate aeration and mixing, adequate velocity in the ditch, and adequate mixed liquor suspended-solid concentration. Regardless of the type of aeration and mixing devices provided (disc or batch type for shallow channels, and jet or bubble aeration) an Environmental Protection Agency (EPA) study[3] indicates that all plants visited for the study have adequate aeration and mixing. In the disc type, the submergence of a disc or brush can be adjusted easily to provide needed aeration and mixing. In bubble or jet aeration, the amount of air supply controls the aeration and mixing.

Flow velocity in the ditch should be a minimum of 1.0 ft/sec. When aeration and mixing are adequate, more than the minimal flow velocity is observed in all plants. Proper mixing and flow velocity help keep the solids in suspension. However, many plant operations have been experimenting with the aeration-mixing device and found that the level of oxygenation and mixing can be reduced without adverse effect. One plant put the aerator on a 1-hour-on and 1-hour-off cycle and reported excellent results for more than a year. Another plant uses a 15-minute-on, 15-minute-off aeration cycle, except on extremely cold winter nights, with excellent results. Using an automatic timer control with this operation helps reduce energy cost. It is important to note that during the off cycle, the flow velocity in the ditch drops significantly below 1.0 ft/sec. When the aerator is turned on again, all settled solids are resuspended. Many plants are considering this scheme to lower operating costs.

As with all other biological treatment processes, start-up is slow because it takes considerable time to build up the biomass. Enough biomass is required to provide and maintain a steady treatment performance which meets the specified effluent quality standards. To shorten the startup period, no biological sludge should be wasted.

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[3] W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (U.S. Environmental Protection Agency, 1978).

Most engineers recommend maintaining a mixed liquor suspended solid (MLSS) level of about 4000 to 8000 mg/L in the oxidation ditch. The level is high because the system is designed for a low food-to-biomass ratio (F/M). Since a wide range of MLSS does not affect the performance, the operator does not have to pay close attention to MLSS control. Incoming BOD is substantially diluted, because of the large liquid volume of the ditch and because the operation is in the complete-mixing mode. This low BOD load and the high MLSS level in the ditch guarantee the low F/M operational requirements with minimal operator effort.

The ditch's liquid volume is the result of the process design. Most oxidation ditch plants are designed for a 24-hour hydraulic detention time. The large liquid volume provides the dilution capacity needed for organic load equalization. This helps explain the relatively steady performance of the oxidation ditch. Simple operational control and reliable performance (see page 61) are important considerations in selecting treatment alternatives for Army installations.

Maintaining a high MLSS concentration in the ditch requires very little sludge wastage. Also, the sludge is 20 to 30 days old, with a high degree of stabilization equivalent to that of a well-stabilized aerobically digested sludge. The waste sludge usually receives no further treatment except dewatering before final disposal. This greatly minimizes operation and maintenance requirements.

An oxidation ditch has 95 to 99 percent nitrification capability without design modifications and without any extra operation and maintenance requirements. The high degree of nitrification, even at wastewater temperatures approaching $0^{\circ}C$ as reported by many oxidation plants, may be due to the 24-hour hydraulic detention time in the ditch and the long sludge age of 20 to more than 50 days.

The operation and maintenance requirements of an oxidation ditch plant are further reduced by eliminating the primary clarifier. However, screening or comminution should be regularly and properly carried out. Otherwise, rags and debris tend to cause problems in the plant, especially in the return sludge system.

Like any treatment plant with mechanical equipment, oxidation ditch plant equipment is subject to problems, malfunctions, and failures. The EPA study shows oxidation ditch plants which operate without mechanical problems for long periods of time and which operate with a very high mean time between equipment failures; this is particularly true in plants using proper preventive maintenance and inspection programs. Mechanical and other problems reported by many plants include:

1. Drives trip out on electrical failures and do not restart after power is restored.

2. Air lift pumps used for sludge return fail more often than centrifugal, non-clog pumps.

3. Comminutors are a continuing maintenance problem.

4. Aerator spray under various wind conditions causes slipperiness, due to algal growths and freezing access walks.

5. Aerators and aerator drives account for a major portion of the mechanical problems. Most plants have the following aerator problems every 2 to 5 years per unit:

a. Loss of disc or some "teeth" from brush-type aerators due to corrosion of bolts or damage by ice jam in very cold weather.

b. Bearing problems in gear drives, line shafts, and aerator shafts.

c. Problems with flexible coupling between line shafts.

d. Gear reducer failure and reducer output shaft seals breakage.

e. Aerator torque tube failure or excessive deflection with very long aerators.

f. Need to lift the aerator out of position to remove the gear drive in some drive configurations.

These mechanical or mechanical-related problems are not excessive in comparison to those of other biological treatment plants of equivalent sizes. In fact, most oxidation ditch plants do not have a primary clarifier and sludge treatment, which greatly reduces mechanical and related problems. Most plant operators agree that oxidation ditch plants are easy to keep in service and operate for long periods of time with very little operation and maintenance. Many plants can be operated unattended for periods of time (evenings and weekends) without significant problems.

The lack of skilled treatment plant operators is a universal problem, which may be even worse at Army installations. Thus, the oxidation ditch technology should be successful in Army applications because of its simple operation and maintenance. However, this process still requires properly trained operators. A small oxidation ditch plant may be run by a minimally certified operator, while a larger plant should be operated by a higher-level certified operator. Treatment plant operator certification is based on the population served by the plant; at a larger plant, operators are classified with extended aeration activated sludge plant operators. Many oxidation ditch plants are manned only during a single shift and often for only a portion of a single shift. However, some attention during weekends is recommended.

A 1977 EPA study[4] concluded that performance difficulties at oxidation ditch plants, activated sludge plants, package plants, trickling filter plants, and three-cell lagoons are caused by two main problems: (1) generally low-skilled operators and inadequate time to optimize facility operation, and (2) failure of design to anticipate industrial loads, plus poor monitoring and in-plant handling of industrial wastewaters. Oxidation ditches seem to be the

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[4] Efficient Treatment of Small Municipal Flows at Dawson, Minnesota, EPA Technology Transfer, Capsule Report 625/2-77-015 (U.S. Environmental Protection Agency, 1977).

type of facility least affected by lack of operator competence. This should appeal to Army engineers responsible for selecting treatment technology.

Reliability in Performance

There are more than 90 municipal oxidation plants in the United States, but only one is known to be operational at an Army installation (Blue Grass Army Depot, Lexington, KY). (See Appendix A.) Performance and reliability data have been developed from actual operating records, the literature, visits and telephone or letter contact with operating plants, EPA records, and engineers' files. The best source of performance data is the EPA report, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes.[5] The performance data from 29 oxidation ditch plants for various U.S. and Canadian locations presented in this report are summarized in Table 9.

Table 10 summarizes the performance of competing biological treatment processes, and Table 11 shows the results of an analysis of 12 operating oxidation ditch plants which showed their reliability for meeting various BOD_5 and total suspended solids (TSS) effluent standards.

Of the plants analyzed in Table 11, the effluent BOD_5 and TSS seldom exceeded a maximum of 60 mg/L. The reliability of competing biological treatment processes was evaluated on the same basis and is summarized in Table 12.

The data represent monthly average performance of the oxidation ditch plants. Winter is arbitrarily determined to be the months of November through March. The data show that as a group, oxidation ditch plants perform consistently well in many cases, despite limited operation and maintenance.

The average BOD_5 and TSS concentrations of the effluent are consistently lower than those of effluents from other biological processes. This is true even when the winter performance of oxidation ditch plants is compared with other processes. This has been confirmed by other studies (see pp 130-139). For example: the Dawson plant, MN (average BOD_5 3 mg/L, suspended solids 8 mg/L); the Winchester plant, NH (99.4 percent BOD removal); the Ash Vale plant and the Cinencester plant, England (4.0 mg/L BOD, 3.5 mg/L SS); the McAdam plant, New Brunswick (BOD <10 mg/L, 27 mg/L SS); the St. Charles plant, MN (BOD 2 to 8 mg/L and SS 2 to 10 mg/L); the West Plains plant, MO (BOD 3 mg/L, SS 1 mg/L); the Little Blue Valley plant, MO (demonstration project with EPA -- BOD 10 to 25 mg/L, SS 5 to 30 mg/L); and the Stowe plant, VT (filtered BOD 5.6 mg/L, SS 2 mg/L). The record is impressive, particularly since many of these plants are operated in very severe winter conditions.

The long hydraulic detention time and the high MLSS concentration in oxidation ditch plants provide the necessary equalization capacity to dilute shock loadings and to assume an effluent with a low BOD concentration. In this regard, the performance should be similar to that of an extended aeration process. In fact, the data show that BOD removal percentage and effluent BOD

[5] W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (U.S. Environmental Protection Agency, 1978).

Table 9

Summary Performance of 29 Oxidation Ditch Plants
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	Effluent, mg/L			Removal		
	Winter	Summer	Average Annual	Winter	Summer	Average Annual
BOD_5						
High Plant	55	34	41	87	86	87
Average	15.2	11.2	12.3	92	94	93
Low Plant	1.9	1.0	1.5	99	99	99
Suspended Solids						
High Plant	26.6	19.4	22.4	81	82	82
Average	13.6	9.3	10.5	93	94	94
Low Plant	3.1	1.9	2.4	98	98	98

Table 10

Performance - Competing Biological Processes
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	Effluent, mg/L		Removal, %	
	TSS	BOD_5	TSS	BOD_5
Activated Sludge (1.0 mgd)	31	26	81	84
Activated Sludge (Package Plants)	28	18	--	--
Trickling Filters	26	42	82	79
Rotating Biological Contactor	23	25	79	78

Table 11

Reliability - Oxidation Ditch Plants
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	% of Time Effluent Concentration (mg/L) Less Than					
	10 mg/L		20 mg/L		30 mg/L	
	TSS	BOD_5	TSS	BOD_5	TSS	BOD_5
Best Plant	99	99	99	99	99	99
Average All Plants	65	65	85	90	94	96
Worst Plant	25	20	55	55	80	72

Table 12

Average Reliability - Competing Biological Processes
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Seconday and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	% of Time Effluent Concentration (mg/L) Less Than					
	10 mg/L		20 mg/L		30 mg/L	
	TSS	BOD_5	TSS	BOD_5	TSS	BOD_5
Activated Sludge (1.0 mgd)	40	25	75	70	90	85
Activated Sludge (Package Plants)	15	39	35	65	50	80
Trickling Filters	---	2	---	3	---	15
Rotating Biological Contactor	22	30	45	60	70	90

concentration are similar for oxidation ditch plant and package treatment plant performance. Oxidation ditch plants seem to carry fewer suspended solids in their effluents.

Since it is important to provide a final clarifier with low overflow rates, most oxidation ditch plants are equipped with relatively large clarifiers with an overflow rate of 400 to 600 gpd/sq ft at the design average daily flow (e.g., Dawson plant, MN; West Plains plant, MO; Stowe plant, VT; Winchester plant, NH; Huntsville plant, TX; Blue Grass Army Depot plant, KY, etc). This overflow rate is lower than the traditional 400 to 800 gpd/sq ft value for final clarifiers for conventional activated sludge plants and results in lower effluent SS concentrations from oxidation ditch plants.

The 1978 EPA report also reveals that oxidation ditch plants can provide 95 to 99 percent nitrification without design modifications. This is possible even at wastewater temperatures approaching 0°C due to the 24-hour hydraulic detention time in the ditch and the capability of operating at a high MLSS level or a long sludge age (up to 50 days). The infrequent sludge wasting helps keep enough nitrifying organisms in the system.

Properly designed and well-operated oxidation ditch plants also effect nitrogen removal by single-stage biological nitrification/denitrification. Denitrification occurs when an anoxic zone is produced downstream from the aerobic zone. The raw sewage, which is fed into the channel upstream of the anoxic zone, is the carbon source for the denitrification or anoxic zone. Eighty percent nitrogen removal has been achieved in one channel oxidation ditch plant; however, more skillful, careful operation is required to achieve this percentage.

In summary, oxidation ditch plants provide more reliable treatment performance than other biological treatment processes, but the operator skills they require are less demanding. Most oxidation ditch plants easily achieve low BOD and SS effluents and full nitrification. Although the performance data were collected primarily from municipal oxidation ditch plants receiving residential, commercial, and some industrial wastes, the basic nature of these wastewaters was not different from those of Army installations. Army installations report 85 to 98 percent residential flow and 1 to 15 percent industrial flow. Influent BOD varies from 173 to 320 mg/L (with some occasional lows in the 20s and 30s mg/L), and SS concentration varies from 149 to 280 mg/L (with occasional lows in the 20s and 40s mg/L). Waste characteristics are equivalent to those of a medium-strength municipal wastewater. Thus, the treatment capability and performance of oxidation ditch plants, as applied to municipal wastewater, should also apply to Army installation wastewater.

Design Flexibility

Wastewater treatment plant design is site-specific. When a design engineer selects a certain treatment technology, he/she must consider the following factors, since they are likely to vary among sites:

1. National Pollutant Discharge Elimination System (NPDES) permit
2. Flow characteristics (hydraulic and organic fluctuations)
3. Availability of trained operators
4. Weather
5. Land availability
6. Sludge treatment and disposal
7. Clarifier requirement
8. Army mobilization construction.

NPDES Permit

The NPDES permit specified for an Army installation treatment facility with surface discharge may require secondary treatment effluent standards. Sometimes tertiary treatment may be required if a very high quality of receiving water must be preserved. The oxidation ditch is a proven technology which meets secondary treatment standards easily. Its performance reliability is better than other biological processes. Also, nearly total nitrification can be obtained without system modification. If nitrogen removal is required, a special control system (e.g., Envirotech's Sparling Control and Analog Sensing Device) can be added to include aerobic-anoxic zones in the same channel for denitrification. Raw sewage instead of methanol is the carbon source for denitrification. The Envirotech controller identifies and regulates the aerator power draw so that adequate DO is maintained in the flow and assures BOD reduction and nitrification. It also insures that the anoxic channel length is sufficient for denitrification. Figure 8 is a schematic diagram of such a control system. The system can consistently remove 40 to 70 percent of the nitrogen without an external carbon source.

The same channel in the oxidation ditch plant can be designed for BOD removal, BOD removal plus nitrification, or BOD removal plus nitrification-denitrification. This offers a great deal of design flexibility, since little to no system modification will be required -- a unique feature that no other competing biological process can match.

The principle and application of inducing aerobic and anoxic zones in the same channel for nitrogen removal can be carried one step further for phosphorus removal, and is called the Bardenpho process (see Figure 9). The anoxic-aerobic sequence in the first two stages provides the condition for a luxury phosphorus uptake in the aerobic zone, resulting in an effluent phosphorus of less than 1.0 mg/L. However, there is not yet a full-scale oxidation ditch plant in the United States using the Bardenpho process.

Flow Characteristics

Flow fluctuation and organic load fluctuation are to be expected in Army installations. Usually a wider fluctuation is expected in smaller treatment facilities. An oxidation ditch plant provides a large equalization capacity which significantly mitigates all load fluctuations. Consequently, the system can be designed for both small and large installations with less concern for load variations.

Many oxidation ditch plants use a much shorter hydraulic detention time. Such a plant -- called a high-rate oxidation ditch process -- provides only 6 to 8 hours hydraulic detention. As a result, the great equalization capacity and other advantages associated with a longer detention time no longer exist; i.e., fluctuating hydraulic loads or organic loads will periodically produce inferior effluents. Since an oxidation ditch plant usually has no primary clarifier, this fluctuating effluent quality will occur more often than in other competing biological processes equipped with primary clarifiers. Also, the short hydraulic detention time will not allow nitrification to occur as a single-stage unit. Thus, the single channel cannot provide nitrification-denitrification treatment. The sludge quality will also be different (see (pp 44, 45).

In summary, the flexibility of choosing various hydraulic detention times for oxidation ditch design is less than it seems. Many of its advantages are eliminated when the detention time is shortened; this could make the total treatment system (including nitrification, sludge treatment, and disposal) less cost-effective.

Availability of Trained Operators

Skilled and highly trained operators are required for most biological processes which provide secondary or tertiary treatment. This could become a factor in selecting a treatment technology and its design. The oxidation ditch technology has a distinct advantage over others in that the same effluent quality can be obtained with higher performance reliability and a lower level of operator competence.

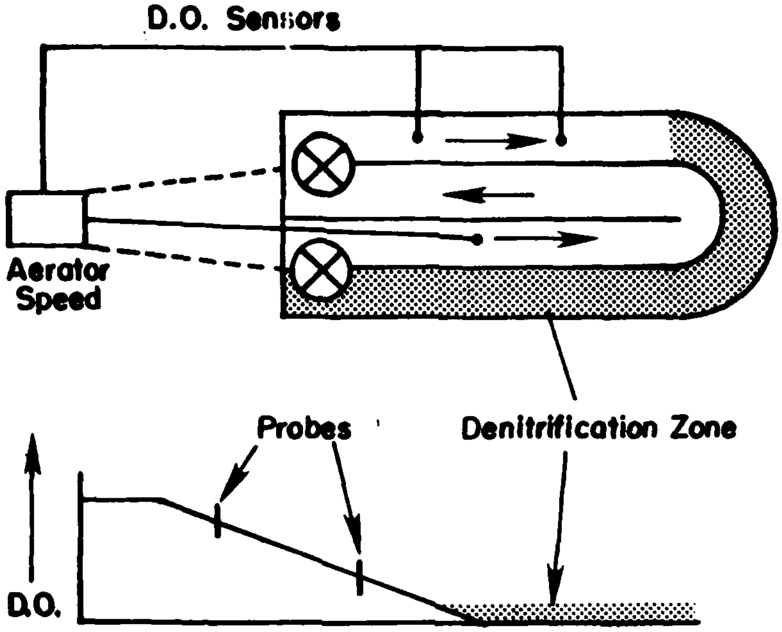


Figure 8. Schematic of aerator power draw controller. (From Envirotech Bulletin [Envirotech Corp.].)

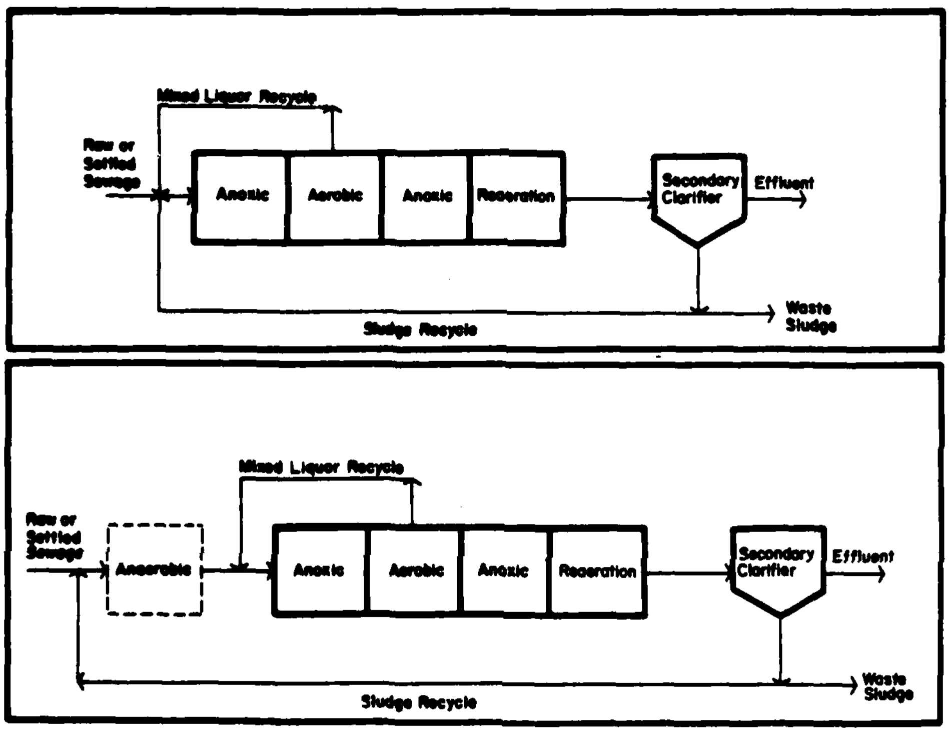


Figure 9. Bardenpho process. (From Envirotech Bulletin [Envirotech Corp.].)

Weather

Many biological treatment processes experience problems during very cold weather and need special protection (e.g., land treatment, lagoons, trickling filters, and rotating biological contactors). The nitrification process in particular is susceptible to cold weather. The EPA's Region VII study and its 1978 study demonstrate that BOD, SS removal, and nitrification are nearly done even when the wastewater temperature is near $0^{o}C$. The winter performance of oxidation ditch plants is better than that of other biological treatment plants in Iowa, Missouri, Kansas, and Nebraska. The Dawson plant in Minnesota also reports excellent nitrification and nitrogen removal (0.1 mg/L NH_3-N concentration in the effluent and 90 percent nitrogen removal, even in winter conditions).

It appears that oxidation ditch technology can be applied anywhere in the United States, including Alaska, without adverse weather effects on the treatment performance, including nitrification/denitrification.

Land Availability

Land availability for wastewater treatment plants is not limited at most Army installations, either domestic or overseas. (However, this does not exclude a few where land availability is critical.) Since an oxidation ditch is normally designed for 24 hours hydraulic detention, it occupies considerable land area. Although the detention time can be reduced for a high-rate oxidation ditch process, which can reduce the land area requirement, the disadvantages of a high-rate oxidation ditch process can outweigh the advantages of reducing the land requirement. If separate nitrification, denitrification, and sludge treatment systems have to be built, the total area requirement may actually have to be increased rather than decreased.

One approach to reducing the land requirement is to use a deep channel, although poor mixing can occur, leading to accumulation of solids in some "dead spots." However, many plants with deep channels report good mixing as well as oxygenation efficiency equal to shallow channels. For example, the St. Charles plant in Minnesota reports the aeration system (Pentech JAC Oxyditch), which uses jets and compressed air, achieves an oxygenation capacity of 5 lb O_2/brake hp-hr in a deep channel. Most rotor aerator systems are reported to achieve 3.0 to 3.5 lb O_2/hp-hr oxygenation in shallow channels. Thus, it appears that deep-channel design can solve the problem of limited land availability. The only major problem with deep channels is that such construction is not suitable in areas where the ground water table is relatively high.

Sludge Treatment and Disposal

Any biological plant design must consider the quantity of sludge produced, its nature and stability, and suitable disposal procedures. An oxidation ditch plant normally designed to operate in the extended aeration mode has certain inherent advantages for sludge handling and disposal. Oxidation ditch plants can be operated at a 20- to 30-day sludge age; this produces a sludge with characteristics similar to well-stabilized, aerobically digested sludge.

Conventional activated sludge plants operated at a 5- to 8-day sludge age, and trickling filters and rotating biological contactors will produce a sludge with high volatile (organic) content. This sludge will become odorous and objectionable if placed on drying beds or on the land. Aerobic digestion of this sludge for 1 or 2 weeks is normally required to produce a stable product suitable for disposal on drying beds or on land.

The oxidation ditch operated in the extended aeration mode provides sludge stabilization equivalent to conventional activated sludge plus aerobic digestion. The sludge is wasted directly to open drying beds. Often sludge is wasted directly to tank trucks which spread the liquid sludge on the plant grounds or adjacent land. Field inspections consistently reveal no odor problems.

The disposal of sludge on drying beds or on land eliminates the sludge treatment process. This should reduce both capital and O&M costs. Sixty-four Army plants (see Chapter 2) use the anaerobic digestion process for sludge treatment. There are often problems in operating these digestors, and use of the oxidation ditch can eliminate them.

Some oxidation ditch plants are designed as a high-rate process with 6 to 8 hours of hydraulic detention time. Such a process generates much more sludge with a much higher volatile (organic) fraction; this sludge will require aerobic digestion or other suitable treatment before it can be disposed of on drying beds or on land. The trade-off is obvious. For simplicity of operation and for more reliable performance, oxidation ditches should be designed and operated in the extended aeration mode.

Some design engineers and regulatory authorities may require some stabilization step for sludge generated from extended aeration. Part of the consideration may be the increase of flow in the future; this could force the plant to use a shorter sludge detention time so that more sludge stabilization will be required. However, this will not be absolutely necessary if expansion of the plant capacity is anticipated to meet the demand.

Clarifier Requirement

Traditional design of oxidation ditch plants eliminates the primary clarifier from the treatment system. This makes the operation and maintenance of a screen and/or a comminutor a very important pre-treatment step. If rags, boards, straws, or similar objects get into the ditch, they will cause trouble with the aerators and plug sludge control valves, pumps, and weirs. There is very little to be gained by continually using an existing primary clarifier (e.g., oxidation ditch added to an existing plant with primary clarifiers). Terminating the use of the primary clarifier(s) will reduce O&M efforts and simplify the sludge-handling process.

A final clarifier is essential for an oxidation ditch plant, and the overflow rate should be low. Most oxidation ditch plants produce effluents with lower SS than other biological treatment plants because the final clarifier design is more conservative. An extended aeration mode usually produces small and lighter flocs which do not have a superior settling characteristic; the mixed liquor entering the clarifier carries a high solids concentration which creates a high solid load. Thus, a proper overflow rate for the design should be between 400 to 600 gpd/sq ft at the design daily average flow.

Army Mobilization Construction

Constructing a wastewater treatment plant and getting it on-line in the shortest possible time is critical in Army mobilization. This can be done with a shallow oxidation ditch or an aerated lagoon better than with any other competing biological treatment process.

Shallow channels with sloped side walls can be built using concrete pour against earth backing, with welded, wire-mesh reinforcing. For example, the shallow oxidation ditch plant at the Blue Grass Army Depot, KY, uses only 3-in.-thick shotcrete with welded, galvanized-wire mesh. No foundation work and no reinforced concrete work are required. A minimal amount of mechanical equipment is required. In full mobilization, such an oxidation ditch plant can be built and put on-line in 3 weeks if all construction and plant equipment are on hand.

System and O&M Costs

Cost-effectiveness is important in selecting a wastewater treatment technology. This section compares the cost of oxidation ditch plants with other competing biological processes.

The most extensive figures are provided by the 1978 EPA report, which compares construction and annual O&M costs for oxidation ditch plants of various sizes and for competing activated sludge processes (extended aeration, contact stabilization, and conventional) for secondary and tertiary treatment. This information has been supplemented by more recent cost data. Table 13 gives the 1978 EPA data.

Construction and annual O&M costs were developed for competing activated sludge processes. These costs were developed on the same basis as the costs for oxidation ditches. Tables 14 and 15 give these costs. Construction and annual O&M costs were developed for competing biological nitrification processes. These are summarized in Tables 16 and 17 for 20 mg/L influent NH_4-N. Construction and annual O&M costs were developed for competing biological denitrification processes. These are summarized in Tables 18 and 19 for 20 mg/L influent NH_4-N. Construction and annual O&M costs were also developed for selected physical-chemical nitrogen removal processes. These are summarized in Tables 20 and 21 for 20 mg/L influent NH_4-N.

Construction costs for competing extended aeration and contact stabilization plants are less than for oxidation ditch plants in the flow range of 0.01 mgd to 2 mgd. However, oxidation ditch plant construction costs are less than for conventional activated sludge plants within the range of 0.01 to 10 mgd. Operation and maintenance costs for oxidation ditch plants are less than for the competing processes in the 0.1 mgd to 2 mgd range, and the total annual costs for oxidation ditch plants are less than for all other competing processes in the range of 0.1 to 10 mgd. Within the flow range of 0.01 to 0.1 mgd, the total annual costs for extended aeration package plants are less than for oxidation ditch plants. The total annual costs for all competing denitrification processes are higher than for oxidation ditch plants. When all

Table 13

Oxidation Ditch Plant Construction and Annual O&M Costs
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Plant capacity, mgd	0.1	1.0	10.0
Construction	195	600	3350
O&M normal	22.1	62.4	446.6
O&M nitrification*	22.6	63.1	467.5
O&M N-removal* (nitrification-denitrification)	28.1	67.4	453.7

*There are generally no increased construction costs for nitrification or nitrogen removal.

Table 14

Construction Costs - Competing Activated Sludge Processes
(BOD and SS Removal Only)
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976			
Capacity, mgd	0.5	1.0	5.0	10.0
Extended aeration (package plants)	390	-	-	-
Contact stabilization (package plants)	320	475	-	-
Conventional activated sludge	-	1045	2645	4138

Table 15

Annual O&M Costs - Competing Activated Sludge Processes
(BOD and SS Removal Only)
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976			
Capacity, mgd	0.5	1.0	5.0	10.0
Extended aeration (package plants)	64.4	-	-	-
Contact stabilization (package plants)	57.3	93.9	-	-
Conventional activated sludge	-	80.9	187.7	308.1

Table 16

Construction Costs for Competing Biological Nitrification Processes (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Capacity, mgd	1	5	10
Activated sludge, 1-stage (20 mg/L NH_4-N)	1,210	3,203	5,107
Activated sludge, 2-stage (20 mg/L NH_4-N)	1,448	3,830	6,031

Table 17

Annual O&M Costs for Competing Biological Nitrification Processes (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Capacity, mgd	1	5	10
Activated sludge, 1-stage (20 mg/L NH_4-N)	89.4	219.3	375.5
Activated sludge, 2-stage (20 mg/L NH_4-N)	102.9	245.4	416.9

Table 18

Construction Costs for Competing Biological Nitrification Processes (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Capacity, mgd	1	5	10
Activated sludge, 1-stage (20 mg/L NH_4-N)	539	1,357	2,291
Activated sludge, 2-stage (20 mg/L NH_4-N)	636	1,192	2,298

Table 19

Annual O&M Costs for Competing Biological Nitrification Processes
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Capacity, mgd	1	5	10
Activated sludge, 1-stage (20 mg/L NH_4-N)	54.1	140.6	244.6
Activated sludge, 2-stage (20 mg/L NH_4-N)	51.3	115.3	195.0

Table 20

Construction Costs for Competing Physical-Chemical Nitrogen Removal Processes
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Capacity, mgd	1	5	10
Breakpoint chlorination (20 mg/L NH_4-N)	114.8	377.1	696.7
Selective ion exchange (20 mg/L NH_4-N)	442.6	1,557.4	2,704.9
Ammonia stripping (20 mg/L NH_4-N)	245.9	1,065.6	1,967.2

Table 21

Annual O&M Costs for Competing Physical-Chemical Nitrogen Removal Processes
(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

	$1000, 1976		
Capacity, mgd	1	5	10
Breakpoint chlorination (20 mg/L NH_4-N)	100.0	280.0	380.0
Selective ion exchange (20 mg/L NH_4-N)	47.0	150.0	250.0
Ammonia stripping (20 mg/L NH_4-N)	18.0	57.0	170.0

factors are considered, the total annual costs for oxidation ditch plants are less than for all competing biological activated sludge processes. Thus, oxidation ditch plants are superior to them. Figures 10 through 15 illustrate the cost differences.

Although these cost data reflect 1976 dollar values, oxidation ditches should still be more cost-effective than activated sludge processes; however, the cost differential must be adjusted. The exception is for a plant size of 0.01 to 0.1 mgd, where the package activated sludge treatment plants have a smaller total annual cost.

Some more recent oxidation ditch cost data provide very detailed estimates close to bid price of construction. These data are only for the biological treatment portion of the liquid train and do not consider the costs associated with sludge treatment, dewatering, and final disposal. Buildings, chlorination, effluent discharge, preliminary treatment (screening and/or comminutor), engineering fees, etc. are not included. Table 22 summarizes these data. It is assumed that the hydraulic detention time of these plants is enough for BOD removal and for nearly complete nitrification. The overflow rates of the final clarifiers are adequate for acceptable effluent SS concentrations. The sludge generated needs neither stabilization nor other forms of treatment before disposal on drying beds or on land. The only exception is the Burns & McDonnell process with intrachannel clarifier design. The large volumes of wasted sludge from this process will require further settling to obtain an acceptable plant effluent, perhaps in a storage lagoon. The cost estimates given in Table 22 do not include provision for this. Therefore, these costs are slightly underestimated.

Table 23 and Figure 16 compare the costs in Table 22 with those of competing biological processes. The oxidation ditch costs have been adjusted to January 1982 dollar values, using the Engineering Record News (ERN) Construction Cost Index of 372.55 (Base Year 1973 = 100). Generally, no increased construction costs are considered for denitrification. Conventional cost data for the competing processes for the liquid train and biological treatment portion, up to and including the final clarifiers only, are taken from EPA data; proper adjustments have been made for changing the system components to satisfy BOD removal, nitrification, and denitrification.

While the O&M cost data are sketchy, it is apparent that the construction cost for the oxidation ditch process is lower than the competing biological process for a plant size of 0.15 mgd or greater. Below 0.15 mgd, a package activated sludge plant or an RBC plant can be less costly. It is important to note that this cost comparison still favors the oxidation ditch, even if only secondary treatment (BOD and SS removal) is included in the competing processes.

The assumption in the EPA's 1978 report and the claim of some manufacturers that construction costs for oxidation ditch denitrification will not increase is not well proven. Since an anoxic zone is required for denitrification and a section of the channel must be designed for this purpose, extra channel capacity should be provided. Otherwise, there is no assurance that the existing channel would have great enough capacity in the aerobic state for BOD removal and successful nitrification. However, it is recognized that the added cost for a single channel system with removal of BOD-nitrification-

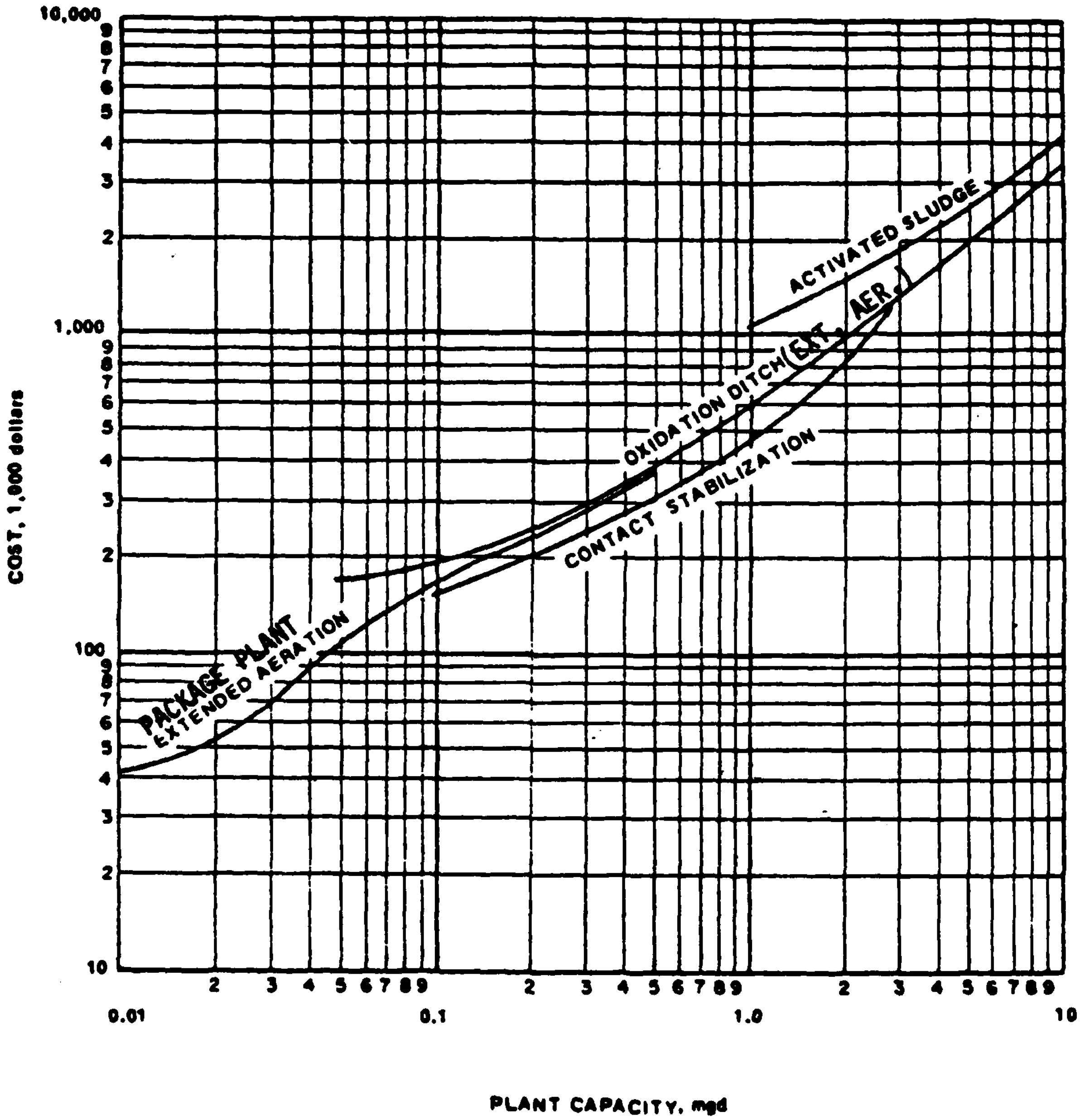


Figure 10. Biological treatment process construction costs, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).)

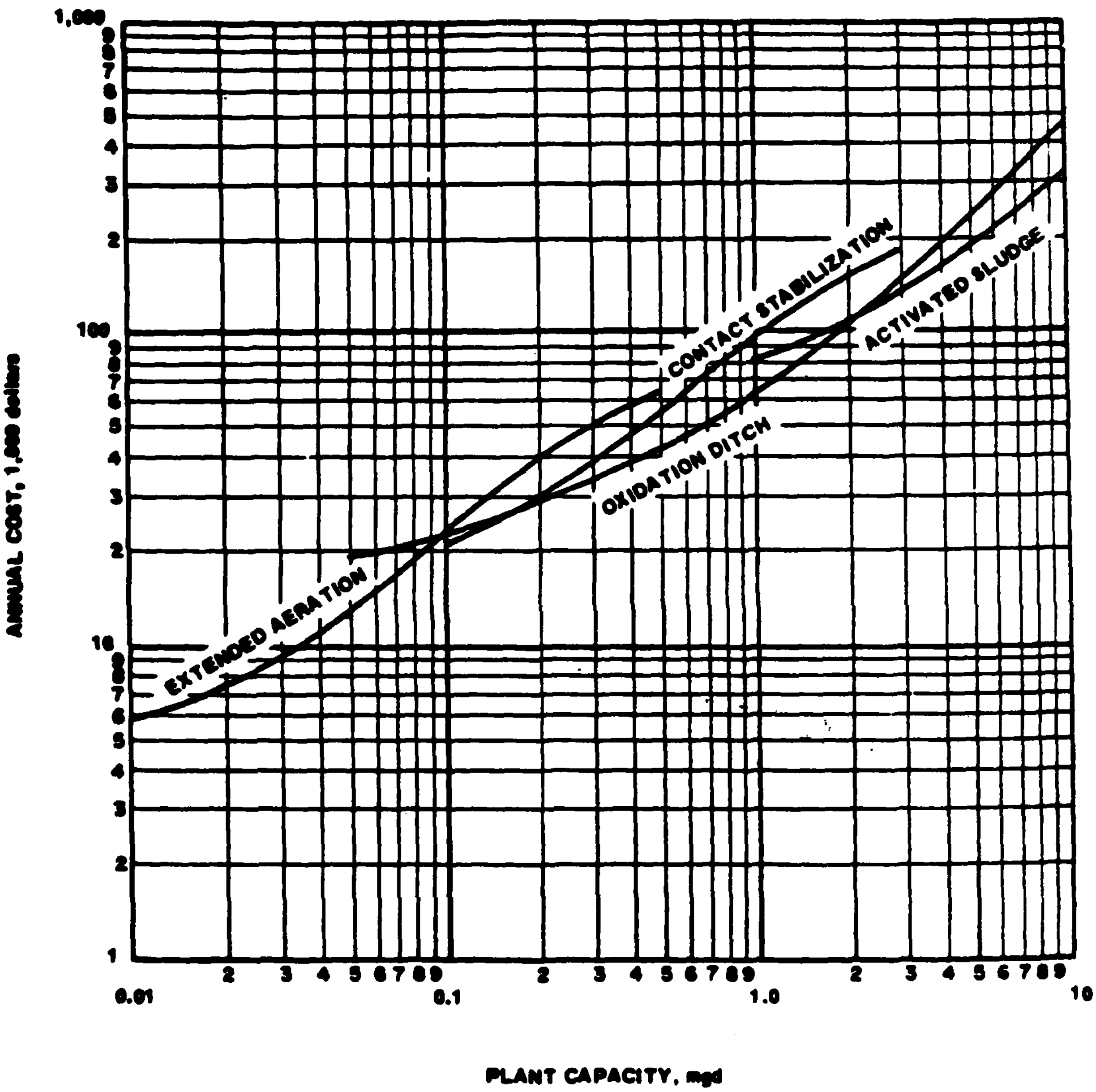


Figure 11. Biological treatment process operation and maintenance cost, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).)

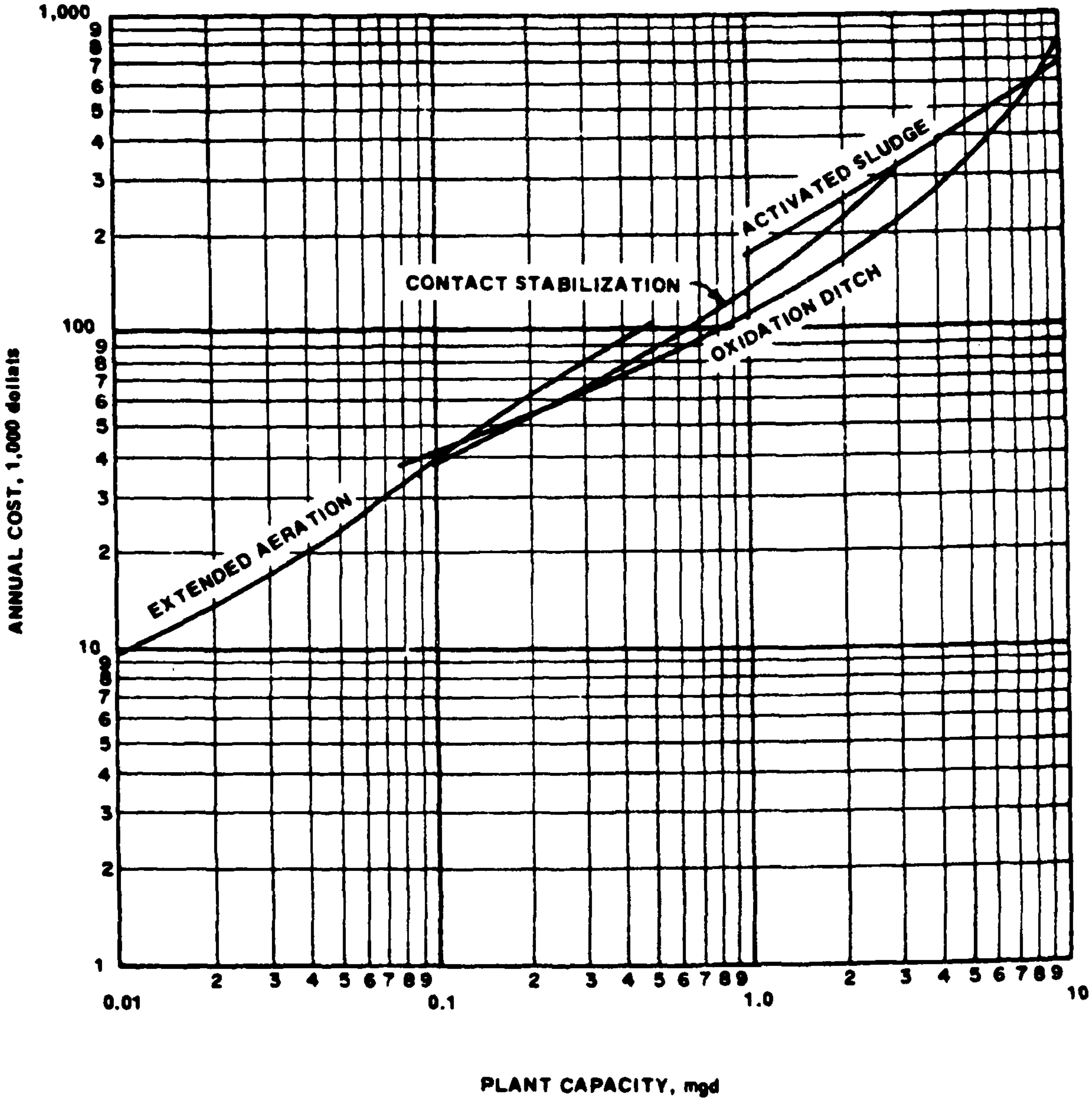


Figure 12. Biological treatment process total annual cost, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).)

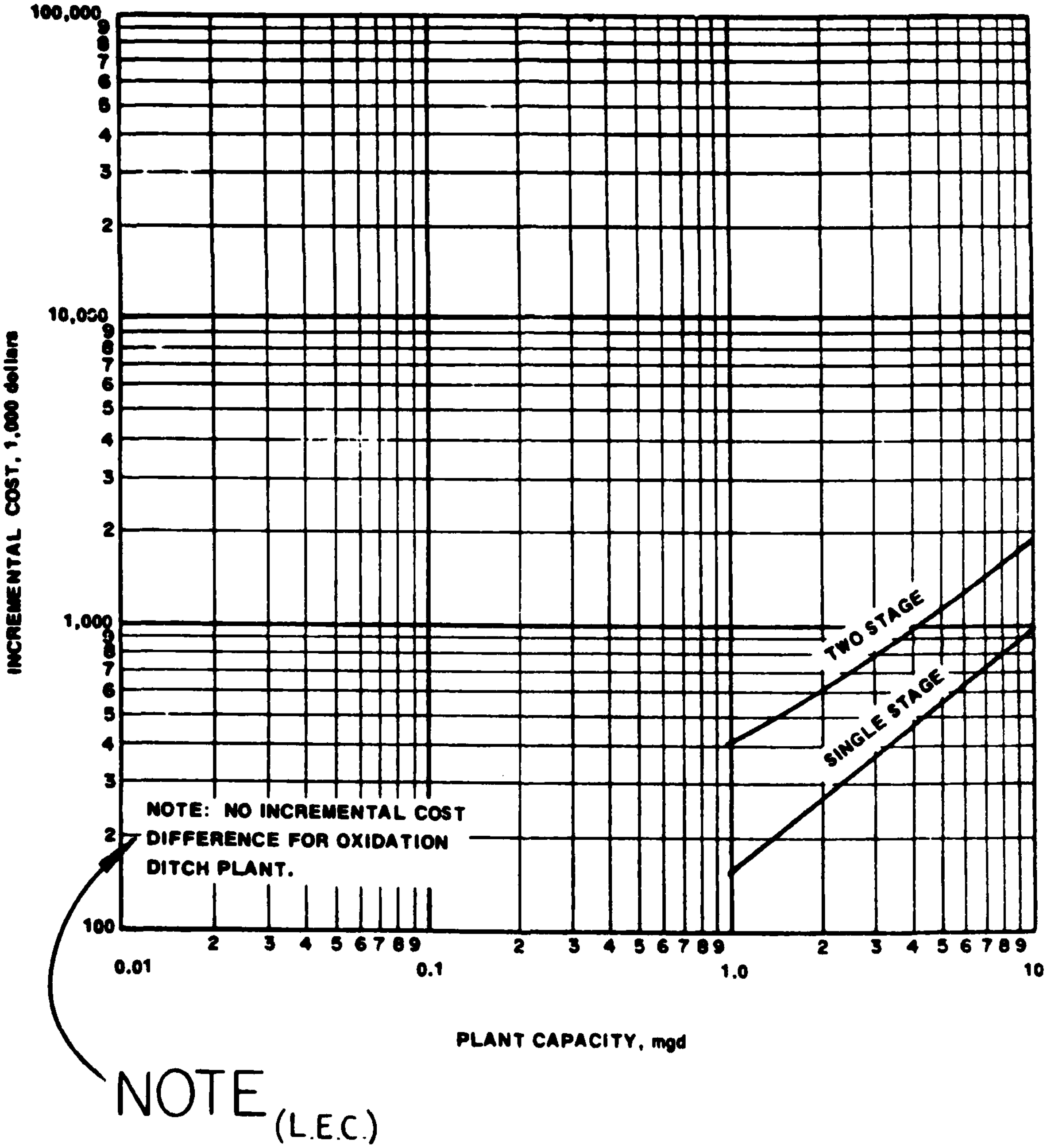


Figure 13. Incremental construction cost for biological nitrification, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).)

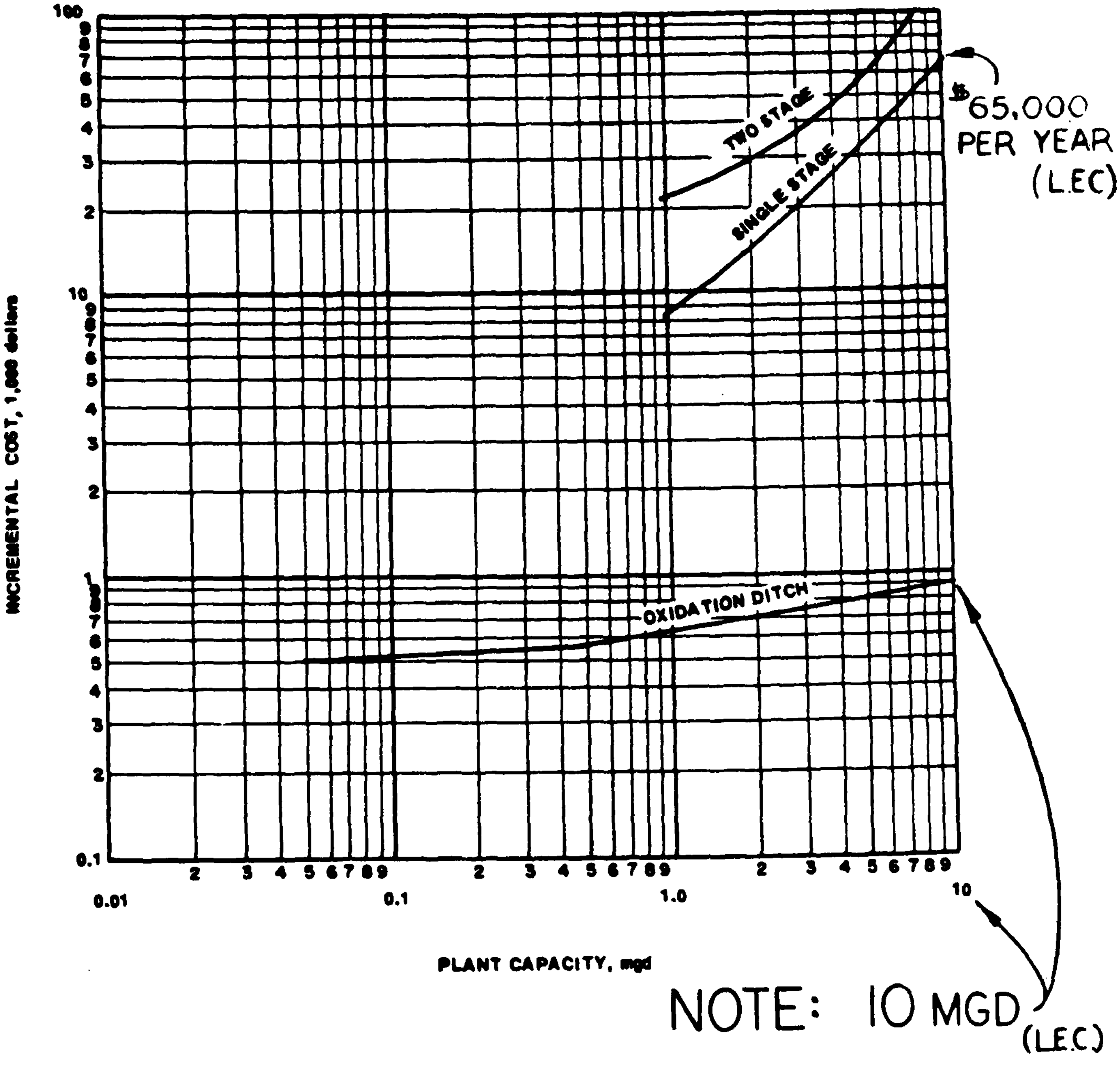
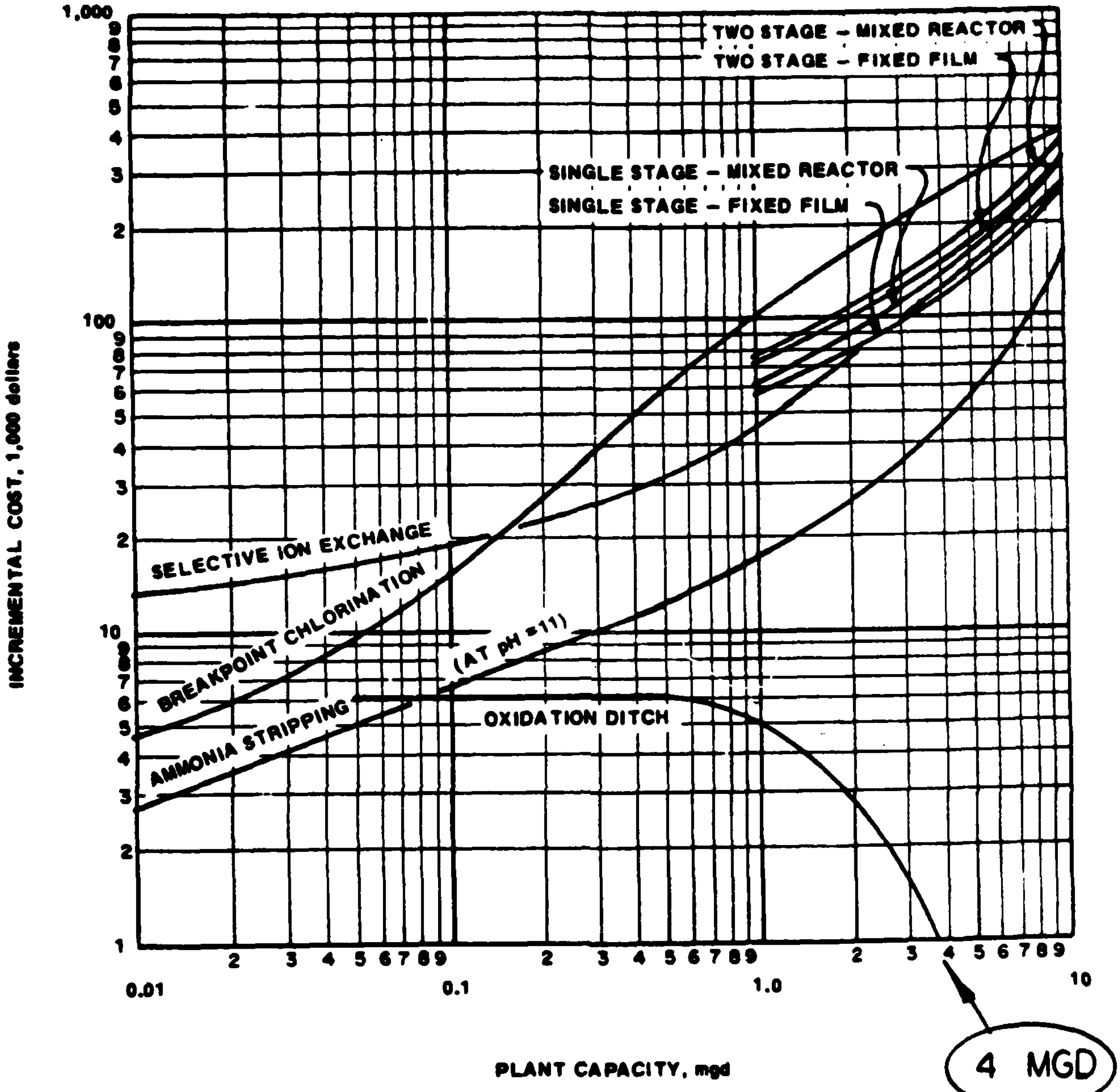


Figure 14. Incremental operation and maintenance cost for biological nitrification, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).)



Mixed reactor and fixed film denitrification include incremental costs of single or two stage biological nitrification (over costs of conventional activated sludge).

NOTE COST SAVINGS (PROFIT)

Figure 15. Incremental operation and maintenance cost for biological and physical-chemical dentrification, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).)

Table 22

Oxidation Ditch Process Cost

Capacity (mgd)	Detention Time (Hours)	Year of Cost Estimation	Construction Cost (Million $)	O&M Cost (Million $/yr)	Source of Information
1.95 deep channel	16 hr	August 1979	0.40	0.070	JAC Oxyditch preliminary design for Fort Mead, MD
4.50 deep channel	16 hr	February 1980	1.51	0.085	JAC Oxyditch preliminary design for Fort Mead, MD
0.011 shallow chennel	24 hr	June 1981	0.055	0.0075	CERL Technical Report N-102 on RBC Technology Evaluation for CE Recreational Area Application
0.25			0.256385		Burns & McDonnell oxidation ditch(s) with intrachannel clarifier design*
0.50			0.325604		
1.00	20 hr	April 1982	0.480299		
2.50			1.189450		
5.00			1.84244		
10.00 shallow channel			2.940749		

Table 23

Cost Comparison of Oxidation Ditch and Competing Biological Processes
(Liquid Trains Only, Excluding Primary Treatment and Sludge Treatment)
1982 Dollar Value

Process	Capacity (MGD)	Sec. Treatment (BOD&SS Removal) Construction cost $\$ \times 10^6$	Sec. Treatment (BOD&SS Removal) O&M cost \$/yr	Sec. Treatment + Nitrification Construction cost $\$ \times 10^6$	Sec. Treatment + Nitrification O&M cost \$/yr	Sec. Treatment + Nitrification and Denitrification Construction cost $\$ \times 10^6$	Sec. Treatment + Nitrification and Denitrification O&M cost \$/yr
Oxidation ditch	0.011	0.0575	7,780				----
	0.25	0.2560	-----				----
	0.50	0.3250	-----	same as secondary treatment			----
	1.00	0.4800	-----				----
	1.95*	0.4800	84,000				----
*deep channel	2.50	1.1900	-----				----
	4.50*	1.7700	99,450				----
	5.00	1.8400	-----				----
	10.00	2.9400	-----				----
Activated sludge (diffused air)	0.10	0.1800	9,200	0.375	34,500	0.525	49,500
	0.50	0.4200	20,000	0.780	49,500	1.020	88,500
	1.00	0.6600	29,300	1.185	67,500	1.500	129,000
	5.00	1.9500	102,000	3.375	177,000	4.275	402,000
	10.00	3.2300	186,000	5.475	291,000	6.870	726,000
Trickling filter	1.00	0.6250	14,900	1.152	52,500	1.468	115,000
	5.00	2.2240	55,000	3.654	123,000	4.556	349,000
	10.00	3.9300	86,900	6.082	192,000	7.481	628,000
RBC	0.50	0.6470	23,300	-----	-----	-----	-----
	1.00	1.1700	35,300	-----	-----	-----	-----
	5.00	5.0000	102,000	-----	-----	-----	-----
	10.00	9.4500	201,000	-----	-----	-----	-----

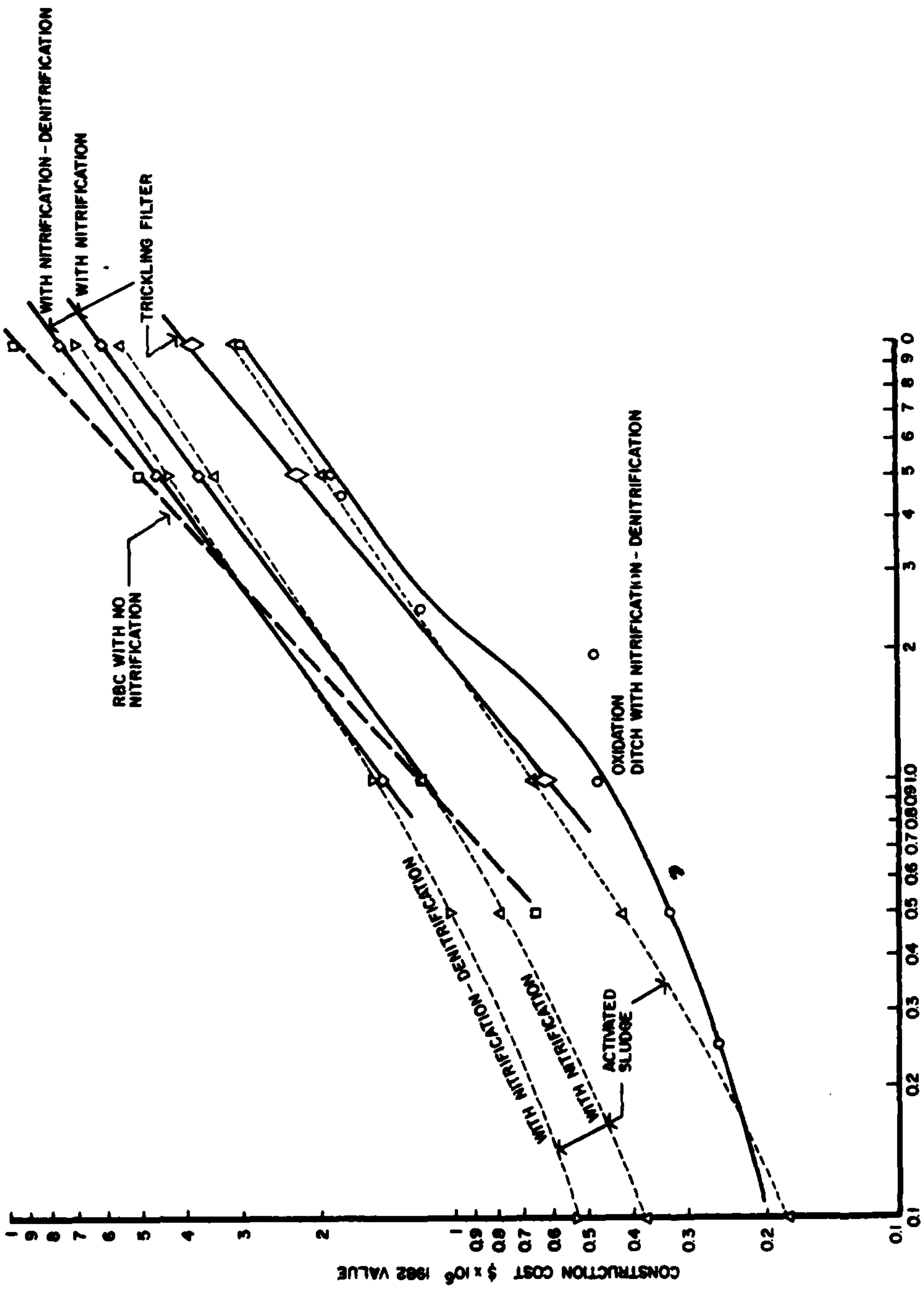


Figure 16. Updated cost comparison of oxidation ditches and competing biological processes.

denitrification capacity is much less than the added cost for a separate denitrification stage to other treatment processes. Thus, even if the construction cost curve for the oxidation ditch in Figure 16 needs adjustment upward for denitrification, the cost increase is not as much as for others. The lower construction cost for oxidation ditch plants with a capacity of 0.15 mgd or more is therefore valid, whether or not nitrification-denitrification is considered.

The 1978 EPA report shows lower O&M costs for oxidation ditch plants up to 1.0 mgd than for activated sludge plants, even if nitrification-denitrification in the ditch is not considered. No O&M cost comparison for plant sizes between 1.0 to 5.0 mgd is available. At 10.0 mgd capacity, O&M for an oxidation ditch, without considering the benefit of nitrification, would be more costly than for a conventional activated sludge process because of the high energy demand in aeration and mixing for a long hydraulic detention time. However, if nitrification or nitrification-denitrification in separate stages is included in activated sludge treatment plants, the O&M costs would be much higher than for single-stage oxidation ditch plants.

The energy requirements of wastewater treatment technology and its associated costs are important considerations. Therefore, one unique feature should be emphasized when the oxidation ditch plant is to be operated as a single stage for BOD removal plus nitrification-denitrification. In denitrification or nitrate respiration, 2.85 lb of ultimate BOD are oxidized during the conversion of nitrate to nitrogen. This is the result of substrate nitrate respiration plus synthesis as indicated in Eq 1.

$$\begin{aligned} \text{organic } C_{10}H_{19}O_3N + 6.5NaNO_3 \longrightarrow\ & 0.125\ NH_3 + 5.625\ CO_2 + \\ & 0.875\ C_5H_7NO_2 \text{ (bacteria)} + \\ & 6.5\ NOH + 3.25\ N_2 + 3\ H_2O \end{aligned} \qquad \text{[Eq 1]}$$

In practice, about 2.6 lb of ultimate BOD per pound of NO_3-N converted has been used. Thus, when denitrification occurs in an oxidation ditch, it is possible to use the oxygen supplied by nitrate. This is because the BOD is circulating in the channel from the anoxic zone back to the aerobic zone for BOD oxidation. This credit can only be claimed in circulated reactors (as in an oxidation ditch), but not in other processes with separate stages for BOD removal, nitrification, and denitrification using an external carbon source. The net effect is that the oxidation ditch saves energy.

Another factor that may lower oxidation ditch costs is the alkalinity requirement for nitrification. Nitrate respiration creates alkalinity which recirculates in the channel back to the nitrification zone. This can reduce the cost for chemicals in a plant where an alkalinity supplement is needed for successful nitrification.

Guidelines for Oxidation Ditch Selection

This section provides guidelines for selecting oxidation ditch technology. The discussion is confined to centralized sewerage systems with wastewater treatment for surface disposal. Thus, small, on-site treatment systems such as septic tank-leaching field, evaporation lagoon, etc., are not considered. Land treatment and lagoons are also not discussed because controlled nitrification-denitrification is difficult, if not impossible, in these processes.

Choice of Oxidation Ditch Over Other Processes

1. Cost. In most cases, cost is very important when selecting a treatment technology. Within the capacity range of 0.15 to 10.0 mgd, oxidation ditch plants have the lowest construction cost and the lowest annual cost.

2. Performance and Reliability. Existing oxidation ditch plants produce an effluent quality equal to or better than that given by competing biological treatment plants, including activated sludge plants, trickling filters, three-cell lagoons, and package plants. Effluent quality is consistently superior in both hot and cold weather. Even nearly complete nitrification is observed at near zero degree temperatures. The large ditch volume provides the equalization capacity needed to mitigate the effects of any shock loads which can produce consistently low effluent BOD. The conservative design of the final clarifier which uses low overflow rates provides effluents with low SS concentrations.

3. Operator Competence Level. The 1978 EPA report indicates that oxidation ditch plants provide superior performance and reliability even though they are operated by personnel with less training. Many plants are attended only on weekdays with no night shift. For a small plant, only a minimal grade level rating is required for the chief operator, even if nitrification is needed. The O&M is greatly simplified because the sludge is stabilized in the channel and requires no further treatment before dewatering on drying beds. The high MLSS level in the ditch allows freedom of operation without affecting the treatment performance. Even under poor operating conditions, plants generally do not generate odors.

4. Army Mobilization. An oxidation ditch plant eliminates the need for a primary clarifier and equipment for sludge treatment (except drying beds for dewatering). Shallow channels with sloped side walls can be constructed easily, using a thin concrete lining (3 in. thick) applied pneumatically and a hand trowel without the use of forms. The one-stage channel for BOD removal and nitrification (and denitrification if needed) eliminates the need for more clarifier and pumping equipment.

Choice of Other Treatment Processes Over Oxidation Ditch

1. Plant Size. Below a 0.15-mgd capacity, a package extended aeration treatment plant or an RBC plant may cost less than an oxidation ditch, although the difference is not significant. On the other hand, when a large treatment plant is to be installed, the economic advantages of an oxidation ditch plant may peak around 10.0 mgd; at that size, the cost of the concrete required could exceed the mechanical costs of other schemes. Therefore,

outside the range of 0.15 to 10.0 mgd, other treatment alternatives should be considered.

2. Sludge Treatment. Some states may require that sludge from oxidation ditch plants be treated before dewatering on drying beds since the fluctuating quality of the sludge may not have been well stabilized in the oxidation ditch. Thus, for oxidation ditches, the requirement for sludge treatment will add capital and O&M costs, as well as increase the complexity of O&M. The advantages of oxidation ditch plants are also greatly reduced when they are designed only for secondary treatment and when nitrification and denitrification are not considered.

3. Land Availability. The land requirement for oxidation ditch plants is greater than for other treatment alternatives, except for land treatment or lagooning. Although deep channels can reduce the land requirement, sometimes a high groundwater table will prohibit such use. Land availability could become a critical factor which may show the treatment alternatives to be more favorable.

In the design examples given in Chapter 5, the land area required for the oxidation ditch component for a 1.0 mgd plant is about:

Carrousel, single ditch, 10-ft channel depth:	12,000 sq ft
Oval oxidation ditch, 2 units, 10-ft channel depth, 45-degree side slope with 10-ft-wide island:	20,000 sq ft
Deep channel, jet aeration 20-ft channel depth:	10,000 sq ft (including final clarifier area)

In comparison, a 1.0-mgd activated sludge plant needs an aeration tank (14 ft deep) plus a primary clarifier, which would occupy about 5000 sq ft.

5 DESIGN AND OPERATION OF OXIDATION DITCH

This chapter provides design guidelines and design features that will be helpful to Army engineers who are considering the oxidation ditch as a treatment alternative. Design examples and their cost estimates are provided and operational and maintenance problems and some suggested solutions are discussed. In addition, suggestions for an operator training program are presented.

Design Features and Guidelines

The following design features and guidelines are recommended for oxidation ditch plants.

Design Flow

Flow usually fluctuates more in small treatment plants. Even though any conventional oxidation ditch plant provides a long hydraulic detention time, it is common practice to assume a much higher flow during a certain portion of the day and a lower flow for the rest of the day. For example, 80 percent of the day's flow may come into the plant in a 12-hour period. The design flow therefore would be:

$$Q_{Des} = \frac{0.8 \text{ x average daily flow}}{0.5} \qquad \text{[Eq 2]}$$

$$= 1.6 \text{ average daily flow}$$

For larger plants (from 0.5 to 1.0 mgd), the design flow can be reduced to 1.3 times the average daily flow. Above 1.0 mgd, the design flow is simply the average daily flow.

Bar Screen and Comminutor

Bar screening -- the most important pretreatment step -- removes objects such as rags, boards, and other bulky objects before they reach the ditch. A comminutor can be used in parallel or in series (downstream) with the bar screen. Problems with comminutor operation due to straws, rags, wire, etc., in the wastewater flow are common. Therefore, to minimize maintenance problems, many plants do not install comminutors.

Channel and Configuration

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. A design engineer usually adopts a specific channel configuration and flow scheme recommended by the equipment manufacturer or supplier.

Depending on the plant size, channel depth and number of loops vary:

Shallow channel..........	For smaller plants and when land availability is not limited; depth up to 14 ft.
Deep channel.............	For larger plants and when land availability may be limited; depth 20 ft and above.
Number of channels.......	Multiple-channel or multiple-loop is preferable, so that part of the plant can be shut down for repair and maintenance.
Drain....................	Should be provided for each channel at the bottom. 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.
Channel lining...........	Deep channels are 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 in. The concrete or gunite should provide a minimum compressive strength of 3000 lb per sq in. in 28 days.

Type of Aerator

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

Rotor Aerator............	Table 24 lists the various rotor aerators available. The minimum length of shaft is 3 ft and the maximum length is from 7 to 30 ft. These aerators are suitable for shallow channels of any plant size. The Carrousel process uses a vertical-shaft, slow-speed, surface aerator which comes in various sizes to accommodate shallow to moderately deep channels.
Induction aerator........	This type of aerator, which is available in various sizes, draws the mixed liquor and air down a U-tube; from here they are discharged for a distance downstream in the channel. Compressed air at low pressure can be injected near the top of the downdraft 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 also suitable for shallow to moderately deep channels.
Jet aeration.............	Jet aeration is specifically designed for deep channels. Both air and the mixed liquor are pressurized (by aspirator pumping) into a mixing

Table 24

Comparative Aerator Characteristics

(From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 [USEPA, 1978].)

Manufacturer	Type or Model	Type Blades	Diameter (in.)	Shaft	Typical Submergence Range (in.)	Typical Speed Range (rpm)	Material of Construction	Max. Length Per Shaft (ft)
Lakeside	Cage & Mini	Hor. toothed blades	27 1/2	6 in. OD torque tube	2 to 10	60 to 90	Steel	16
Lakeside	Magna	3 in. wide brush, 14 in. long max.	42	14 in. OD torque tube	4 to 14	50 to 72		30
Passavant	Mammoth Series 5200	3 in. wide brush, 9.5 in. long	27	8 in. OD torque tube	3 to 9.5	90		12
Passavant	Mammoth Series 5300	3 in. wide brush, 12 in. long	39 3/8	torque tube	4 to 12	70		30
Walker*	Reelaer Class 6227	3 in. wide brush, 11 in. long	38	16 in. OD torque tube	7 to 10		Steel torque tube galv. blades and hardware	12 min 25 max
Envirex	Disc	1/2 in. wide plastic disc	52	5 in. OD torque tube	11 to 21	56 to 58	Steel torque tube	--
Cherne*	OTA Aerator	Perforated fiberglass blades	30			up to 110		7 (one size)

* No longer active in oxidation ditch equipment manufacturing.

	chamber; from here, the mixture is discharged as a jet stream into the surrounding channel liquid. Deep channels are used to take advantage of better oxygen transfer.
Diffused aeration plus slow mixer..............	This type of aeration is more suitable for deep channels. Air bubbles are sent up to the mixed liquor to provide oxygenation through a pipe grid system with diffusers, while a slow propeller mixer provides the flow circulation and mixing.

Size of Aerator

Rotor aerators -- either horizontal shaft or vertical shaft -- should be sized to provide adequate mixing and oxygenation. However, the same size of 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 BOD 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. Details of this procedure are presented on pp 67-90. 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.

Final Clarifier

A conservative design of the final clarifier is needed because oxidation ditch plants operating in an extended aeration mode produce biological flocs that do not settle as well as conventional activated sludge. Existing plants with clarifiers designed for an overflow rate of 450 to 500 gpd/sq ft report good floc settlement. Therefore, 450 gpd/sq ft are recommended for the design, based on average daily flow; 1000 to 1200 gpd/sq ft are recommended for peak flows. The design should be checked against a solid loading of 15 lb solid/sq ft-day. The larger of the two surface areas calculated with these two design criteria should be adopted.

Traditionally, the final clarifier is constructed outside of the channel. Recently, the Burns & McDonnell process successfully demonstrated a new concept for an intrachannel clarifier (see pp 163-173); with this procedure, a clear effluent is produced while the settled sludge is returned automatically without pumping. The new process eliminates pumping for sludge return, thereby reducing equipment and O&M costs; however, the need to waste large volumes of diluted, settled, less-stabilized sludge is a drawback. If this new process is improved and can show that it is cheaper than the traditional design (final clarifier outside of the channel) when sludge-handling costs are included, it will be a better alternative. Sludge return from the clarifier can be pumped by using air-lift pumps in small plants or centrifugal sludge pumps in larger plants. Since an air supply is generally not available in oxidation ditch plants and air-lift sludge pumps often clog, centrifugal sludge pumps should be used. Pump capacity design should be based on 100 percent of the designed daily average flow or 50 percent of the maximum hourly flow.

Sludge Dewatering and Disposal

Sludge from oxidation ditch plants operating in the extended aeration mode (sludge retention time [SRT] 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 SRT followed by aerobic digestion of the sludge for 7 to 15 days.

In most climates, 1.0 sq ft drying bed surface area per population equivalent (0.17 lb BOD/cap-day) should be used. This capacity can accept 2.2 cu ft of wasted sludge/100 cap-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.

Mechanical Design

Often, plant operations stop unexpectedly because mechanical drives trip out on momentary electrical failures and do not restart when power is restored. This may happen during evenings or on weekends when the plants are unattended. Designers should consider installing maintained-contact type electrical control for sewage-lift-pump, aerator, and sludge-return-pump drives. Time delays should also be provided so that all drives do not restart at the same time.

Centrifugal, nonclog pumps for sludge return give much better service than air-lift pumps. Many plants return sludge by gravity to the raw wastewater lift station and pump it to the ditch with the raw wastewater. This will require increasing the size of the lift pumps, but will eliminate one set of pumps.

Self-aligning bearings should be used. Double seals could reduce some bearing problems in gear drives, line shafts, and aerator shafts.

Use of very long, horizontal-shaft aerators should be avoided. Some plants experience aerator torque tube failure or excessive deflection of the torque tube. If a wide ditch is used, the width can be spanned, using two shorter aerators driven by a common drive.

Protective covers around bearings, couplings, and drive units should be provided to keep aerator spray away from them. Corrosion and grit on shafts often shorten seal life.

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 moderately cold areas, the spray from surface aerators will freeze on adjacent structures, bearings, gear reducers, etc., making maintenance difficult. Drive components 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.

Design Examples

This section gives three examples of oxidation ditch design, providing design procedure and design data from equipment manufacturers. Additional information and design examples for design by kinetics is presented in Mandt and Bell.[6] Pilot studies should be performed when possible.

Vertical-Shaft Rotor Aerators

An oxidation ditch with vertical-shaft rotor aerators, such as the Carrousel system (Figures 17, 18, and Table 25) or Activox system, is operated in an extended aeration mode with nitrification. The system is designed as a complete mix reactor.

Influent:

$Q_{avg.\ daily\ flow}$	= 1.0 mgd
$Q_{peak\ flow}$	= 2.0 mgd
BOD_5	= 250 mg/L typical of Army installations
TSS	= 250 mg/L typical of Army installations
VSS	= 200 mg/L
TKN	= 25 mg/L (All NH_3-N)
P	= 8 mg/L
pH	= 6.5 to 8.5
Minimum temp.	= 10°C
Maximum temp.	= 25°C

[6] M. G. Mandt and B. A. Bell, Oxidation Ditch in Wastewater Treatment (Ann Arbor Science, 1982).

Effluent Requirements:

BOD_5	$\leq$ 10 mg/L
TKN	$\leq$ 5 mg/L
NH_3-N	$\leq$ 1 mg/L
TSS	$\leq$ 20 mg/L

Waste Characteristics:

Y	= 0.8 lb solids produced/lb BOD_5 removed = maximum yield coefficient
k_d	= 0.05 day^{-1}
$k_{influent}$	= 0.23 day^{-1}
$k_{effluent}$	= 0.05 day^{-1}
$VSS_{effluent}$	= 0.70 $TSS_{effluent}$

Oxidation Ditch and Aerator Characteristics:

MLSS	= 4000 mg/L
MLVSS	= 0.8 MLSS
Channel velocity	= 1 fps
α	= 0.9
β	= 0.97

Aerator delivers 3.5 lb O_2/hp-hr at standard condition

DO	= 2.0 mg/L

Clarifier Characteristics:

Overflow rate	= 450 gpd/sq ft @ $Q_{avg.}$
	= 1000 gpd/sq ft @ Q_{peak}
Solids loading	= 15 lb/sq ft-day
Side water depth	= 15 ft

Return sludge concentration = 1.5%

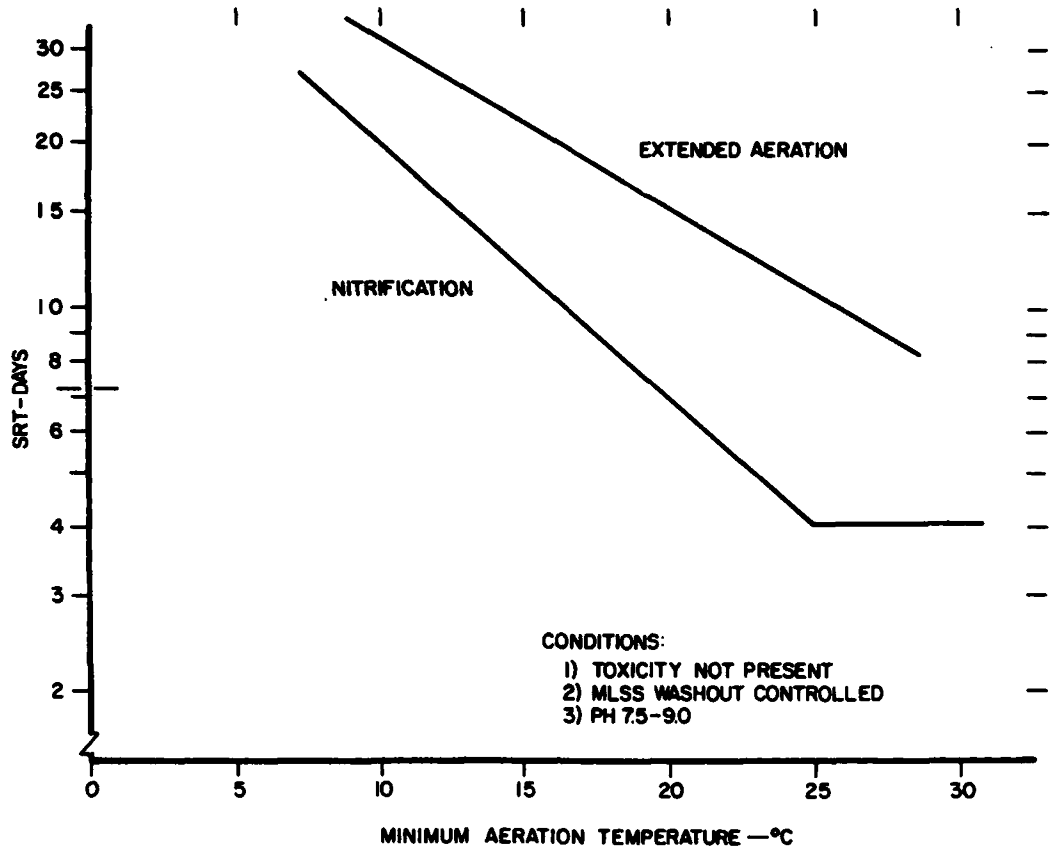


Figure 17. Oxidation ditch with vertical-shaft rotor aerators. (From Wastewater Treatment Plant Design [USEPA, 1977].)

Design Calculations:

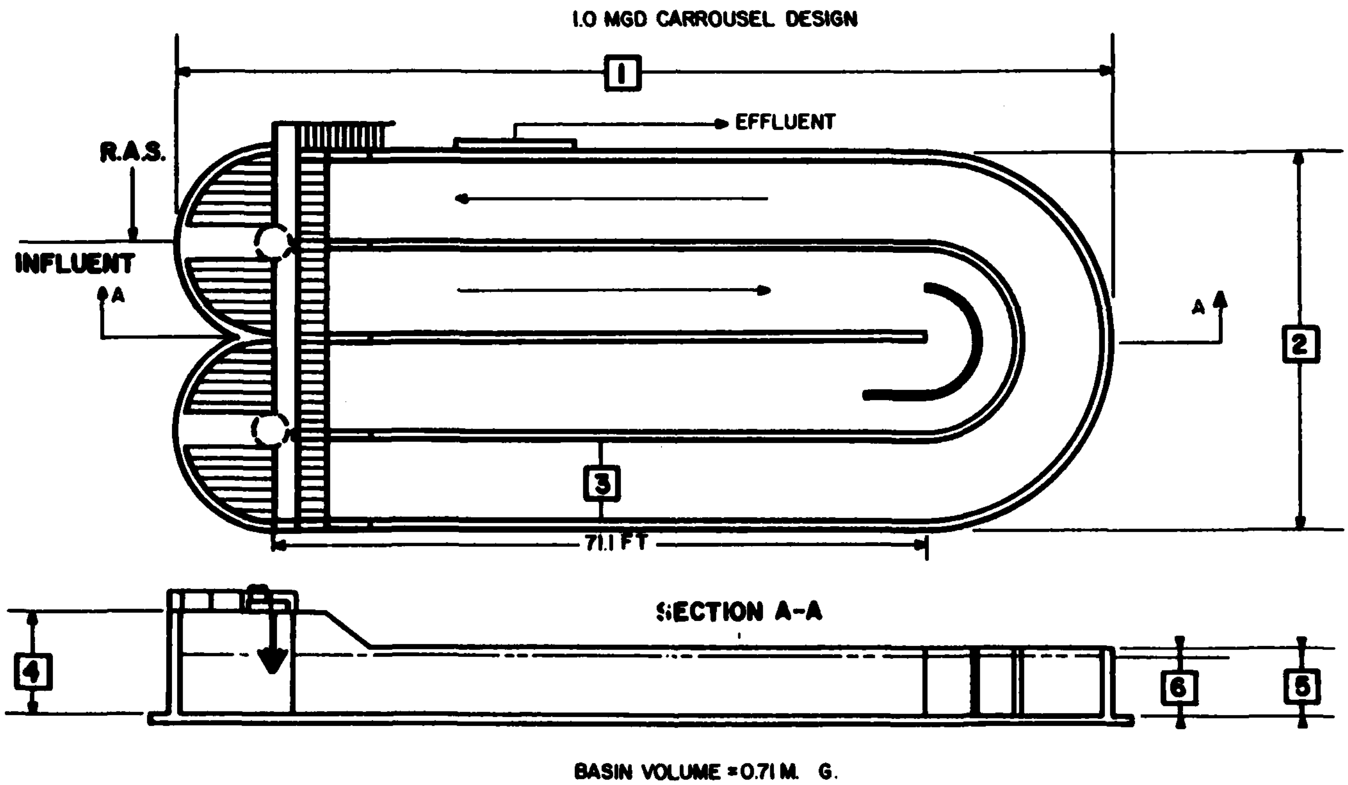
Choose the sludge retention time from the manufacturer's instructions. For a minimum wastewater temperature of 11°C, a sludge retention time (Θ_c) of 28 days is selected for extended aeration.

Calculate BOD_5 of the Effluent Solids:

BOD_5 of effluent solids = (fraction of the effluent solids which are volatile) x (total suspended solids concentration permitted in the effluent) x (typical ratio of ultimate BOD to mass of bacterial cells) x (fraction of the ultimate BOD which will be expressed in 5 days)

$$= 0.7 \times 20 \times 1.42 \times (1 - e^{-.05 \times 5})$$

$$= 4.4 \text{ mg/L}$$



HP	1	2	3	4	5	6
2 @ 25 HP	133' 10"	84'-0"	20'-0"	13'-6"	11'-0"	10'-0"

Figure 18. 1.0 mgd Carrousel design. (Information for this figure was obtained through personal communication with Envirotech Corp.).

Table 25

Carrousel[R] System Layout Summary
for
1.0 mgd Carrousel Design

Overall Length*....................................	134 ft, 2 in.
Overall Width*....................................	84 ft, 0 in.
Basin Water Depth....................................	10 ft, 0 in.
Aeration Zone Water Depth....................................	10 ft, 0 in.
Channel Width....................................	20 ft, 0 in.
Estimated Concrete Volume*....................................	See Below

*Assumes the following concrete thicknesses:

Outer Walls.......................	12 in.
Inner Partitions..................	8 in.
Floor..............................	6 in.

Concrete Volumes:

Aeration Zones	- 96.3 cu yd
Outer Curve	- 51.2 cu yd
Outer Wall	- 42.0 cu yd
Inner Partitions	- 67.2 cu yd
Footings	- 74.6 cu yd
Floor	- 175.8 cu yd
Total	- 507.1 cu yd

Calculate Allowable Soluble BOD_5 of the Effluent:

Allowable effluent soluble BOD_5 = total BOD_5 permitted in influent - BOD_5 of the effluent solids

= 10 - 4.4

= 5.6 mg soluble BOD_5/L

Calculate Oxidation Ditch Volume:

$$V = \frac{YQ(S_o - S_e)\theta_c}{X_d(1 + K_d \theta_c)} \qquad \text{[Eq 3]}$$

where

V = volume of oxidation ditch (million gallons)

Y = maximum yield coefficient (ratio of the mass of cells formed to the mass of substrate consumed)

Q = average flow to plant (mgd)

S_o = influent soluble BOD_5 (mg/L)

S_e = effluent soluble BOD_5 (mg/L)

θ_c = sludge retention time (days)

X_d = mixed liquor volatile suspended solids concentration (MLVSS) in oxidation ditch (mg/L)

K_d = endogenous decay coefficient (1/days)

$$V = \frac{(.8)(1.0)(250 - 5.6)(28)}{(3200)(1 + (.05)(28))}$$

$$V = 0.71 \text{ million gallons}$$

Check Hydraulic Detention Time:

$$\theta = \frac{V}{Q} \qquad \text{[Eq 4]}$$

where

θ = hydraulic detention time (days)

V = volume of the oxidation ditch (million gallons)

Q = average flow to plant (mgd)

$$\theta = \frac{.71}{1.0} = .71 \text{ days}$$

$$.71 \text{ days} = 17 \text{ hr}$$

which lies within the acceptable range of 16 to 24 hr.

Channel Sizing:

Use channel depth = 10 ft

channel width = 20 ft

depth at aerator zone = 10 ft (flat bottom)

0.71 million gallons = 95,000 cu ft

$$\text{surface area required} = \frac{V}{\text{depth}}$$

where

V = oxidation ditch volume (cu ft)

$$\text{surface area required} = \frac{95{,}000}{10}$$

surface area required = 9500 sq ft

$$\text{length of channel} = \frac{\text{surface area required}}{\text{channel width}}$$

$$\text{length of channel} = \frac{9500}{20} = 475 \text{ ft}$$

For the configuration shown, the length of the curved sections (calculated along the mid-lines) is 190.6 ft.

$$\text{Length of curved section} = 2(\pi)(10) + \pi(30.67) + \pi(10) = 190.6 \text{ ft}$$

Length of straight section, $L_s = \dfrac{475 - 190.6}{4}$
(inside dimension)

$= 71.1$ ft

Overall length $= 71.1 + 41.67 + 21$
(outside dimension)

$= 133.8$ ft

The thicknesses of the various concrete walls have been included in these calculations.

The designer should check with the manufacturer to see if this configuration has the best hydraulic efficiency with the aerator size selected.

Calculate Oxygen Requirements:

$$\text{Net yield of MLVSS} = 8.34\ YQ\ (S_o - S_e) - 8.34\ k_d X_d V \qquad \text{[Eq 5]}$$

where

Y = maximum yield coefficient (dimensionless)

Q = average flow to plant (mgd)

S_o = influent soluble BOD_5 (mg/L)

S_e = effluent soluble BOD_5 (mg/L)

K_d = endogenous decay coefficient (1/days)

X_d = MLVSS concentration in oxidation ditch (mg/L)

V = volume of oxidation ditch (million gallons)

Net yield of MLVSS = (8.34)(.8)(1.0)(250 - 5.6) - (8.34)(.05)(3200)(.71)

Net yield of MLVSS = ΔX = 683 lb/day

Oxygen will be required for the oxidation of much of the influent total kjeldahl nitrogen (TKN).

TKN concentration of effluent = 1 mg/L NH_3-N + TKN due to volatile suspended solids (VSS).

Since 70 percent of the effluent solids are volatile and 20 mg/L of suspended solids are permitted in the effluent, the assumption that the VSS has the empirical formula $C_5H_7NO_2$ (i.e., VSS is 12 percent N) permits the following calculation:

$$\text{TKN concentration of the effluent} = 1 + (.12)(.7)(20) = 2.7 \text{ mg/L TKN}$$

$$TKN_{out} \text{ in effluent} = (TKN)(Q)8.34 \qquad \text{[Eq 6]}$$

where

TKN = concentration of TKN in the effluent (mg/L)

Q = average flow to plant (mgd)

$$TKN_{out} \text{ in effluent} = (2.7)(1.0(8.34) = 22.5 \text{ lb/day}$$

An additional amount of nitrogen present in the wasted VSS is equal to the net yield of MLVSS.

$$TKN_{out} \text{ in wasted sludge} = (.12)(683) = 82 \text{ lb/day}$$

Total TKN which must be oxidized is the amount present in the influent, minus the ammonia-nitrogen permitted in the effluent, plus the TKN in the effluent VSS and the TKN in the wasted sludge; i.e.,

$$\text{TKN to be oxided } (\Delta N) = (25-1)(1)(8.34) - 22.5 - 82$$
$$\Delta N = 95.7 \text{ lb/day}$$

$$\text{Oxygen requirement} = \frac{8.34Q(S_o - S_e)}{1 - e^{-5K}} - 1.42\ \Delta X \qquad [\text{Eq } 7]$$

$$+ 4.5\ \text{lb/day}\ \Delta N$$

where

Q = average flow to plant (mgd)

S_o = influent soluble BOD_5 (mg/L)

S_e = effluent soluble BOD_5 (mg/L)

K = rate constant for influent soluble BOD removal (1/days)

ΔX = net yield of MLVSS or sludge wasted (lb/day)

ΔN = total TKN to be oxidized (lb/day)

BOD_5 is converted to ultimate BOD. The factors 1.42 and 4.5 are proportionality constants connecting masses of VSS and TKN, respectively, to the masses of oxygen required to stabilize them.

$$\text{Oxygen requirement} = \frac{(8.34)(1.0)(250 - 5.6)}{1 - e^{-5(.23)}}$$

$$- (1.42)(683) + (4.5)(95.7)$$

$$\text{Oxygen requirement} = 2443\ \text{lb } O_2\text{/day}$$

$$2443\ \text{lb } O_2\text{/day} = 102\ \text{lb } O_2\text{/hr}$$

Taking into account the α and β coefficients:

$$O_2\ \text{requirement} = \frac{102}{\alpha \dfrac{C_{sw} - C_L}{C_s} 1.024^{T-20}} \qquad [\text{Eq } 8]$$

where

$C_{sw} = \beta C_{ss}$ P @ 25°C; C_{ss} = 8.34 mg/L @ sea level; P = 1.0

C_{sw} = 0.97 x 8.34 x 1 = 8.13 mg/L

$$O_2\ \text{requirement} = \frac{102}{0.9 \dfrac{8.13-2}{9.17} 1.024^5}$$

$$= 150\ \text{lb/hr}$$

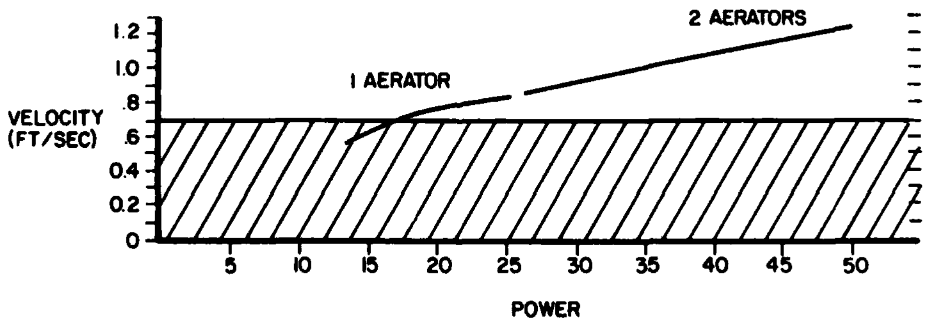
Sizing of Aerators:

$$\text{Horsepower requirement} = \frac{150 \text{ lb/hr}}{3.5 \text{ lb } O_2\text{/hp-hr}}$$

$$= 42.9 \text{ hp}$$

Use two aerators of 25 hp each. (NOTE: This is not the motor hp.) Information supplied by Envirotech (EIMCO PMD) (Figure 19) indicates that a power turndown from 50 to 18 hp, or 64 percent, can still maintain a minimal velocity of 0.7 ft/sec.

OPERATING CONDITION	NO. AERATORS	POWER	VELOCITY (FT/SEC)
Maximum Submergence	2	50 HP	1.21
(+ 7 1/2 inches)	1	25 HP	0.82
Mid Submergence	2	37.8 HP	1.05
(+ 2 3/4 inches)	1	18.9 HP	0.69
Minimum Submergence	2	26.4 HP	0.85
(- 1 3/4 inches)	1	13.2 HP	0.56



NOTE: Operation in shaded area may result in unacceptable solids disposition. However, once placed in operation specific plant operating experience may allow operation at lower velocities.

Figure 19. Carrousel design channel velocities. (Information for this figure was obtained through personal communication with Envirotech Corp.)

Calculate the Return Sludge Flow Rate, Q_R:

Mass balance around the secondary clarifier (assuming no change in sludge/content):

Influx of VSS = mass of VSS in the oxidation ditch influent + mass of VSS in the recycle flow + mass of VSS generated in the oxidation ditch

Outgo of VSS = mass of VSS which goes to the secondary clarifier + mass of VSS consumed by endogenous respiration

$$\text{Influx of VSS} = 8.34\ X_oQ + 8.34\ (.8)\ X_uQ_r + 8.34\ (S_o - S)V \qquad \text{[Eq 9]}$$

$$\text{Outgo of VSS} = 8.34\ X_d\ (Q + Q_r) + 8.34\ k_dX_d\ V$$

where

X_o = VSS concentration of influent (mg/L)

Q = average flow to the plant (mgd)

X_u = underflow SS concentration from clarifier (mg/L)

Q_r = recirculation flow (mgd)

S_o = influent soluble BOD_5 (mg/L)

S_e = effluent soluble BOD_5 (mg/L)

V = volume of oxidation ditch (million gallons)

K_d = endogenous decay coefficient (1/days)

X_d = MLVSS concentration in oxidation ditch (mg/L)

Since influx must equal outgo,

$$(8.34)(200)(1.0) + (8.34)(.8)(15000)Q_r$$
$$+(8.34)(250 - 5.6)(.71) = (8.34)(3200)(1.0 + Q_r)$$
$$+(8.34)(0.05)(3200)(.71)$$

Q_r = recirculation flow = .33 mgd

Choose two sludge pumps, each capable of pumping 0 to 0.5 mgd.

Sizing of Variable Overflow Weir:

Design flow = 1.0 mgd = 1.55 cfs

Peak flow = 2.0 mgd = 3.10 cfs

Carrousel system design recommends that weir length be such that installed horsepower is developed at peak flow and 90 percent of installed horsepower is developed at design flow:

$$L = \left[\frac{(0.3 \times 3.1)^{0.67} - (0.3 \times 1.55)^{0.67}}{\Delta h}\right]^{1.5} \qquad \text{[Eq 10]}$$

where Δh is weir head difference from design flow to peak flow. This head difference is determined from an impeller characteristic curve as shown in Figure 20. From the figure, Δh is 0.1625 ft.

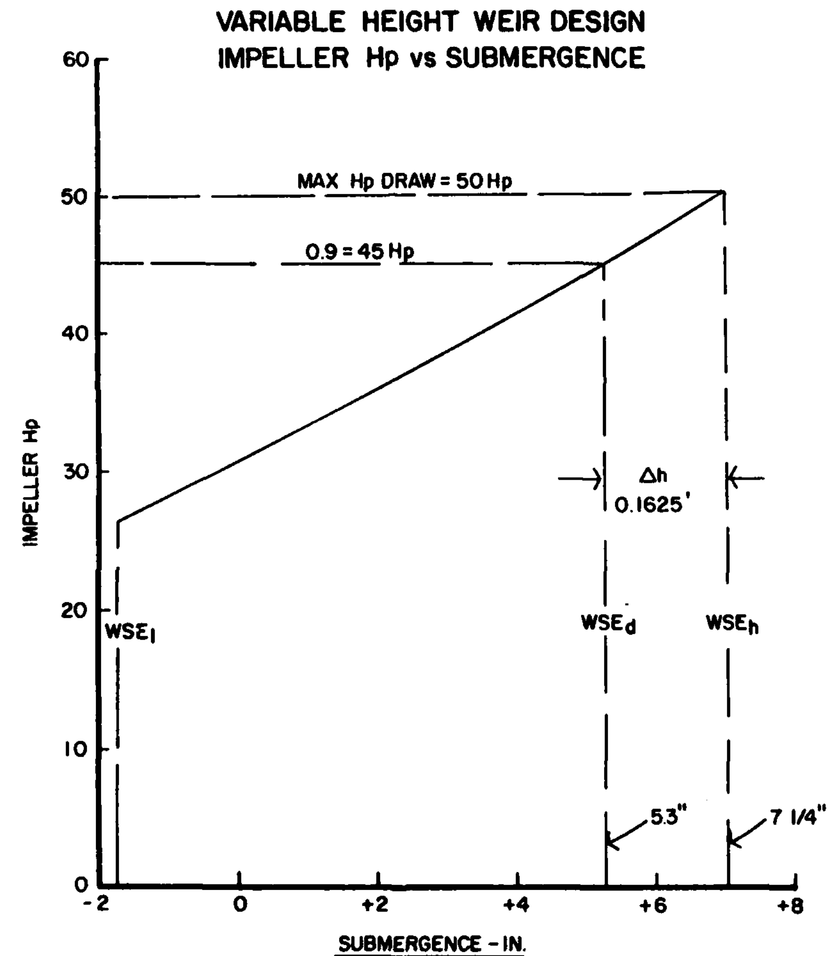


Figure 20. Impeller characteristic curve. (Information for this figure was obtained through personel communication with Envirotech Corp.)

$$L = \left[\frac{(0.93)^{0.67} - (0.465)^{0.67}}{0.1625}\right]^{1.5}$$

$$= 3.2 \text{ ft.}$$

Sizing of Secondary Clarifier:

Based on overflow rate,

$$A = \frac{1 \text{ mgd}}{450 \text{ gpd/sq ft}} = 2222 \text{ sq ft}$$

Based on solids loading rate,

$$A = \frac{4000(1+0.34)(8.34)}{15 \text{ lb/sq ft-day}}$$

$$= 2980 \text{ sq ft} - 2222 \text{ sq ft}$$

Area chosen = 2980 sq ft

Use two clarifiers of equal size:

$$\text{Diameter} = \left(\frac{2980 \times 1/2 \times 4}{\pi}\right)^{1/2}$$

$$= 43.6 \text{ ft}$$

Use two standard-size 45 ft x 15 ft SWD clarifiers.

Calculate Amount of Sludge For Disposal:

Total sludge = inert influent TSS + wasted sludge, P_x - $TSS_{effluent}$ [Eq 11]

$$= 50 \text{ mg/L} \times 1 \times 8.34 + 683.2 \text{ lb/day} - 20 \text{ mg/L} \times 1 \times 8.34$$

$$= 933.4 \text{ lb/day dry solids.}$$

At 1.5 percent concentration:

$$\text{Flow rate} = \frac{933.4 \text{ lb/day}}{0.015 \times 8.34 \text{ lb/gal.}} = 7461 \text{ gpd}$$

To maintain 3 fps in a 4-in. diameter sludge line, Q = 120 gpm

Schedule of wasting = 65 min/day at 120 gpm

Two pumps, each with a capacity of 120 gpm, wasting 32 min/day.

Sizing of Sludge Drying Beds:

$$\text{BOD population equivalent} = \frac{1 \times 250 \times 8.34}{0.17}$$

$$= 12{,}265 \text{ persons}$$

Drying bed area = 12,265 x 1 sq ft/cap.
= 12,265 sq ft

Use 10 drying beds, each at 35 ft x 35 ft

Total area = 12,250 sq ft

Table 26 summarizes the Carrousel oxidation ditch design.

Table 26

Design Summary of Carrousel Oxidation Ditch

Quantity	Unit	Size
1	Carrousel, vertical, reinforced-concrete wall	134 ft x 80 ft x 10 ft deep 95,000 cu ft
1	Variable weir	3.2 ft motor-operated
2	Vertical-shaft rotor aerator	25 hp each
2	Secondary clarifier	45-ft dia. x 15 ft SWD
2	Return sludge pump	0.5 mgd each
2	Sludge wasting pump	120 gpm each
10	Sludge drying beds	35 ft x 35 ft each

Horizontal-Shaft Rotor Aerators

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

Influent: identical to example for vertical-shaft rotor aerators.

Effluent requirement: identical to example for vertical-shaft rotor aerators.

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 $\leq$ 450 gpd/sq ft @ Q_{avg}
Solid loading $\leq$ 30 lb/sq ft-day, using the Spiraflo clarifier

Design calculations follow the procedure recommended by Lakeside Equipment Corporation.

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 x 250 mg/L x 8.34
= 1042.5 lb BOD/day

Calculation of Oxidation Ditch Volume:

Use 20 lb BOD/1000 cu ft-day

$$V = \frac{1042.5 \text{ lb/day}}{20 \text{ lb/1000 cu ft-day}} = 52{,}125 \text{ cu ft}$$

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

$$= 18.75 \text{ hr}$$

Calculation of Rotor Requirements:

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

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

Oxygenation requirement = 2.35 lb O_2/lb BOD applied -- recommended for domestic sewage.

Assume the following operating conditions:

RPM = 72

Immersion = 8 in.

Oxygenation = 3.75 lb O_2/hr-ft (from Figure 21), using a 42-in.-diameter MAGNA rotor)

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

Use two rotors per each unit oxidation ditch or 2- x 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 Figure 22), using 42-in.-diameter MAGNA rotor)

Aerator brake horsepower requirement = 1.34 x 0.84 x 14 = 15.76 Bhp for each rotor.

Calculation of Motor Horsepower:

Size all motors to allow at least 1 1/2-in above the 8-in. actual immersion to allow for peak flows.

Motor horsepower required at (8 + 1.5) or 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.

Channel Sizing (Figure 23):

Ditch liquid volume = 52,125 cu ft

Choose:

Depth: 10 ft

Side wall slope: 45 degrees

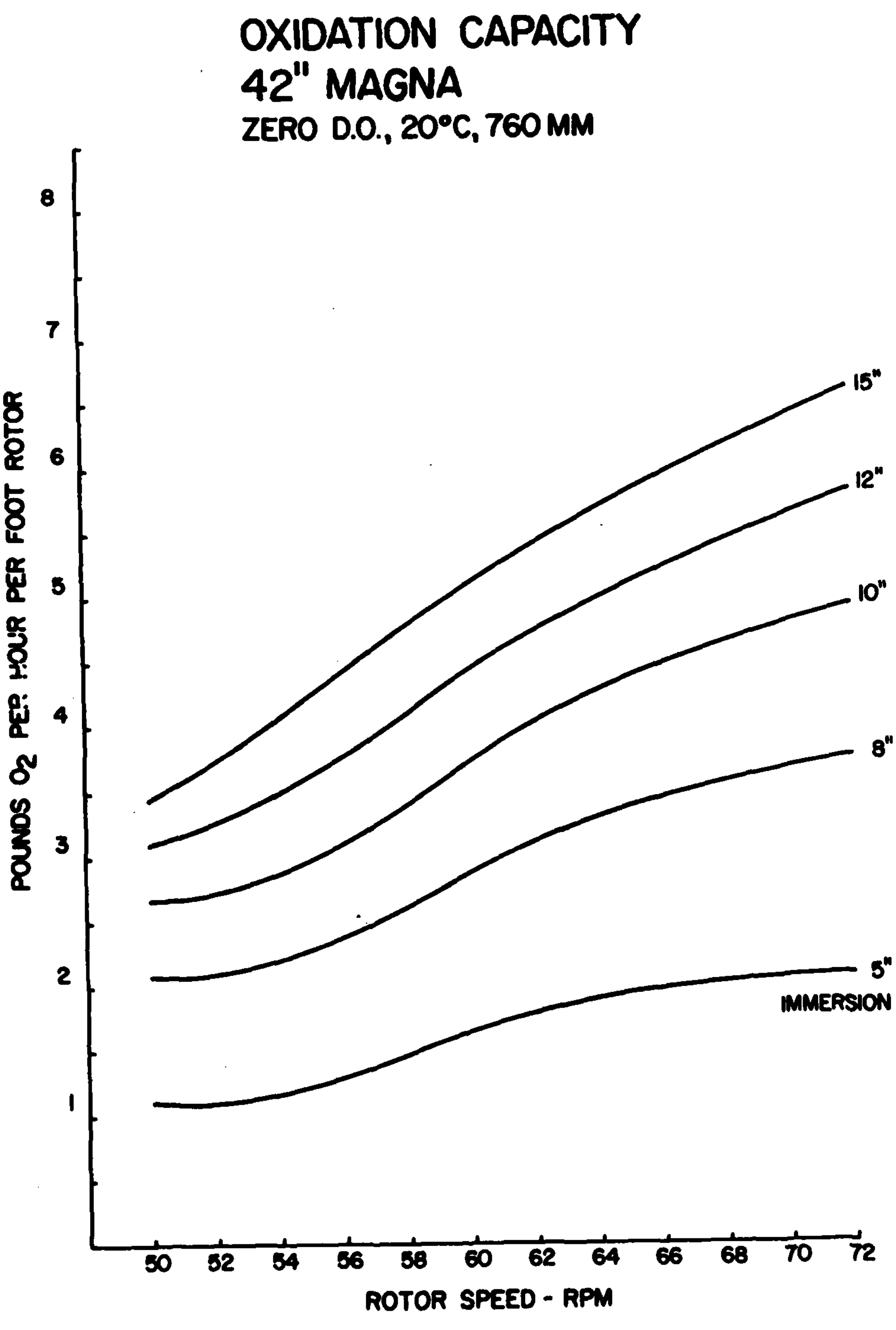


Figure 21. Oxygenation capacity curve. (From Bulletin RAD-390 [Lakeside Equipment Corp.].)

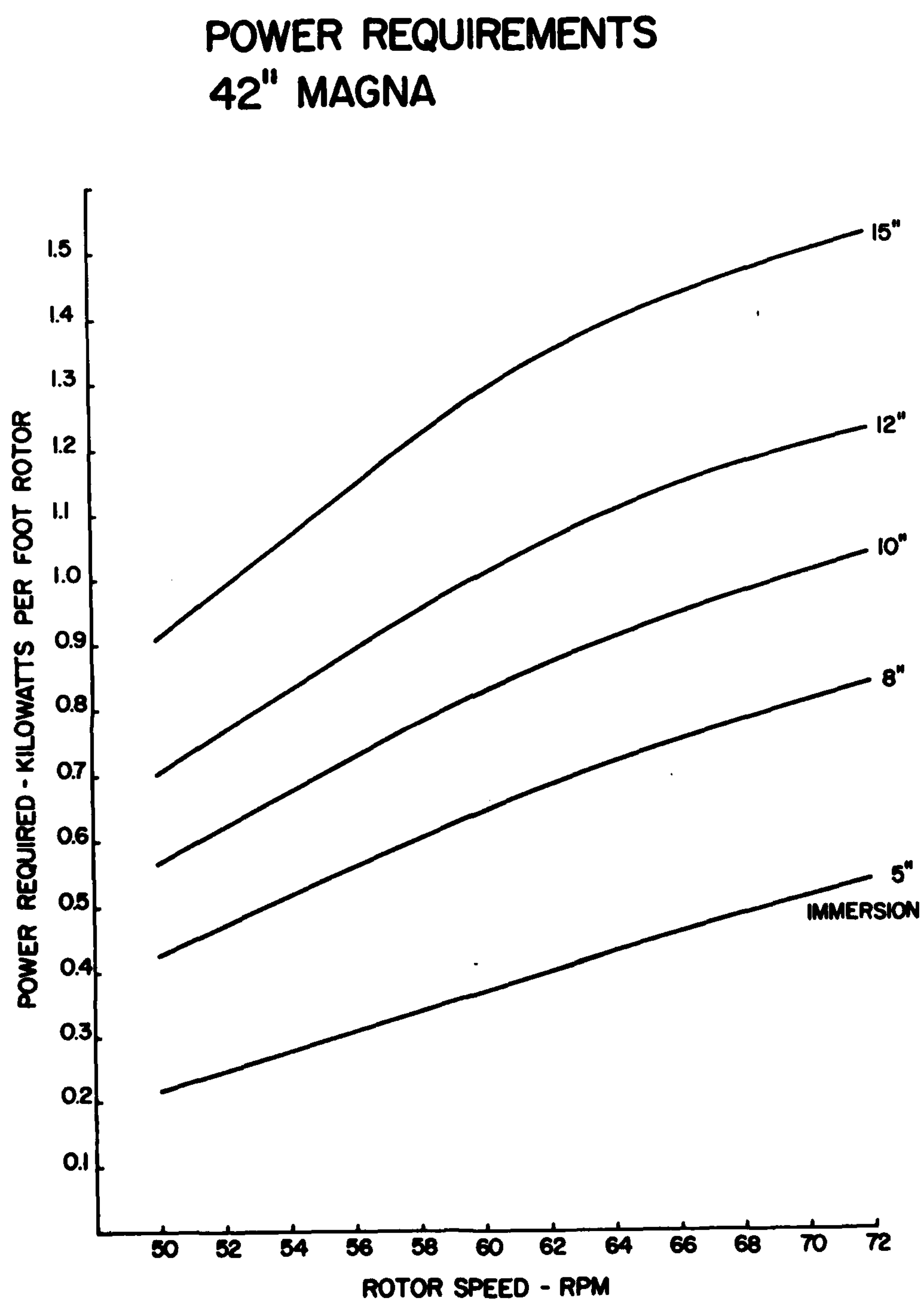


Figure 22. Power requirements curve. (From Bulletin RAD-390 [Lakeside Equipment Corp.].)

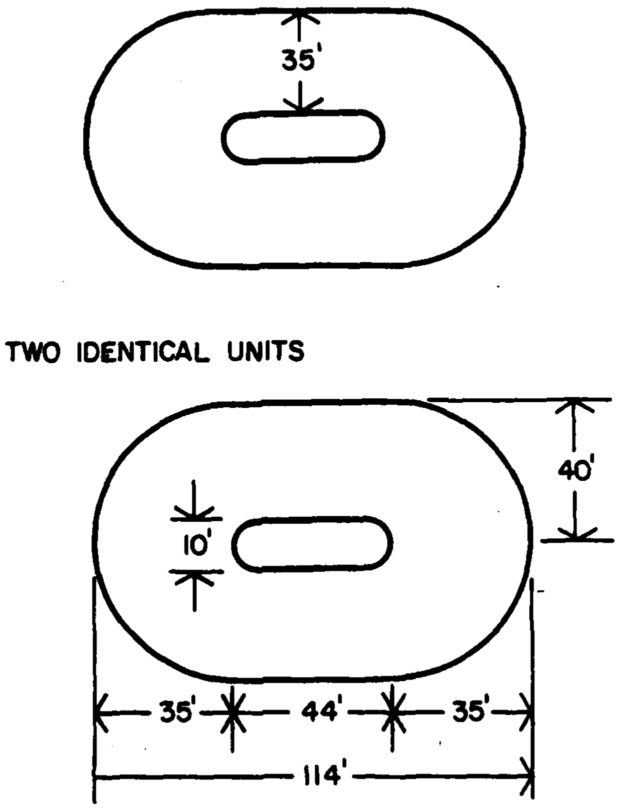


Figure 23. Channel sizing.

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

Ditch cross section area $= \left(\dfrac{15 + 35}{2} \right) \times 10 = 250$ 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

Total length of the ditch at the water line

$= \dfrac{16800}{2 \times 250} = 33.6$ or 34 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.

Final Clarifier:

One clarifier required for each ditch unit (Spiraflo clarifier)

Overflow rate $\leq$ 450 gpd/sq ft

Detention time = 3 hr

$$\text{Area required} = \frac{0.5 \times 10^6}{1 \times 450} = 1111 \text{ sq ft}$$

$$\text{Diameter} = \left(\frac{1111}{0.785}\right)^{0.5} = 37.6 \text{ or } 38 \text{ ft}$$

$$\text{Actual area} = (38)^2 \times 0.785 = 1133.5 \text{ sq ft}$$

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

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

$$\text{Straight wall depth} = \left(\frac{8356.5}{1133.5}\right) = 7.4 \text{ ft (use 8 ft)}$$

$$\text{Actual detention time} = 8 \times 1133.5 \times 24 \times 7.48/0.5 \times 10^6$$

$$= 3.25 \text{ hr.}$$

Check for Nitrification in Oxidation Ditch:

Find MLVSS concentration required at standard conditions (20°C, pH 8.5) to completely nitrify ammonia shown in Figure 24.

$$\text{MLVSS} = \frac{1}{0.0075} \quad \frac{25 \text{ mg/L } NH_3\text{-N}}{18.75 \text{ hr}} = 177.8 \text{ mg/L}$$

where 0.0075 mg NH_3-N/mg MLVSS-hr is the rate of nitrification given in Figure 24.

Change MLVSS concentration at standard conditions to design conditions (10°C, pH 6.5 to 8.5; assume pH is 7.0), using Figures 25 and 26 for temperature and pH corrections.

Temperature correction = 0.46 (from 20°C to 10°C)

pH correction = 0.50 (pH 8.5 to 7.0)

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

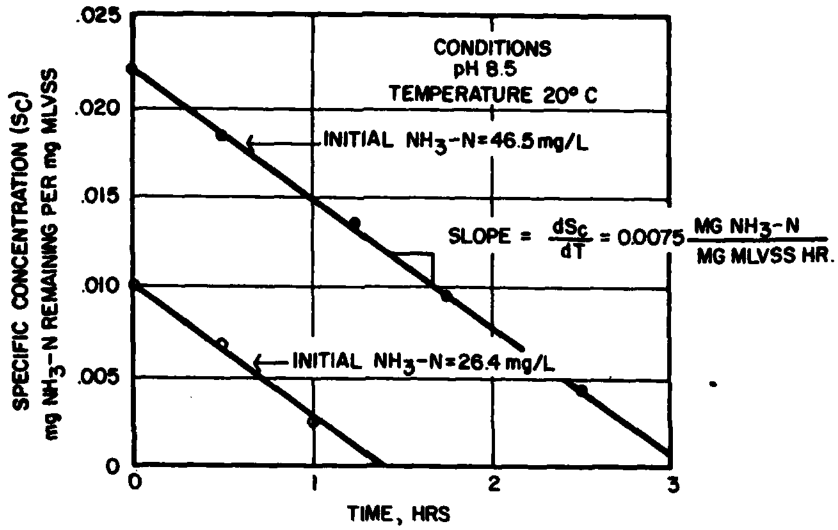


Figure 24. Ammonia nitrification conditicns. (From Metcalf & Eddy, Nitrification and Dentrification Facilities, Wastewater Treatment [USEPA, 1973].)

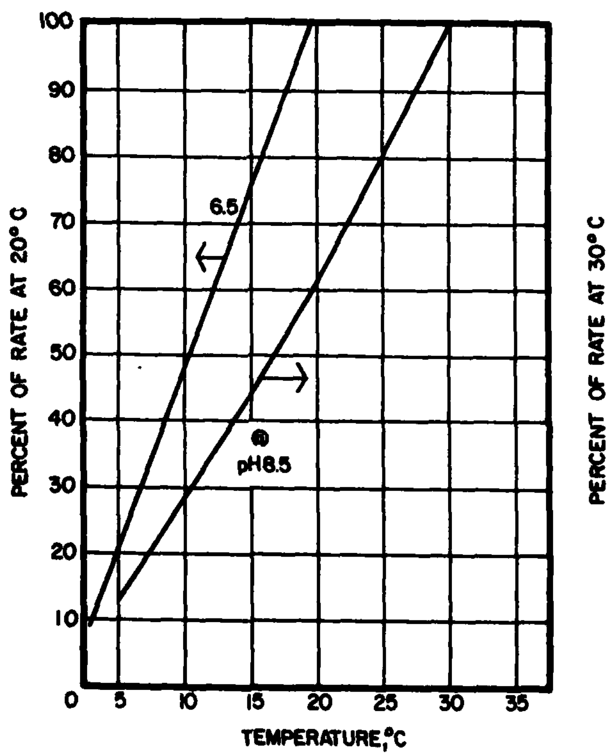


Figure 25. Temperature corrections. (From Metcalf & Eddy, Nitrification and Dentrification Facilities, Wastewater Treatment [USEPA, 1973].)

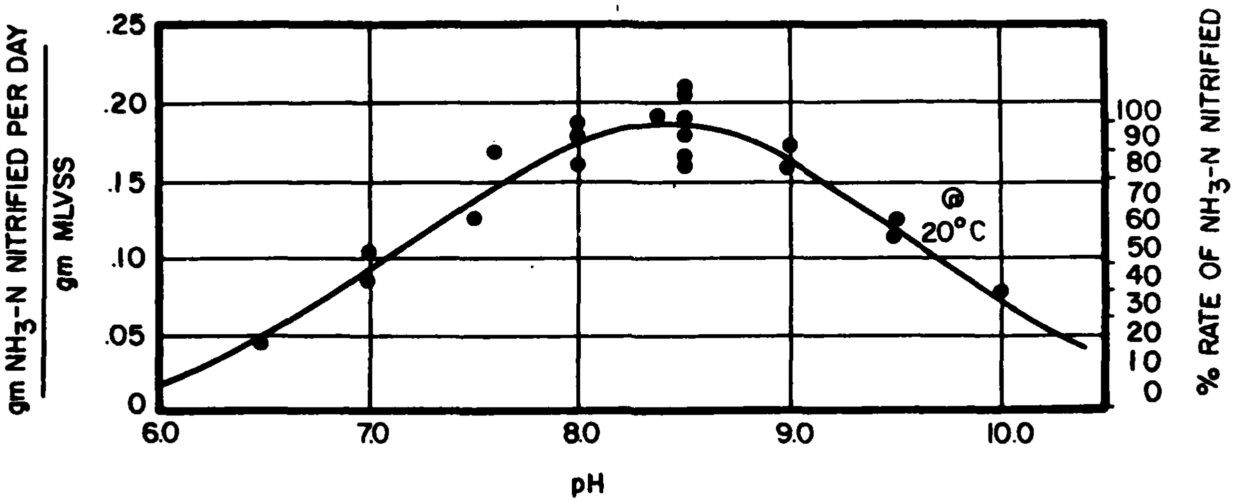


Figure 26. pH corrections. (From Metcalf & Eddy, Nitrification and Denitrification Facilities, Wastewater Treatment [USEPA, 1973].)

This concentration is low; the sludge age would be too short to give a stable performance and to obtain a stabilized sludge.

Use 3200 MLVSS:

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

Check sludge age (with negligible sludge wasting)

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

This sludge age is considered adequate.

Check Size of Final Clarifier With Solids Loading:

Spiraflo clarifier allows 30 lb solids/sq ft-day

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

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:

$$L = \frac{3.5 \, xQ_{avg} \text{ gpm}}{102}$$

$$= \frac{3.5 \times 347.2}{102} = 11.9 \text{ or } 12 \text{ ft.}$$

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

Two pumps, each with 44 to 174 gpm capacity.

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 two oxidation ditches

Total sludge = 933.4 lb/day dry solids from two oxidation ditches

Two pumps each with a capacity of 120 gpm for a 52 min/day wasting schedule as in the vertical-shaft aerator example.

Sizing of Sludge Drying Beds:

Use 10 drying beds, each 35 ft x 35 ft as in the vertical-shaft aerator example.

Total area = 12,250 sq ft

Table 27 summarizes the design criteria for the horizontal-shaft aerators and multiple ditch units.

Deep Channel and Jet Aeration

This section describes design criteria for an oxidation ditch with deep channel and jet aeration, operated in an extended aeration mode with nitrification and denitrification.

Influent: Identical to horizontal-shaft example

Effluent: Identical to horizontal-shaft example; additional requirement -- total N = 8 mg/L

Waste Characteristics:

$Y = 0.8$ lb solids produced/lb BOD_5 removed

= total sludge production

$k_d = 0.05\ day^{-1}$

Table 27

Design Summary of Oxidation Ditch with Horizontal Shaft Aerators and Multiple Ditch Units

Quantity	Unit	Size
2	Oval-shaped oxidation ditch with 10-ft-wide median strip. 45-degree side slope. Lining using gunite or shotcrete method of construction.	114 ft x 80 ft 10 ft deep
4	Horizontal-shaft rotor aerator; two each in oxidation ditch.	14 ft long each; 16 BHp each (motor 20 Hp)
2	Secondary clarifier	38-ft diameter x 8-ft SWD (Spiraflo clarifier)
2	Return sludge pump	0.125 to 0.5 mgd each (44 to 174 gpm)
2	Sludge wasting pump	120 gpm each
2	Adjustable oxidation ditch weir	12 ft each
10	Sludge-drying bed	35 ft x 35 ft each

$k_{influent}$ = 0.20 day^{-1}

$k_{effluent}$ = 0.04 day^{-1}

$VSS_{effluent}$ = 0.80 $TSS_{effluent}$

D/N rate = 0.6 mg NO_3-N/g MLSS-hr

Maximum air temperature = 100°F.

Oxidation Ditch and Aerator Characteristics:

MLSS = 4000 mg/L

MLVSS = 0.8 MLSS

Channel velocity = 1 fps

α = 0.92

β = 0.98

DO = 2.0 mg/L

Aerator efficiency at 20-ft channel depth or 3.5 lb/hr-hp (judged to be conservative by the equipment manufacturer).

Clarifier Characteristics:

Overflow rate = 450 gpd/sq ft @ Q_{avg}

Solids loading = 15 lb/sq ft-day

Side water depth = 15 ft

Return sludge concentration = 1.5 percent

Design Calculations:

Select a sludge age (θ_c) = 32 days

(suitable for nitrification-denitrification and operated in the extended aeration mode)

Calculate BOD_5 of the effluent solids

$= 0.8 \times 20 \times 1.42\ (1 - e^{0.04} \times 5)$

= 4.1 mg/L

Effluent BOD_5 allowable = 10 - 4.1 = 5.9 mg/L.

AD-A133 140 OXIDATION DITCH TECHNOLOGY FOR UPGRADING ARMY SEWAGE TREATMENT FACILITIES(U) CONSTRUCTION ENGINEERING RESEARCH LAB (ARMY) CHAMPAIGN IL J T BANDY ET AL. 2/2

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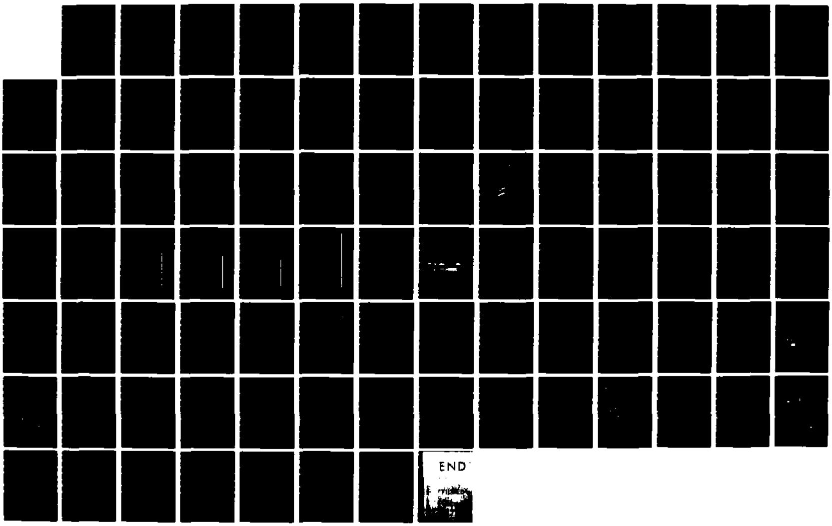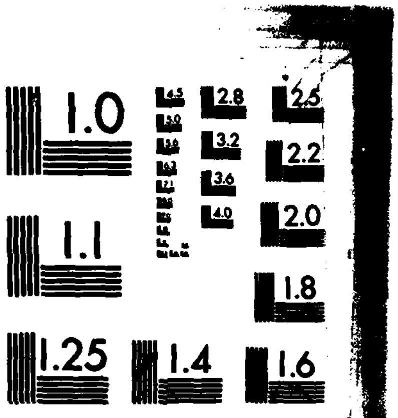

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Calculation of Oxidation Ditch Volume for Aerobic Zone (for BOD Removal and Nitrification):

$$V = \frac{0.8 \times 1(250-5.9) \times 32}{0.8 \times 4000\ (1 + 0.05 \times 32)}$$

$$= 0.75 \text{ million gallons}$$

Calculation of Oxidation Ditch Volume Required for Anaerobic Zone (Denitrification Zone):

Nitrogen in = 25 mg/L x 1 mgd x 8.34 = 208.5 lb/day

Effluent N = 8 mg/L x 1 mgd x 8.34 = 66.7 lb/day

Amount of wasted sludge, P_x

P_x = 0.8 x 1 x(250-5.9) x 8.34 - 0.75 (million gallons) x 0.8 x .05 x 4000 mg/L x 8.34
= 627.8 lb/day

Nitrogen in wasted sludge = 0.12 x 627.8
= 75.3 lb/day

Denitrification requirement = 208.5 - 66.7 - 75.3
= 66.5 lb/day

Denitrification rate given = 0.6 mg NO_3-N/g MLSS-hr
= 0.0144 lb N/lb MLSS-day

Mass of MLSS/million gallons = 8.34 x 1 x 4000
= 33,360 lb MLSS/million gallons

$$\text{Volume for denitrification} = \frac{66.5 \text{ lb/day}}{0.0144 \text{ lb/lb MLSS-day}} \times \frac{1 \text{ million gallons}}{33360 \text{ lb MLSS}}$$

= 0.1384 million gallons (138,400 gal)

Calculation of Total Volume of Oxidation Ditch:

Total volume = 0.75 + 0.1384 million gallons
= 0.89 million gallons or 890,000 gallons, using a concentric arrangement with a circular clarifier in the center.

$$\text{Hydraulic detention time} = \frac{0.89 \text{ million gallons}}{1.0 \text{ mgd}} \times 24 \text{ hr/day}$$

= 21.4 hr.

Calculation of Final Clarifier Diameter:

Using overflow rate of 450 gpd/sq ft

$$A = \frac{1 \text{ mgd}}{450 \text{ gpd/sq ft}} = 2{,}222 \text{ sq ft}$$

(Based on solids loading with the same Q_R(0.34 mgd) as in the vertical-shaft design example.)

$$A = \frac{4000(1 + 0.34)(8.34)}{15 \text{ lb/sq ft-day}}$$
$$= 2980 \text{ sq ft} > 2222 \text{ sq ft}$$

Area chosen = 2980 sq ft

Use one clarifier:

$$\text{Diameter, } D = \left(\frac{2980 \times 4}{\pi}\right)^{1/2}$$
$$= 61.6 \text{ ft.}$$

Use size of 62 ft.

Configuration of Oxidation Ditch:

Use 20-ft channel depth

$$\text{Area required} = \frac{890{,}000 \text{ gal}}{7.48 \times 20}$$
$$= 5950 \text{ sq ft}$$

$$\text{Area total (ditch + clarifier)} = \frac{\pi(62)^2}{4} + 5950$$
$$= 8970 \text{ sq ft}$$

$$\text{Outer diameter} = \left(\frac{8970 \times 4}{\pi}\right)^{1/2}$$
$$= 106.9 \text{ ft}$$

Use 107-ft outer channel diameter around a 62-ft clarifier (Figure 27) (wall thickness has to be added).

Calculation of Oxygen Requirement:

$$\begin{aligned} \text{Total } O_2 \text{ requirement} &= O_2 \text{ demand for BOD removal} \\ &+ O_2 \text{ demand for nitrification} \\ &- O_2 \text{ demand for denitrification} \end{aligned}$$

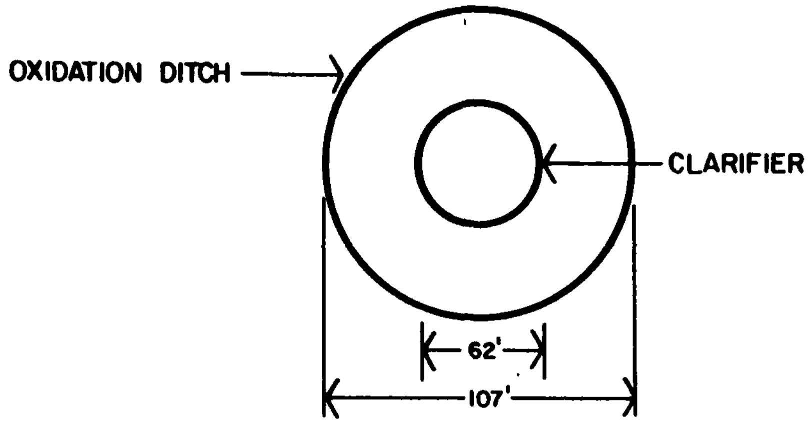

Figure 27. Channel diameter around clarifier.

$$O_2 \text{ demand for BOD} = \frac{1(250-5.9)8.34}{(1-e^{-0.2} \times 5)} - 1.42(627.8)$$
$$= 2329.5 \text{ lb/day}$$

NH_3-N oxidized = influent NH_3-N - effluent NH_3-N - NH_3-N in wasted sludge

$$= (25 \times 8.34 \times 1) - (1 \times 8.34 \times 1) - 75.3$$
$$= 124.9 \text{ lb/day}$$

$$O_2 \text{ demand for nitrification} = 4.5(124.9) = 562 \text{ lb/day}$$

$$O_2 \text{ demand for denitrification} = 2.6(66.5 \text{ lb/day})$$

$$= 172.9 \text{ lb/day}$$

Therefore, total O_2 requirement = 2329.5 + 562 - 172.9 = 2718.6 lb/day

Convert to standard conditions:

$$O_2 \text{ demand} = \frac{2718.6}{0.92\left(\frac{0.97 \times 8.38 - 2}{9.17}\right)(1.024)^5}$$
$$= 3926.4 \text{ lb/day}$$

Air requirement:

$$= 3926.4 \frac{\text{lb}}{\text{day}} \times \frac{1 \text{ lb air}}{0.232 \text{ lb } O_2} \times \frac{1 \text{ scf}}{0.075 \text{ lb}} \times \frac{1 \text{ day}}{1440 \text{ min}} \times \frac{1}{0.22}$$
$$= 712 \text{ scfm.}$$

$$\text{Bhp of jet aerator} = \frac{3926 \text{ lb/day}}{24 \text{ hr/day} \times 3.5 \text{ lb/hr-hp}}$$
$$= 46.7 \text{ hp or } 50 \text{ hp.}$$

Calculation of Return Sludge Flow Rate, Q_R:

Identical to vertical-shaft design example.

$Q_R = 0.34$ mgd

Choose a sludge pump capable of pumping 0 to 1.0 mgd.

Calculate Amount of Sludge for Disposal:

Total sludge = inert influent TSS + wasted sludge P_x − TSS effluent

$= 50 \text{ mg/L} \times 1 \times 8.34 + 627.8 \text{ lb/day} - 20 \text{ mg/L} \times 1 \times 8.34$

$= 878$ lb/day dry solids

At 1.5 percent concentration

$$\text{Flow rate} = \frac{878}{0.015 \times 8.34} = 7018 \text{ gpd}$$

To maintain 3 fps in a 4-in.-diameter sludge line

$Q \approx 120$ gpm

$$\text{Schedule of wasting} = \frac{7018}{120} = 58.5 \text{ min.}$$

One pump at 120 gpm capacity operating 60 min/day.

Sizing of Sludge Drying Beds:

Identical to vertical-shaft design example

10 drying beds, each 35 ft x 35 ft

Total area = 12,250 sq ft

Table 28 summarizes the design criteria for a deep channel oxidation ditch with jet aeration.

Cost Estimates

This section gives cost estimates for the design examples discussed on pp 68-96. Equipment and installation costs were furnished by manufacturers or equipment suppliers in May 1982. For reinforced concrete work, \$175/cu yd and excavation costs of \$1.75/cu yd more accurately reflect the costs of the

Table 28

Design Summary of Deep Channel Oxidation Ditch with Jet Aeration

Quantity	Unit	Size
1	Circular channel around a final clarifier in the center. Vertical walls, reinforced concrete construction.	Outer diameter, 106.9 ft Inner diameter, 62 ft Depth 20 ft
1	Jet aerator	50 Hp
1	Final clarifier	62-ft diameter
1	Return sludge pump	0 to 1.0 mgd capacity
1	Sludge wasting pump	120 gpm capacity
10	Sludge-drying bed	35 ft x 35 ft each
1	Dissolved oxygen control system for maintaining both the aerobic and the anoxic zones in the channel.	

northeastern region of the United States. The cost of gunite or shotcrete with wire mesh for ditch lining (shallow channel) is assumed to be \$5.50/sq ft. Land cost is not considered. The cost estimates are for the biological treatment portion of the plant only (oxidation ditch, final clarifier, and sludge drying beds) and do not include costs for preliminary treatment (screening and comminutor) or disinfection.

Carrousel System

Oxidation ditch, site preparation, and excavation -- 5275 cu yd x \$1.75/cu yd	=	\$9,230
Reinforced concrete walls and bottom, with foundation -- 515 cu yd x \$175/cu yd	=	\$90,125
2 aerators, 25 hp each \$ 30,000, plus 20 percent installation cost	=	\$72,000
1 variable height overflow weir installation cost	=	\$7,200
2 final clarifiers, 45-ft diameter x 15 ft SWD each, plus 30 percent installation cost	=	\$241,485
2 return sludge pumps, 175 gpm each, plus 20 percent installation cost	=	\$7,567
2 waste sludge pumps, 120 gpm each, plus 20 percent installation cost	=	\$7,075
10 sludge drying beds, 35- x 35-ft each	=	\$45,000
Piping, valves, and walkway	=	\$20,000
		\$499,682

Table 28 (Cont'd)

Oval Ditch

2 oxidation ditches, site preparation, and excavation, 2765 cu yd x $1.75/cu yd	=	$4,840
Shotcrete lining, 45-degree side wall, and bottom, 17,600 sq ft x $5.50/sq ft	=	$96,800
4 aerators, 42-in.-diameter MAGNA, 14-ft length each, 16 Bhp (20-hp) motor) $16,150 each, plus 20 percent installation cost	=	$77,520
2 adjustable overflow weirs, each 12 ft long, $3,600 plus 10 percent installation cost	=	$7,920
2 final clarifiers, each 38-ft diameter x 8-ft SWD, $28,300 plus 30 percent installation cost	=	$102,119
2 return sludge pumps, 175 gpm each, plus 20 percent installation cost	=	$7,567
2 waste sludge pumps, 120 gpm each, plus 20 percent installation cost	=	$7,075
10 sludge-drying beds, 35- x 35-ft each	=	$45,000
Piping, valves, and walkways	=	$20,000
		$368,841

Jet Aeration, Deep Channel System

Oxidation ditch, site preparation, and excavation, 8350 cu yd x $1.75/cu yd	=	$14,513
Reinforced concrete walls, bottom with foundation, 780 cu yd x $175/cu yd	=	$136,500
1 jet aspirator aeration system, 50 hp	=	$80,000*
1 final clarifier, 63 ft-diameter x 20-ft SWD, plus 30 percent installation	=	$195,000
1 return sludge pump, 350 gpm	=	$5,814
1 sludge wasting pump, 120 gpm	=	$3,537
10 sludge drying beds, 35- x 35-ft each	=	$ 45,000
Piping, valves, and walkways	=	$ 20,000
		$500,364

Comparison of Oxidation Ditch Systems

Table 29 summarizes the salient features of the three oxidation ditch designs and their cost estimates. The oval ditch system is the most inexpensive because of its construction method (shallow ditches using a 45-degree slope and a 4-in. gunite lining). The final clarifier size is smaller for this system, following the design procedure suggested by Lakeside Equipment Corporation. No solid loading is considered in sizing the clarifier with this procedure.

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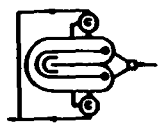
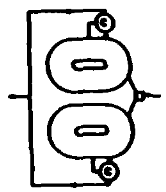
*Price for Jet Aspirator system (estimated at 50 Hp).

Table 29

Comparison of Oxidation Ditch Systems
1.0 MGD Capacity

Design	Configuration *	Design Requirement	Channel Liquid Volume (ft^3)	Liquid Depth (ft)	Hydraulic Detention Time (hr)	Sludge Retention Time (days)	Total* Installation Cost ($)
Carrousel System		BOD Removal Nitrification	95,000	10	17.1	32	499,682
Oval Ditch		BOD Removal Nitrification	104,250	10	18.75	16	368,841
Deep Channel Aeration		BOD Removal Nitrification Denitrification	119,459	20	21.4	32	500,364

c = final clarifier

*Includes oxidation ditch, clarifier(s), aerators, sludge return pumps, sludge wasting pumps, sludge-drying beds, valves, and walkways all installed.

The deep channel jet aeration system provides denitrification. This function does not add much to the system installation cost for any system operated in the extended aeration mode. The system eliminates the need for a variable height overflow weir, since aeration is controlled directly by the jet aspirator system. The cost of this system is comparable to that of the Carrousel system. It should be noted, however, that the cost of an automatic aeration control, using DO probes, is not included, since some plants may prefer manual control in nitrification-denitrification zones.

Operation and Maintenance

Oxidation ditch plants are conservatively designed to provide up to 24 hours of hydraulic detention based on daily average flow. A long solid retention time (sludge age) of 20 to 30 days is normally required to provide the stability needed to remove BOD and provide nitrification. The following guidelines are suggested for oxidation ditch plant operation and maintenance. (A section of Army TM 5-665,[7] Operation and Maintenance of Domestic and Industrial Wastewater Systems, which pertains to oxidation ditches, is reprinted in Appendix B and provides additional guidance on the basic horizontal rotor configuration.)

Maintaining the MLSS Concentration Within a Proper Range

Most design engineers recommend an MLSS concentration of 4000 to 8000 mg/L. The optimal range for operation is site-specific and depends on the expected range of BOD loadings. Loadings of 8.6 to 15 lb BOD/1000 cu ft aeration volume-day are common. Because of the long hydraulic and solid retention time (a large amount of biomass in the channel), fluctuation in flow and MLSS concentration are not critical for the operation as long as the MLSS is not far from the optimal range. Old sewer lines which would allow significant stormwater infiltration would wash out the solids in the channel. The operator should therefore determine the MLSS concentration and return more sludge to maintain the optimal range concentration. Normally, MLSS should be determined daily.

Solid Inventory

Oxidation ditch plants which provide significant nitrification or nitrification-denitrification should maintain a longer solid retention time, preferably 30 days or more. Sludge retention time should be determined twice a week by dividing the biomass in the ditch on dry weight basis (volume of liquid x MLSS concentration) by the rate of sludge wasting on a dry weight biomass wasting per day basis (volume of wasting sludge per day x MLSS of wasting sludge).

If an intrachannel clarifier is used, no solid inventory is needed. The sludge retention time is hydraulically controlled, since the mixed liquor MLSS concentration is the same as the wasting sludge concentration. The sludge retention time is determined by dividing the channel liquor volume by the flow rate of the wasting sludge.

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[7] Operation and Maintenance of Domestic and Industrial Wastewater Systems, Technical Manual 5-665 (Department of the Army, January 1982).

pH and Alkalinity

Some low-alkalinity wastewater may require pH adjustment when subjected to extended aeration with nitrification. The operator should add alkalinity to the wastewater to a pH of between 8.0 to 8.4 to maximize nitrification. Determination of pH should be conducted daily and more often if enough manpower is available.

Provide Adequate Mixing and Oxygenation

The design engineer selects the proper size of aerator(s) for adequate mixing and oxygenation to accommodate the peak design flow. For a surface aerator (either horizontal or vertical shaft), the mixing and oxygenation provided depends on aerator submergence, which depends on the liquid level in the channel. Many plants find that while the mixing is adequate (through occasional checking of surface liquid velocity at 1.0 fps or above), the aerators provide more oxygen than required to meet the BOD and nitrification demand. Thus, operators can turn off the aerator(s) periodically and still provide adequate mixing and maintain a minimum of 1 to 2 mg/L DO concentration in the mixed liquor. However, operators will still probably have to monitor the DO concentration frequently and determine the frequency and duration of the on-off cycles by experimenting.

If mixing or oxygenation is insufficient, the aerators usually can be lowered to increase submergence. It is important that the motor(s) have enough horsepower for aerator operation at the greater submergence. The same applies to changing the aerator speed to provide better mixing and oxygenation. Similar controls can be exercised for the induction aerator or jet aeration (aspirator aeration). With diffused bubble aeration plus a slow mixer, the operator can vary the amount of air supply by controlling the degree of oxygenation.

Denitrification requires an anoxic zone having little or no oxygen. Unless an automatic control system with a DO probe and controlled-aerator speed is used, the operator must monitor the DO much more closely to insure the presence of an anoxic zone which will be sufficient for successful denitrification.

Clarifier Operation

When washout of MLSS from the oxidation ditch plant occurs, or when the MLSS concentration in the channel becomes too high, the clarifier may be overloaded (solid loading greatly exceeds the designed loading value). A sludge blanket depth finder or a "sludge judge" should be used daily to make sure that the sludge zone remains a safe distance from the liquid surface. More sludge must be wasted to keep the rising sludge blanket down. On the other hand, when a sludge blanket is not well established, there may not be enough sludge in the clarifier. The sludge wasting schedule must then be reduced to reestablish the blanket so that solid removal can be effective.

Plant Services and Maintenance Program

Oxidation ditch plants have less mechanical equipment and are simpler to operate than competitive treatment plants; however, they must be serviced and maintained regularly to guarantee steady performance.

Comminutors are a continuing maintenance problem, requiring regular cleaning and care. The wastewater should be diverted to a bar screen when they are serviced. Unscreened sewage will cause problems in the return sludge system by clogging the rate adjustment valve (generally a telescoping valve) or pumps.

Brush-type aerators may lose some "teeth" due to corrosion of bolts or may be damaged by ice jamming in very cold weather. Periodic shutdown is required for repair. Aerators may also loosen from the shafts and have to be shimmed and reclamped periodically. Gear reducer output shaft seals may need replacing about once a year.

Regular service and maintenance for all mechanical equipment recommended by the manufacturer should be provided. All equipment, including clarifiers, should be drained and inspected annually. Any corrosion should be removed and the area recoated.

Instrumentation required for proper control of oxidation ditch plants includes: raw sewage or effluent flow measurement, recording and totaling; return sludge measurement and secondary; waste sludge flow measurement and recording; chlorine feed proportional to flow; dissolved oxygen measurement; recording and aerator control; and normal laboratory instrumentation.

In summary, overall control of an oxidation ditch plant depends on biomass control and control over dissolved oxygen levels.

Operator Training

Since oxidation ditch technology is relatively new to Army installations, very few operators will have the proper training and experience to run them. However, operation contributes more to the success or failure of a plant than does design. Training can start with the operation and maintenance manual prepared by the design engineer. However, most of these manuals simply follow the outline recommended by the EPA and are of little practical value, since specific information is hard to find. Equipment suppliers also provide information on start-up and operation. The information on operational control is based on empiricism and observation. Thus, the best training program is a "hands-on" operator training approach.

Before an oxidation ditch plant goes on-line, the chief operator and other operators, if possible, should visit a similar oxidation ditch plant in the same region. Depending on the technical capability of the operator, 1 to 2 weeks of hands-on operational training may be required. The visits can occur at suitable intervals and be augmented by frequent telephone conversations. Each visit should focus on some facet of operation such as:

Laboratory procedure and field tests for monitoring and recordkeeping.

Sludge wasting and methods for determining the amount to be wasted.

Service and maintenance of the aerator and other mechanical equipment.

Clarifier operation and sludge blanket control.

Sludge transfer to drying beds.

Generally, demonstrations and instructions should be scheduled to allow each operator to learn at his/her own pace. Experience has shown that one day of supervised, hands-on instruction is worth many days of instruction in formal workshops and many more days of unsupervised work. The hands-on training costs for the chief operator should be only a few percentages of one year's total outlay for operation and amortization.

6 CONCLUSIONS

Oxidation ditches operate reliably and well with a small number of relatively unskilled operators. They are flexible in their operation, capable of performing nitrification and denitrification, and can oxidize organics. They do not require a primary clarifier, and they produce a stabilized sludge which needs no further treatment. They have few moving parts which could wear out or fail. Finally, within the flow range of 0.15 to 10 mgd of sewage, oxidation ditches have lower capital and O&M costs than any other biological treatment technology.

The guidance provided in this report for selecting, designing, operating, and maintaining oxidation ditch facilities will be useful to Facilities Engineering personnel, District design engineers, and Architect/Engineering firms involved with implementation of Army pollution abatement plans.

CITED REFERENCES

Adema, D., "The Largest Oxidation Ditch in the World for the Treatment of Industrial Wastes," Proc. 22nd Purdue Ind. Waste Conference (May 1967).

Applegate, C. S., et al., "Total Nitrogen Removal in a Multi-Channel Oxidation System," J.W.P.C.F., Vol 52 (1980), pp 568-577.

Aspirating Aerators Technical Bulletin (Pentech Houdaille).

Aufiers, F. G., "Advanced Wastewater Treatment: Stowe, Vermont," Journal New England Water Pollution Control Association, Vol 15, No. 2 (1981), pp 149-151.

Boettcher, G., "A Construction Grants Program Science Story," Public Works, 1981, pp 35-37.

Bulletin RAD-390 (Lakeside Equipment Corp.).

Cerwick, J. A., "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan (October 1981).

Costly Wastewater Treatment Plants Fail to Perform as Expected (General Accounting Office, 1980).

Cox, G. C., et al., "Production of High Quality Effluents Using Carrousel and High Rate Filtration Processes," Colloque International, Traitement Combine Des Eaux Usees Domestiques et Industrielles (16-19, Mai, 1978).

Drews, R. J. L. C., et al., "The Orbal Extended Aeration Activated Sludge Plant," J.W.P.C.F., Vol 44, No. 2 (1972), pp 221-231.

Efficient Treatment of Small Municipal Flows at Dawson, Minn., EPA Technology Transfer, Capsule Report, 625/2-77-015 (USEPA, 1977).

Envirotech Bulletin (Envirotech Corporation).

Ettlich, W. F., A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051 (USEPA, 1978).

Jacobs, A., "Looped Aeration Tank Design Offers Practical Advantages, Parts I and II," Water and Sewage Works (1975).

Jones, D. D., et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," Proc. 24th Purdue Ind. Waste Conf., Part I (1969).

Kaneshige, H. M., "Performance of the Somerset Ohio Oxidation Ditch," Jour. Water Pollution Control Federation, Vol 42 (1970), pp 1370-1378.

Keffer, W. J., Winter Performance of Secondary Wastewater Treatment Facilities (USEPA, Region VII, 1977).

Mandt, M. G. and B. A. Bell, Oxidation Ditches in Wastewater Treatment (Ann Arbor Science, 1982).

Metcalf & Eddy, Nitrification and Denitrification Facilities, Wastewater Treatment (USEPA, 1973).

Operation and Maintenance of domestic and Industrial Wastewater Systems, Technical Manual 5-665 (Department of the Army, January 1982).

Stensel, H.D., et al. (Envirotech), "Cost Effective Advanced Biological Wastewater Treatment Systems," paper presented at the Nevada Water Pollution Control Association (1977).

Viranaghavan, T., et al., "Oxidation Ditch Plus Alum Take Phosphorus Away," Water and Sewage Works (1979), pp 54-56.

Wastewater Treatment Plant Design (USEPA, 1977).

Wiskow, R., "Oxyditch Concept Proven in Pioneer Municipal Plant," Public Works (April 1979).

UNCITED REFERENCES

Beer, C., "Looped Reactors for the Activated Sludge Process," Clear Waters (September 1978), pp 22-27.

Bogen, S. A., "Eighteen Months of Nuisance-Free Operation," The American City (June 1971), pp 1-3.

Chambers, J. V., "Improving Waste Removal Performance Reliability of Wastewater Treatment System Through Bioaugmentation," Proc. 36th Purdue Industrial Waste Conference (1981), pp 631-643.

Chong, G. M. W., et al., "Comparison of the Conventional Cage Rotor and Jet-Aero-Mix Systems in Oxidation Ditch Operations," Proc. 28th Purdue Industrial Waste Conf. (May 1973), pp 713-723.

Golding, R. F., et al., "The Pentech JAC Oxyditch™," paper presented at the 52nd Annual Meeting, Central States, WPC Association, Inc. (May 1979).

Innovative and Alternative Technology Assessment Manual, CD-53, 430/9-78-009 (U.S. Environmental Protection Agency [USEPA], 1980).

Innovative Technology, Treatment Process with Intrachannel Clarification Device (Burns & McDonnell, 1982).

Johnson, H., "New Solutions to Plant Problems," Food Processing (August 1976), p 42.

Johnstone, D. W. M., et al., "Settlement Characteristics and Settlement Tank Performance in the Carrousel System," paper presented at London for the U.K. Institute of Water Pollution Control (1978).

Lecompte, A. R., et al., "A Pasveer Ditch, American Jet Style," Proc. 29th Purdue Ind. Waste Conference (1974), pp 631-639.

Lecompte, A. R., Deep Channel Secondary Effluent Treatment (Kimberly-Clark Corp, 1981).

"Oxidation Ditches Perform Well in Winter," J.W.P.C.F., Vol 49 (April 1977), p 537.

Sanks, R. L., et. al., "Operator Training is Key to Oxidation Ditch Start-up and Operation," J.W.P.C.F., Vol 53 (1981), pp 444-450.

METRIC CONVERSION TABLE

1 mgd	= 3788 m^3/day
1 ft	= .3048 m
1 in.	= 25.4 mm
1 gal	= 3.785 L
1 sq ft	= .0929 m^2
1 lb	= .4539 kg
1 gpd/sq ft	= .04 m^2/day
1 cu ft	= .0283 m^3
1 cu yd	= .7645 m^3
1 cu ft/sec	= .0283 m^3/sec
1 ft/sec	= .3048 m/s
^oF	= (^oF - 32)(5/9)
1 kWh/million gal	= 1 kWh/3788 m^3
1 kWh/lb	= 1 kWh/.4539 kg

APPENDIX A: LITERATURE REVIEW AND OXIDATION DITCH PLANT VISITS

This appendix provides a review of literature dealing with the application of oxidation ditch technology to wastewater treatment. Design criteria, operational and maintenance requirements, system performance responding to steady and fluctuating loadings, and system cost-effectiveness are reviewed. In addition, information about oxidation ditch treatment systems at different locations which were contacted or visited is presented.

Literature Review

Adema, D., "The Largest Oxidation Ditch in the World for the Treatment of Industrial Wastes," Proc. 22nd Purdue Ind. Waste Conf. (May 1967).

A large Pasveer ditch was put into operation in 1964 to treat the wastes from the Maunits coking plant, the Emma coking plant, and several chemical plants preparing polyethylene and formaldehyde. The retention time was fixed at 3 days and the capacity of the ditch at 30,000 m^3. The total length of the aerators was 100 m, and was later increased to 250 m to handle discharge of organic waste from 200,000 to 300,000 mg/L chemical oxygen demand (COD).

The low BOD concentration of the effluent shows that the purification efficiency was at least 99 percent for the degradable substances. Nitrification was poor because of the lack of alkalinity. Shortage of phosphorus in the industrial waste caused slime growth with very poor settling characteristics. The problems were overcome by adding alkalinity and phosphorus to the wastes. Table A1 summarizes data on the Pasveer ditch and its associated costs.

Jones, D. D., et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," Proc. 24th Purdue Ind. Waste Conf., Part I (1969).

The oxygenation capacities of five aerator rotors were tested. Figures A1 through A5 show the results, which are for standard conditions of temperature and pressure and zero DO in the tap water being tested. A beta value of 0.95 and an alpha value of 0.8 would be appropriate for application to livestock waste treatment in an oxidation ditch.

Under the standard conditions, the rotor adds from 1.47 to 1.6 lb O_2/hr-ft of rotor. The oxygenation capacity is increased linearly with the inversion or with the rotor speed.

Kaneshige, H. M., "Performance of the Somerset Ohio Oxidation Ditch," Jour. Water Pollution Control Federation, Vol 42 (1970), pp 1370-1378.

The Somerset plant consists of two concrete-lined oxidation ditches, each having a volume of 208,000 gal. At the time of the study, only one ditch was needed. Each ditch has two cage rotor aerators, each 27.5 in. in diameter and 8 ft long. Two final settling tanks follow the ditches. Sludge is returned to the ditch by air lift. Waste sludge goes to sand drying beds. Two pumps, each with a capacity of 300 gpm, lift the raw wastewater to the comminutor.

Table A1

Data on the Pasveer Ditch

(From D. Adema, "The Largest Oxidation Ditch in the World for the Treatment of Industrial Wastes," Proc. 22nd Purdue Ind. Waste Conference [May 1967].)

Dimensions:

Capacity:	30,000 cu m	(1 x 10^6 cu ft)
Depth:	1.60 m	(5 ft)
Width:	25 m	(82 ft)
Developed Length:	800 m	(0.5 mile)

Walls protected with mine refuse, bottom unprotected.

Brushes:

Make: Passavant
10 rows of aeration rotors, each wide 2 x 12.5 m (2 x 41 ft)
Total length: 250 m (820 ft)
Speed: 80 rpm
Pockwood bearings, splash lubrication
Maximum power input: 800 HP

Settling Tank:

Diameter: 28 m (92 ft)
Surface area: 615 sq m (6600 sq ft)
Capacity: 920 cu m (32,500 cu ft)
Sludge return pumps: 2 x 200 cu m/h. To be supplemented with 1 x 400 cu m/h (2 x 7100 cu ft).

The plant is at a distance of two to three km from the nearest production unit. There is no operator at the site. Failure of motors, etc., can be observed on an instrument panel at Emma coking plant. A small stream of purified wastewater is continuously pumped to the coking plant laboratory.

Investment:

Buildings	US $180,000
Equipment	200,000
Conduits	80,000
Electrical equipment	60,000
Instrumentation	15,000
Miscellaneous	5,000
TOTAL	US $540,000

The item "Conduits" relates to the sludge return line, water pipes for lubricating the brushes (no longer in operation), an excessive sludge line, and a sampling line (each of the latter two having a length of two to three km).

Not included are two feed lines:

from Maurits coking plant to Emma coking plant	US $40,000
from the Organic Products plants to Emma coking plant	50,000
TOTAL	US $90.000

The figure for "Electrical equipment" is rather high, because of the large distance between the ditch (cables) and the nearest production unit.

With 400,000 population equivalents the investment is only about $1.40/PE.

		U.S. $	
Annual Cost	1964	1965	1966
Power	23,000	40,000	32,000
Phosphorus	4,000	4,000	8,000
Operation + Supervision	4,000	8,000	9,000
Maintenance	9,000	36,000	48,000
Depreciation	33,000	55,000	54,000
Miscellaneous	17,000	22,000	26,000
TOTAL	90,000	165,000	177,000
Average amount of COD, kg/h	260	900	780
Cost per kg of COD	0.06	0.02	0.03
Cost per cu m of wastewater	0.03	0.04	0.05

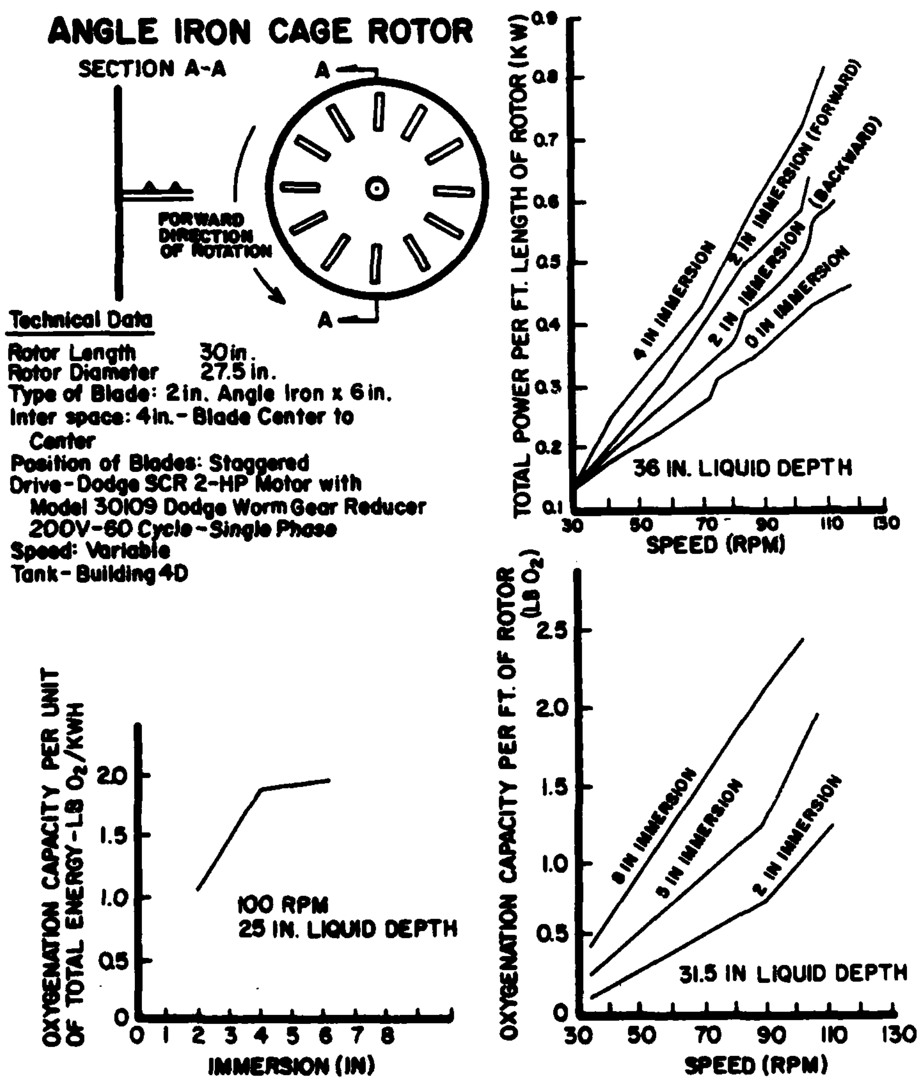

Figure A1. Oxygenation capacity and energy consumption of an angle iron bladed cage rotor. (From D. D. Jones, et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," Proc. 24th Purdue Ind. Waste Conf., Part I [1969].)

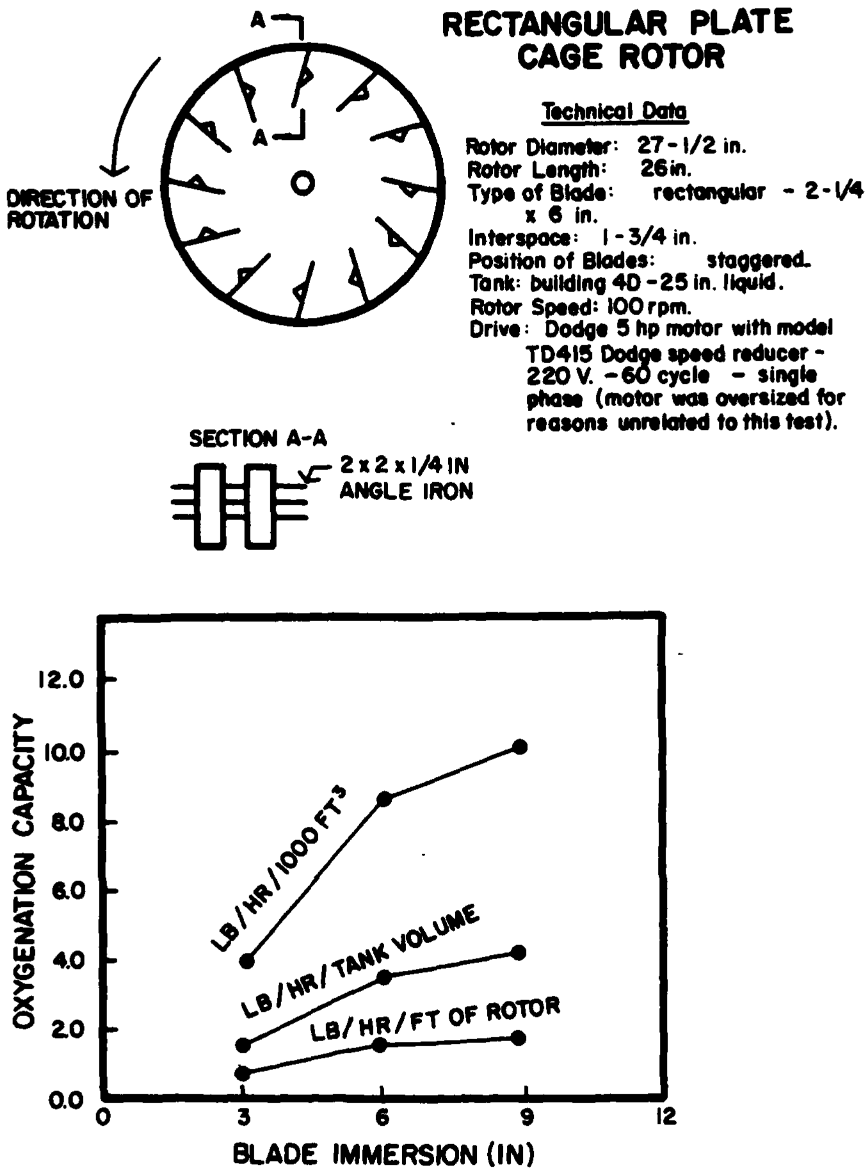

Figure A2. Oxygenation capacity of a rectangular plate blade cage rotor. (From D. D. Jones, et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," Proc. 24th Purdue Ind. Waste Conf., Part I [1969].)

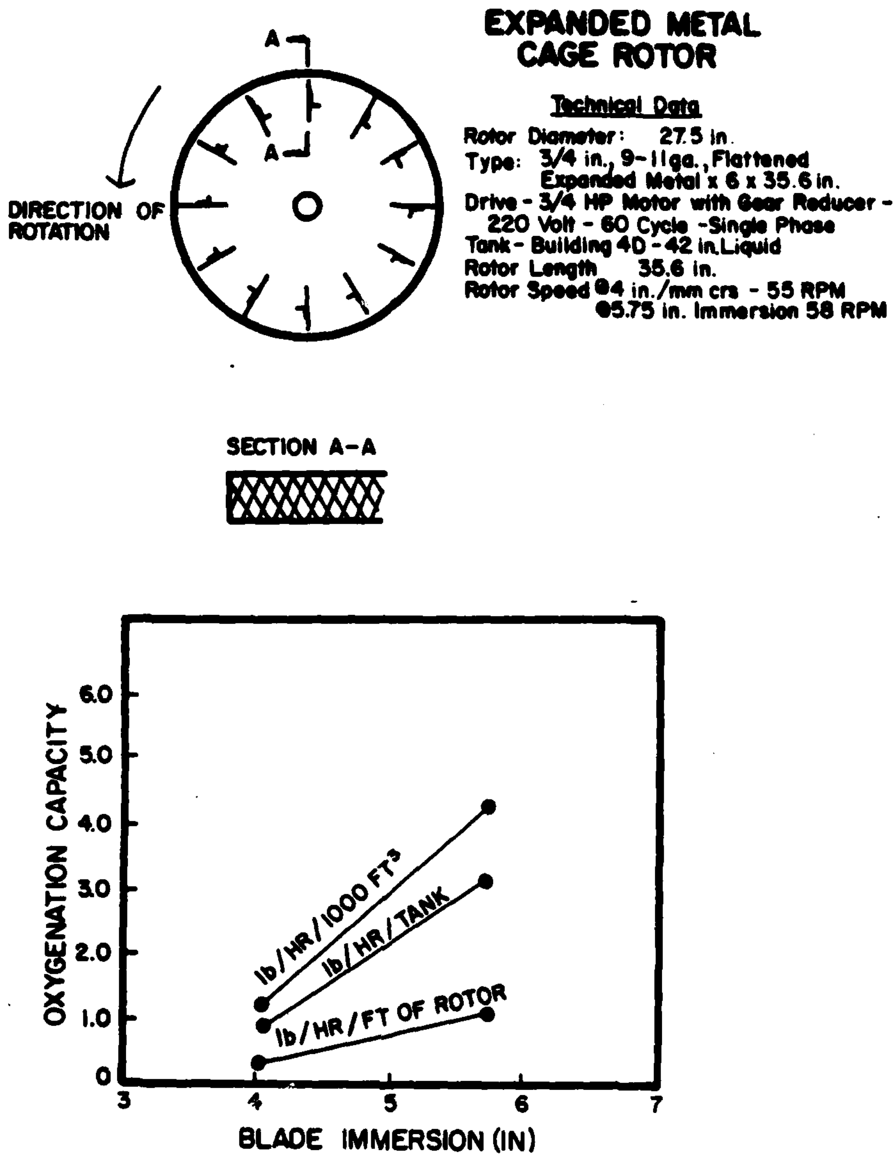

Figure A3. Oxygenation capacity of an expanded metal cage rotor. (From D. D. Jones, et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," <u>Proc. 24th Purdue Ind. Waste Conf., Part I</u> [1969].)

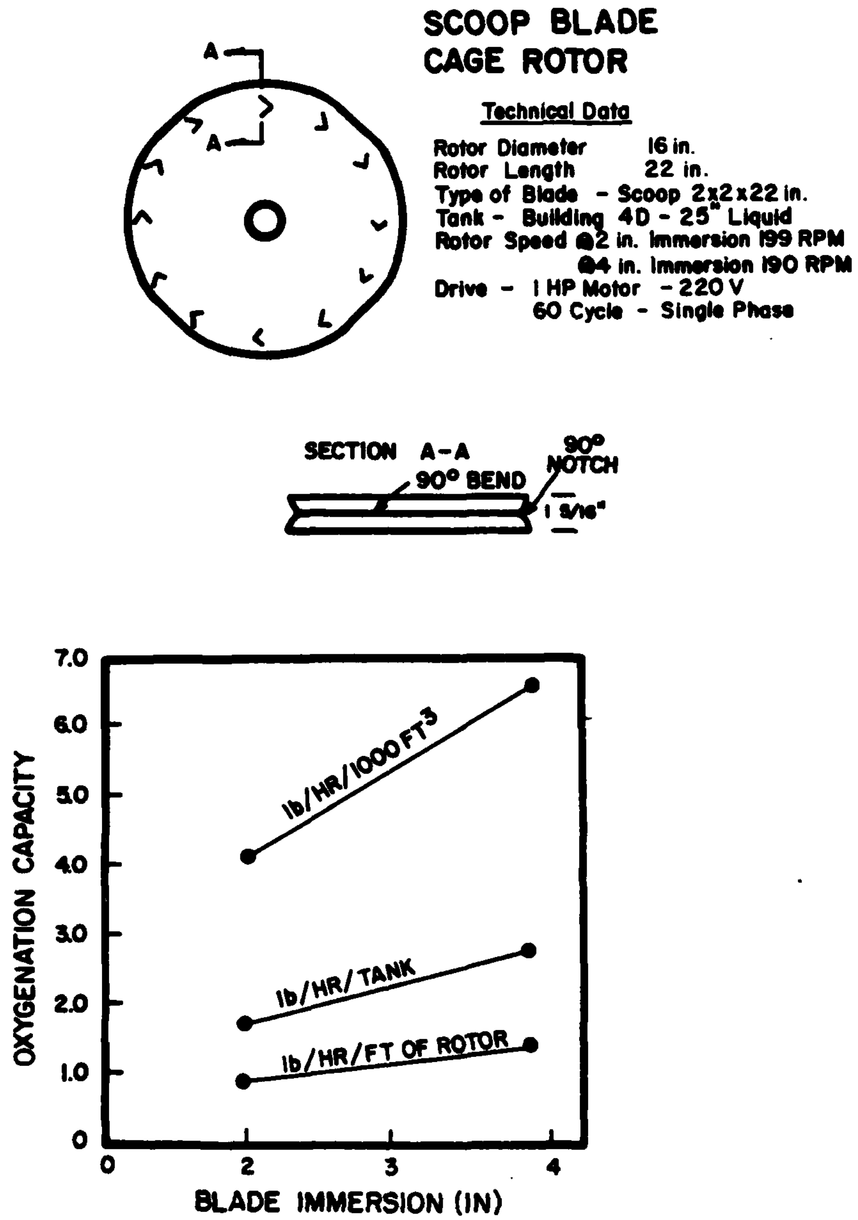

Figure A4. Oxygenation capacity of a scoop type bladed rotor. (From D. D. Jones, et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," Proc. 24th Purdue Ind. Waste Conf., Part I [1969].)

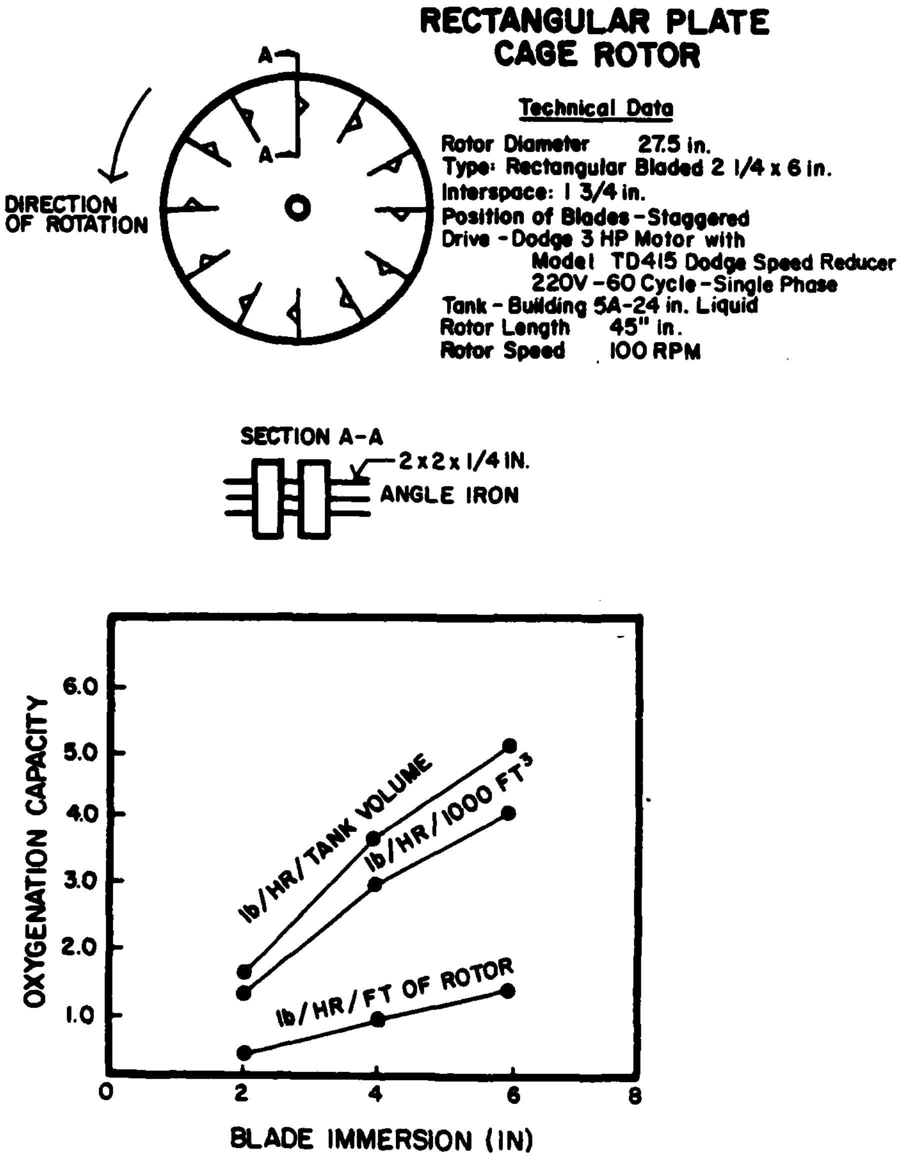

Figure A5. Oxygenation capacity of a rectangular plate blade cage rotor. (From D. D. Jones, et al., "Oxygenation Capacities of Oxidation Ditch Rotors for Confinement Livestock Buildings," <u>Proc. 24th Purdue Ind. Waste Conf., Part I</u> [1969].)

The plant serves a population of about 1500, and its influent is predominantly domestic waste with a slug contribution from a small slaughterhouse once a week (average four hogs and five beef cattle per week).

The average velocity in the channels increases from 1.55 fps at 4 in. immersion to 1.65 fps at 6 in. to 1.92 fps at 9 in. The velocities are distributed so that the higher values are near the water surface and slightly off center toward the outer side of the ditch cross section. Table A2 shows the average performance of the oxidation ditch.

The design BOD loading criterion used for this plant was 12.5 lb/day-1000 cu ft of ditch volume. Thus, the plant has often been operating at about half of its design load. The plant has achieved an overall excellent removal of BOD and suspended solids. Average power consumption for all uses at the plant was:

1966-67 3,010 kWh/million gallons 1.74 kWh lb/BOD removed

1967-68 3,550 kWh/million gallons 1.83 kWh/lb/BOD removed

1968-69 4,720 kWh/million gallons 2.32 kWh/lb/BOD removed

At an average rate of about $0.02/kWh, the cost of the electrical energy ranged between $60 and $94/million gallons of wastewater treated. This is less than the average and median power costs of $111 and $104/million gallons, respectively, reported for 14 extended aeration plants.

The design BOD loadings were exceeded several times. Excellent BOD removals were still obtained, even with an overload of about 40 percent, if the detention time in the ditch was longer than 18 hr and the flow did not occur in surges. Flow surges did flush out sludge and occasionally caused high-effluent BOD. One high-effluent BOD was caused by a plastic sheet fouling the airlift return sludge pump; this impaired removal of the sludge, which rose by means of gas bubbles during denitrification in the final tank. One extremely cold week (averaging 15°F or -9°C) with low flows caused ice formation on the surface of the final tanks and impaired settling. The subsequent loss of sludge affected treatment performance.

The drying rate of the waste sludge placed on the sand beds was recorded sporadically during the 3-year period. The few observations recorded indicate that during the summer and early fall, the sludge was dry enough for removal from 5 to 20 days after application. Depths of the freshly applied sludge varied from about 6 to 12 in. During the summer, 5 to 6 days were enough when rain did not hinder the drying process. Longer drying times were almost always associated with large rainfalls. For example, for one batch that required 18 days to dry, rainfalls of 0.2, 0.3, 0.4, and 0.5 in. occurred on the fourth, seventh, eighth, and fifteenth days. For another batch that required 19 days to dry, 3.1 in. of rain fell between the 3rd and 13th days of the drying period.

Bogen, S. A., "Eighteen Months of Nuisance-free Operation," The American City (June 1971), pp 1-3.

Table A2

Average Performance of the Somerset Plant Oxidation Ditch
(From H. M. Kaneshige, "Performance of the Somerset Ohio Oxidation Ditch," Jour. Water Pollution Control Federation, Vol 42 [1970], pp 1370-1378.)

	Flow (gpd)	Suspended Solids (mg/L)		BOD (mg/L)		BOD Loading (lb/day/1,000 cu ft)
		Raw	Final	Raw	Final	
1966-67 Avg.	122,000	298	16	215	7	7.0
1967-68 Avg.	105,000	333	68	258	25	7.2
1968-69 Avg.	90,000	290	16	252	8	6.2
3-Yr Avg.	106,000	307	34	241	13	6.8

Note: Gpd x 3.785×10^{-3} = cu m/day; lb/day/1,000 cu ft x 16 = g/day/cu m.

A Pasveer type oxidation ditch plant designed for 2000 residents plus 5600 non-residents was installed at the New York Institute of Technology, Long Island, NY. The plant provided a ditch volume of 58,800 cu ft divided equally into two streams; the volume was based on a maximum BOD loading of 12.5 lb/1000 cu ft and a design flow of 0.34 mgd. Each stream structure was 174 ft long by 84 ft wide, with a center island 128 ft by 38 ft, and an effective depth of 4.0 ft. The 14-ft-long brush aerator with a 15 Hp motor immersed to a depth of 9.0 in. and rotating at 90 rpm maintained a flow velocity of 1.0 fps and 2.0 ppm of DO in all parts of the stream. The aerator was designed for 1.5 lb O_2/lb of BOD removal.

Each of the two clarifiers has a diameter of 27 ft and a depth of 10 ft, substantially exceeding the 3-hr detention during an 18-hr operating day. The whole plant occupies only 200 sq ft, since the clarifiers and sludge holding tanks and drying beds are placed within the islands in the center of the oxidation streams. Only the laboratory, comminutor and flume, and chlorine-contact chamber occupy space outside the streams.

A fiberglass housing is placed over each brush aerator to keep ice from forming on the brushes. Ice can damage the blades, impair their action, or overload the rotors. A gas line is also placed under each hood for installation of infra-red heaters, if necessary.

A totally enclosed gear motor instead of the more customary belt or chain drive is used to rotate the brushes. This keeps the oxidation from being contaminated by lubricants from the rotor drive. However, the gear motor had to be placed on a platform slightly below stream level. So far there have been no problems caused by water splashing on its platform. Another unique feature of the plant is a desudsing spray at the edge of each rotor housing. The simple perforated-pipe arrangement, which is installed by the plant operator, uses supernatant water to combat foam. Since the system was installed, a single operator has performed all tests and maintenance functions.

Drews, R. J. L. C., et al., "The Orbal Extended Aeration Activated Sludge Plant," Jour. Water Poll. Control Fed., Vol 44, No. 2 (1972), pp 221-231.

The orbal extended aeration plant has up to a 2250 m^3/day capacity. The plant is essentially composed of (1) versatile, concentrically arranged, multi-compartment aeration ditches followed by sludge settling tanks, and (2) a trouble-free, efficient aeration disk system. It is meant to handle raw wastewater with only screening and detritus removal as preliminary treatment. Oxygenation efficiency for a 4-ft aeration disk was found to be 1820 g O_2/kWh, and for a 4.5-ft disk at 18 in. immersion was 2000 g O_2/kWh, both at a 52-rpm speed. Figure 7 (p 28) shows the plant layout.

Two plants in South Africa use the orbal oxidation ditch system. Both plants produce effluent quality acceptable to the South African general standards. The Modderfontein Plant (0.6 mgd capacity) treats a combined municipal and chemical factory waste. The total installed aeration capacity consists of six drive shafts; each contains twenty 4-ft-diameter disk aerators and is driven by a 15-hp geared motor. Total mixed liquor aeration volume is 40,000 cu ft, giving an aeration period of 12 hr based on design flow. The South Witbank Mine orbal plant treats exclusively domestic wastes and was designed for a 0.06 mgd capacity. The plant has the usual screens and detritus channels followed by three orbal aeration ditches connected in series. Aeration is supplied by twelve 4.5-ft aeration disks on two shafts.

The most suitable range of application of the orbal system, where it appears competitive with other systems, is for communities of less than 4000. It is probably less competitive for communities of 4000 to 20,000 persons.

Chong, G. M. W., et al., "Comparison of the Conventional Cage Rotor and Jet-Aero-Mix Systems in Oxidation Ditch Operations," Proc., 28th Purdue Industrial Waste Conf. (May 1973), pp 713-723.

A Jet-Aero-Mix (JAM) system, designed on the basis of Fanning's (or Darch) Equation was tested at the USDA Fur Animal Research Farm at Cornell. The system pumped the oxidation ditch liquor through discharge nozzles above or along the liquor's surfaces. The mixing results from energy and momentum transfer from the jet stream to the bulk liquid; the turbulence and aspirator capacity of the jets bring about aeration. A conventional cage rotor device consists of blades radiating from a horizontal axis. The weight of the rotor is a load on the motor, demanding almost 65 to 90 percent of the input energy for rotor-driving, depending on the immersion depth. However, the oxygenation studies with tap water showed that both the JAM system and the cage rotor system were comparable in oxygen transfer rate K_{La} and in lb O_2/hp-hr.

From a general maintenance standpoint, the JAM system does not have the problems of bearings and belt slippage associated with a cage rotor. However, there is a potential problem in nozzles that get plugged up by straw-like materials. From an economic standpoint, the operating costs for energy for the JAM system are comparable to those for the cage rotor. The capital expenditure for the JAM system is considerably lower. Application of the JAM system to an oxidation ditch treating mink waste was successful; BOD and COD removal were 90 and 80 percent, respectively.

LeCompte, A. R., et al., "A Pasveer Ditch, American Jet Style," Proceedings, 29th Purdue Industrial Waste Conference (1974), pp 631-639

Jet aerators, which surround the final clarifier, were tested for an 85-ft inside-diameter reactor-clarifier and 165-ft outside-diameter channel. These dimensions yielded a channel section 37 ft, 4 in. wide by 22 ft deep with an operating depth of 20.5 ft. The jet manifolds were set at 17.5 ft submergence. There were 56 jets, arranged in seven 8-jet manifolds, spaced 45 degrees apart.

Although the head provided by the momentum flux of the jet can be calculated, as can the channel frictional and bend losses, they do not account for the vertical pumping by the jet bubble plumes. Actual testing revealed that an optimal air rate expressed in scfm/jet can be found; this rate provides the smallest values of the overall head loss coefficient and the highest O_2 adsorption efficiency. The best oxygenation capacity was 4.2 lb O_2/Bhp-hr, when the optimal air rate (15 to 18 scfm/jet) was applied. BOD reduction was consistently 90 percent or higher.

Operating conditions were the following:

Sludge recycle rate, average	65 percent
Mixed liquor DO	0.2 to 4.4 mg/L (uncontrolled)
MLSS	3000 to 6800 mg/L
MLVSS	1000 to 3480 mg/L
F/M (apparent)	0.08 to 0.20
SVI	61 to 229 mL/g
Net suspended solids increase	16.8 percent
Sludge yield on soluble BOD	0.29 lb/lb soluble BOD_5

Jacobs, A., "Looped Aeration Tank Design Offers Practical Advantages, Parts I and II," Water and Sewage Works (1975).

The Carrousel system incorporates a mechanically aerated continuous channel. A conventional vertical-shaft, mechanical aerator, located at the influent end of the channel supplies oxygen to the mixture through the channel section. The channel section is fitted with an extended central partition wall and a round channel end to maintain minimum velocities of 0.75 to 1.0 fps; this is sufficient to suspend the mixed liquor solids. At these velocities, the tank contents revolve through the channel once every 10 to 30 minutes, depending on the channel length and the design loading. Constructing the central partition wall close to the aerator allows a uniform turbulent flow in the channel section that is both longitudinal and spiral. Based on calculations applying the Chezy formula for open channel flow, the energy required to provide the channel velocity is about 1 percent of the installed horsepower required for O_2 transfer. The vertical-shaft mechanical reactors provide a complete-mix section and permit substantial channel depths. The Carrousel channel depth is generally 1.1 times the impeller diameter, and the channel width is about 2.0 times the channel depth. The depth at the aerator section of the channel is greater -- about 1.5 times the impeller diameter. Tables A3 and A4 present the design parameters and operational results of the Lichtenvoorde plant in the Netherlands. As of September 1973, there were 38 Carrousel plants in operation -- 19 in the Netherlands and 19 in France.

Johnson, H., "New Solutions to Plant Problems," <u>Food Processing</u> (August 1976), p 42.

An oxidation ditch treatment plant was built in 1972 for the Gay Lea Foods plant in Tara, Ontario. Although the flow was only 48,000 gpd, the BOD of the wastewater often exceeded 1500 mg/L.

The oxidation ditch is a 118-ft-long concrete-lined channel with trapezoidal cross section. The ditch is 20 ft wide at the water line and 10 ft wide at the bottom. A concrete island, 58 ft long and 9 ft wide at the center provides proper liquid flow and supports the rotor aerator. The aerator is 12 ft long and 42 in. in diameter. It has numerous 3-in.-wide steel blades spaced on 6-in. centers. The aerator is placed at an immersion depth of about 6 in. and is driven by a 25-hp electric motor.

Before entering the ditch, the wastewater flow is equalized in a 5000-imperial-gallon storage tank. The detention time is 2.67 days. The final settling tank is a 14-ft-diameter, nonmechanized concrete unit with a hopper bottom. The wastewater is peripherally fed downward with the mixed liquor

Table A3

Design Parameters for Lichtenvoorde Plant
(Reprinted with permission from Water and Sewage Works, "Looped Aeration Tank Design Offers Practical Advantages, Parts I and II" [1975]; by A. Jacobs.)

Capacity:	48,000 p.e. (1 p.e. 0.12 lbs BOD_5)
Average flow:	3.2 mgd
MLSS:	4000 mg/L
F/M ratio:	0.05 lbs BOD_5/lb MLSS
Sludge age:	30 days
Oxygen capacity:	2.5 lbs O_2/lb BOD_5, maximum capacity 1.8 lbs $O/_2$/lb BOD_5, average operational capacity
Aerator size:	
Horsepower	88
RPM	28.5
Impeller dia.	144 inches
Channel dimensions:	
Width	26 feet
Depth	13 feet

spiral; as it travels around the raceway, floatables are released to a scum takeoff pipe for return to the oxidation ditch. Duplex submersible pumps are used for sludge return and for disposal.

The final effluent contains an average of 10 mg/L of BOD and 15 mg/L of SS; this represents more than 99 percent of BOD removal and 97 percent SS removal. The total cost of the oxidation ditch and clarifier system was about $70,000 (1972 dollars). The successful operation of this treatment system has prompted several dairy farms to consider similar installations.

"Oxidation Ditches Perform Well in Winter," J.W.P.C.F., Vol 49 (April 1977), p 537.

Oxidation ditches perform well in winter, as shown by a study conducted in 1976 by EPA Region VII (Iowa, Missouri, Kansas, and Nebraska).* Results indicate that on the average, secondary facilities (with the exception of oxidation ditches) were not meeting secondary treatment effluent standards during the winter of 1975-76.

* The survey did not provide a truly representative cross section of the facility types in Region VII based on actual numerical levels of each type.

Table A4

Summary of Operation Results and Removal Efficiencies for the Lichtenvoorde Treatment Plant (Reprinted with permission from Water and Sewage Works, "Looped Aeration Tank Design Offers Practical Advantages, Parts I and II" [1975]; by A. Jacobs.)

	Influent	Effluent	Percent Removal
BOD_5 (mg/L)	663	4.7	99.3
Settleable Matter (mg/L)	16	0.1	99.4
COD (mg/L)	1316	54.9	95.8
Total Phosphorus	52	40.0	22.6
Kjeldahl (TKN)[1]	108.3	3.1	97.1
Ammonia (NH_3-N)	---	0.8	---
Nitrite (NO_2-N)	---	0.06	---
Nitrate (NO_3-N)	---	43.5	---
Degree of Nitrification[2]	---	---	93.2
Nitrogen Removal[3,4]	---	---	56.9

[1]TKN = Total Kjedahl Nitrogen = NH_3-N + ORG-N

[2]Degree of Nitrification = $\dfrac{NO_3 \text{eff.}}{(NO_3 + NO_2 + TKN) \text{ eff.}}$

[3]Nitrogen Removal = $\dfrac{TKN \text{ inf.} - (TKN + NO_2 + NO_3) \text{ eff.}}{TKN \text{ inf.}}$

[4]Lichtenvoorde plant was not designed or operated for nitrogen removal.

Thirty facilities were surveyed in the study: 16 trickling filter plants (covered and uncovered), seven oxidation ditch facilities, and seven activated sludge plants. Twenty of these facilities averaged less than 85 percent removal of a composite parameter comprising BOD, nonfilterable solids, and ammonia-nitrogen. Of the ten facilities that were averaging 85 percent removal under the difficult winter conditions, five were oxidation ditch facilities, four were activated sludge plants, and one was a covered trickling-filter plant. The activated sludge (compressed air) and oxidation ditch facilities had peak performances of up to 99 percent removal of BOD and nonfilterable solids. The data also indicated that well-operated activated sludge and oxidation ditch facilities can be expected to produce effluents with less than 0.5 mg/L ammonia-nitrogen, even at water temperatures under 5°C.

Performance difficulties were caused by two major problems: (1) generally low-skilled operators and inadequate time for optimizing facility operation; and (2) failure of designers to anticipate industrial loads, plus poor monitoring and in-plant handling of industrial wastewaters. Oxidation ditches seemed to be the type of facility least affected by lack of operator competence.

H. D. Stensel, et al. (Envirotech), "Cost Effective Advanced Biological Wastewater Treatment Systems," paper presented at the Nevada Water Pollution Control Association (1977).

This paper discusses design considerations for biological wastewater treatment systems. If only BOD removal is required, then a sludge age of about 5 days would be used for the design. (An application for removing only BOD would not take full advantage of the Carrousel's energy savings for mixing, but could still take advantage of its other aerator application features.) For nitrification designs, the sludge age would be in the range of 10 to 20 days, depending on the wastewater temperature. For an extended aeration design, a Carrousel system would be designed on the basis of about a 30-day sludge age or 20- to 24-hour hydraulic detention time to achieve complete sludge stabilization as well as nitrification and low effluent BOD_5 values. Once the sludge age is selected, the following equation is used to determine the necessary aerobic volume in the system:

$$V = \frac{(SRT)(Yu)(\Delta BOD)}{MLSS(8.34)} \qquad \text{[Eq A1]}$$

where

V = aeration volume (million gallons)
BOD = lb of BOD removed per day
Yu = net solids production coefficient, lb SS/lb BOD_5
MLSS = mixed liquor SS concentration mg/L.

The MLSS concentration is selected at about 3500 mg/L to provide conservative solid floor loadings in the secondary clarifier. However, it has been common to operate Carrousel systems in Europe at MLSS of 4000 to 7000 mg/L.

To incorporate denitrification into the system, the volume in the equation must be increased to account for the denitrification anaerobic volume.

$$VD = \frac{N}{DNR(MLSS)8.34} \qquad \text{[Eq A2]}$$

where

VD = denitrification volume, mg
N = quantity of nitrate nitrogen to reduce, lb/day
DNR = specific solids denitrification rate, lb N/lb MLSS day.

The next step is to determine the aerator horsepower requirements. Figure A6 is an example of a curve used to select the amount of O_2 for BOD removal as a function of the design sludge age. Besides this oxygen requirement, the amount of oxygen needed for nitrification is 4.3 lb O_2/lb NH_3-N oxidized. If denitrification is anticipated, 2.86 lb O_2/lb NO_3-N reduced is deducted from the O_2 requirements of BOD/NH_3-N removal. Once the O_2 requirements are determined, the aerator standard oxygen transfer efficiency value is converted to a mixed liquor condition value and is used to determine the total horsepower needed for the system.

Various configurations can be laid out in a Carrousel system as shown in Figure A7. Normally, the channel depth is about 1.2 times the impeller diameter, and its width is twice the depth. The hydraulic model developed by Dwars, Heederik, and Verhay, Ltd., is used to optimize the aeration basin and aerator design to provide maximum channel velocity.

<u>Efficient Treatment of Small Municipal Flows at Dawson, Minn.</u>, EPA Techn. Transfer, Capsule Report, EPA-625/2-77-015 (U.S. Environmental Protection Agency, 1977).

High effluent suspended solids caused by improper plant operation, surges of raw sewage flow, or mechanical failures within the plant frequently degrade the effluent quality of small plants such as that at Dawson, MN. The problems are common in small plants which receive limited operator attention and are frequently subjected to severe flow variations. A survey in 1960 of extended aeration plants throughout the United States showed that they removed an

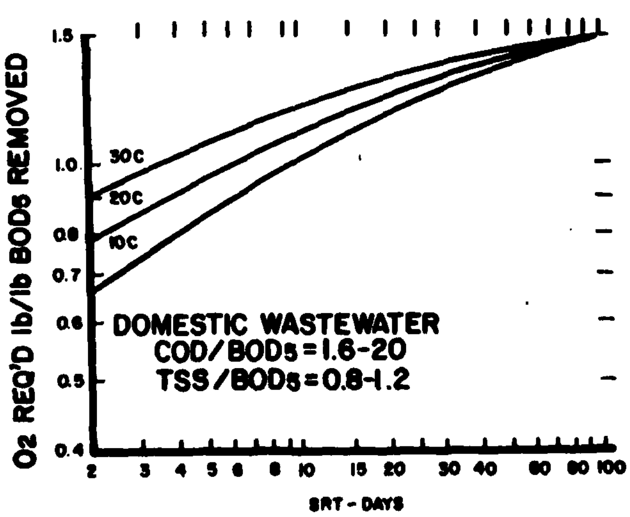

Figure A6. Oxygen requirements for carbonaceous BOD_5 removal vs. SRT and temperature (add O_2 for nitrification as required). (From H. D. Stensel, et al., [Envirotech], "Cost Effective Advanced Biological Wastewater Treatment Systems," paper presented at the Nevada Water Pollution Control Association [1977].)

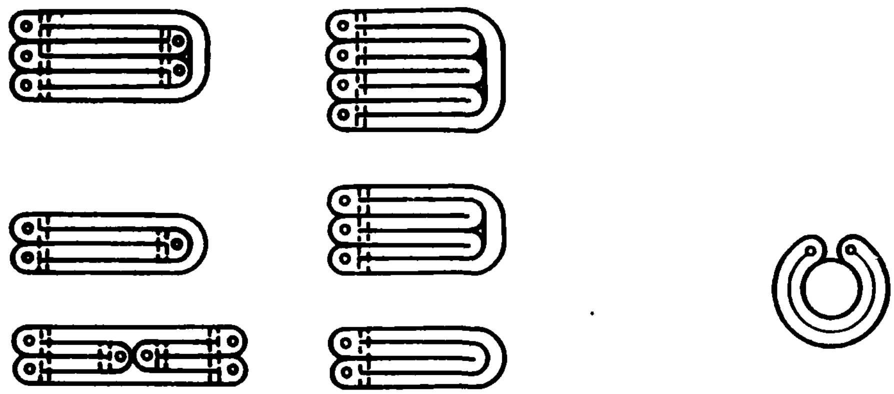

Figure A7. Carrousel tank configurations. (From H. D. Stensel, et al., (Envirotech), "Cost Effective Advanced Biological Wastewater Treatment Systems," paper presented at the Nevada Water Pollution Control Association [1977].)

average of 86 percent of the BOD, but only 62 percent of the suspended solids from the raw sewage.

Figures A8 and A9 illustrate the extended aeration process. Following screening, the raw wastewater is pumped to an aeration channel or "oxidation ditch." The process is a modification of the activated sludge process. Aeration is provided by two floating aerators which also keep the liquid moving around the aeration channel at a velocity high enough to prevent deposition. The depth of mixed liquor can then be varied from 3 to 4.1 ft, providing 83,000 gal of surge storage capacity within the aeration channel. At design flows, the aeration time in the channel is 17.7 to 25.4 hr, depending on the depth. Under the actual flows received, aeration times averaged 35 hours during a 300-day test period. The mixed liquor flows from the channel to a chamber which controls the flow to the downstream clarifiers. By using the storage volume in the aeration channel, peak flows were reduced by about 31 percent, stabilizing the operation of the secondary clarifiers.

The mixed liquor flows to two downstream clarifiers in series. Design overflow rates are 580 and 400 gpd/sq ft for clarifiers 1 and 2, respectively. Provisions were made to feed chemicals and provide flocculation between the clarifiers to insure that the stringent effluent suspended solids goal of 5 mg/L would be met. It has not been necessary to feed chemicals. Chlorination is provided before discharge. Excess sludge is taken to nearby farmland where it is spread. The performance of the plant over the first 300 days of operation is summarized in Table A5. If suspended solids from only 2 days of operations are eliminated, the average suspended solids content of the final effluent becomes 5 mg/L.

Nearly complete nitrification was achieved throughout the entire study, since ammonia concentrations in the effluent were generally 0.1 mg/L or less. Nitrification was achieved even during severely cold weather when water temperatures in the aeration channel were very near 0°C. Temperatures were below

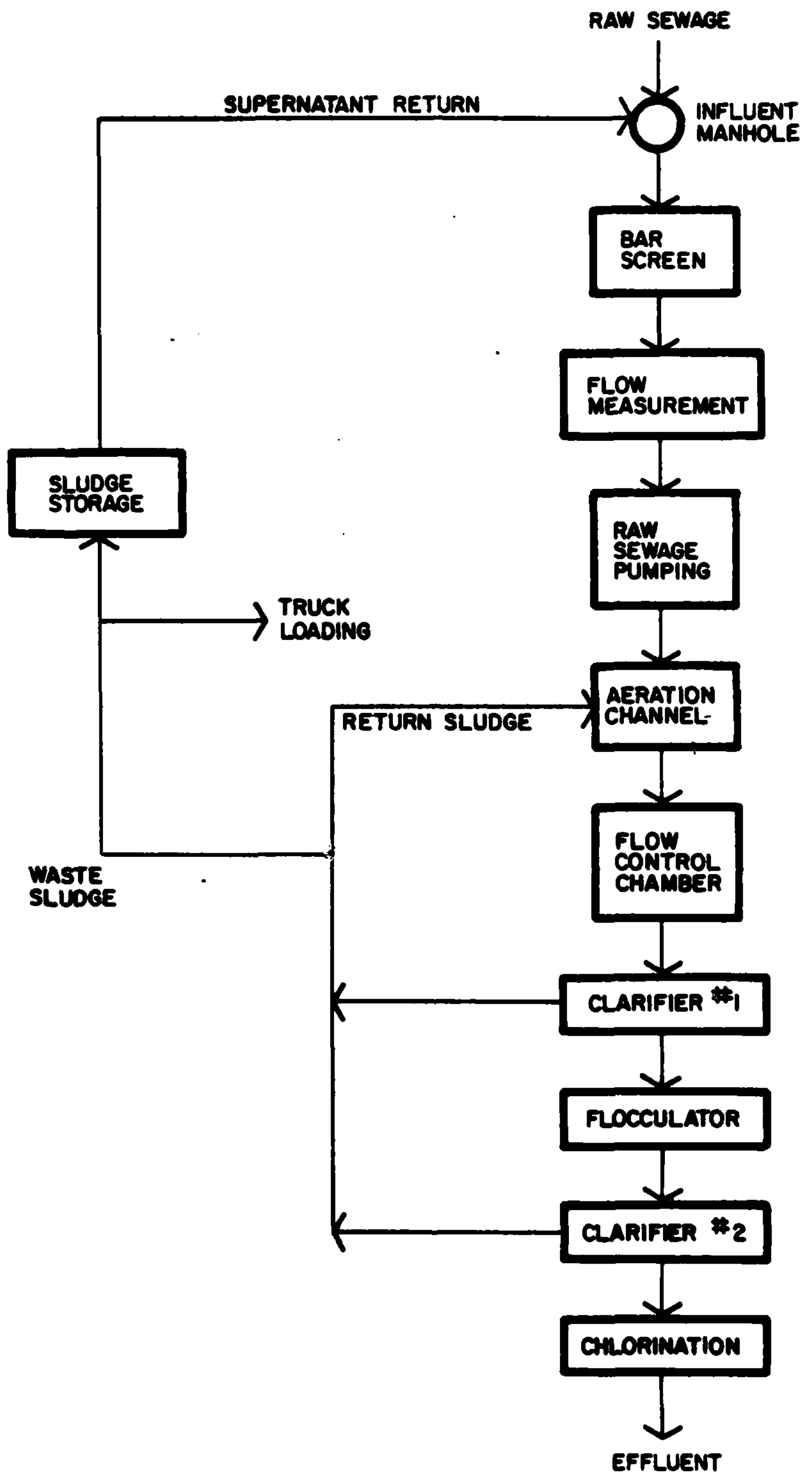

Figure A8. Process flow sheet. (From Efficient Treatment of Small Municipal Flows at Dawson, Minn., EPA Technology Transfer, Capsule Report, 625/2-77-015 [USEPA, 1977].)

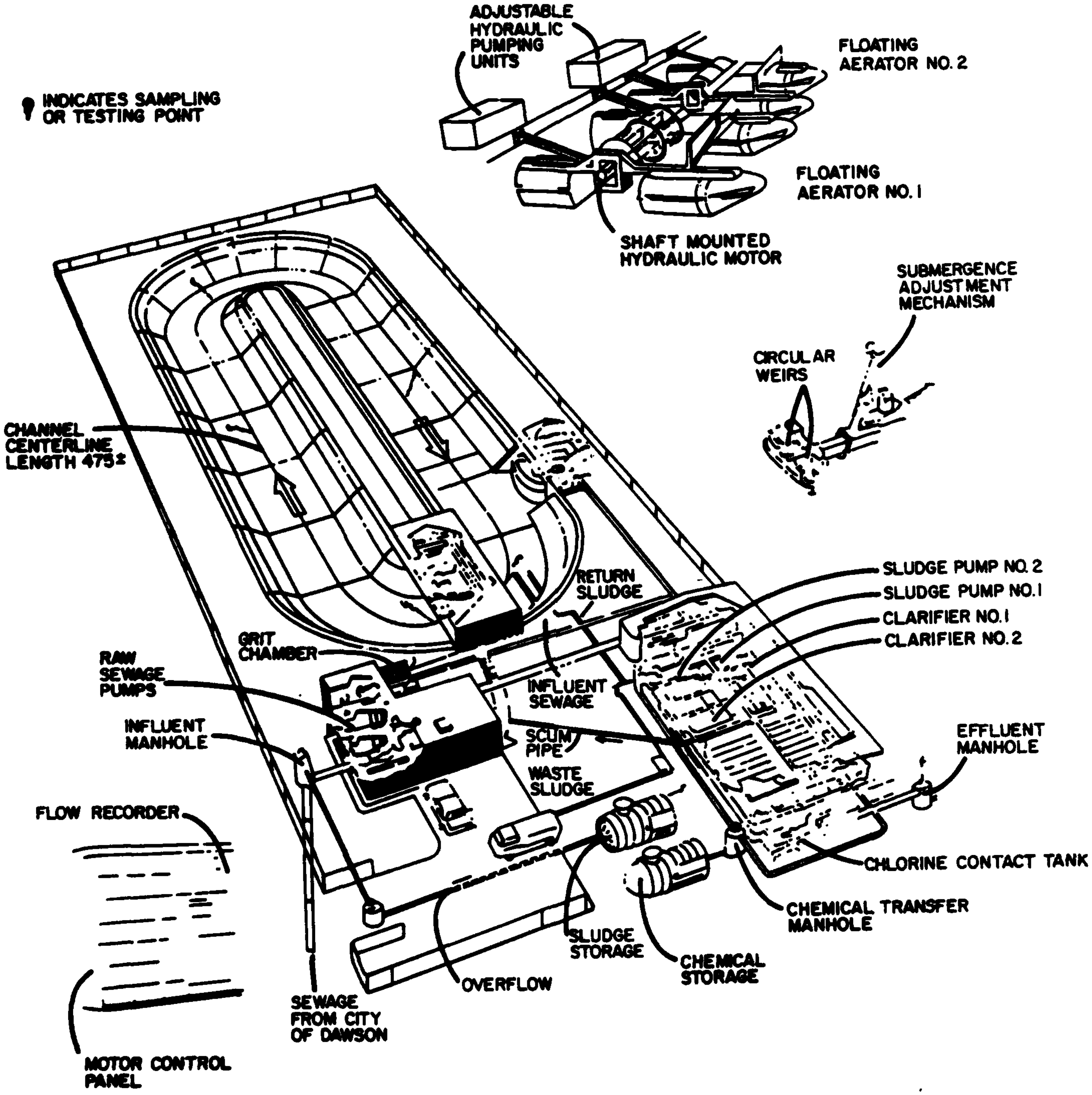

Figure A9. Dawson treatment plant. (From Efficient Treatment of Small Municipal Flows at Dawson, Minn., EPA Technology Transfer, Capsule Report, 625/2-77-015 [USEPA, 1977].)

Table A5

Summary of Dawson, MN, Plant Performance
(From Efficient Treatment of Small Municipal Flows at Dawson, Minn., EPA Technology Transfer, Capsule Report, 625/2-77-015 [USEPA, 1977].)

	Average	Range
BOD, mg/L		
Influent	155	60-250
Effluent	3	1-7
Suspended Solids, mg/L		
Influent	200	70-1621
Effluent	8	1-30

6°C for the first 100 days of the 300-day study period. Sludge ages were always 20 days or more. It was found that denitrification would occur if aeration was carefully controlled so that DO levels were at zero before the mixed liquor completed its circuit around the aeration channel to the aerator. Over the 65-day period that these controls were used, nitrogen removals of 60 to 80 percent were typically achieved, with some values as high as 90 percent. The wastewater itself provided the carbon source for the denitrifying organisms, so that no additional methanol or other carbon sources were required.

The plant could be operated for as long as 90 days without the need for sludge wasting by allowing the mixed liquor solids to build up to 10,000 mg/L before wasting sludge. The mixed liquor solids readily settled to solid concentrations of 2.5 to 3.5 percent.

This high degree of treatment has been achieved consistently without any extraordinary operator skill or attention. About 4 to 6 manhours per day are spent at the plant. Typical duties of a plant operator are:

1. Read influent flow matter.

2. Clean bar screen.

3. Skim settling tanks.

4. Visually check all equipment to make sure it is operating.

5. Measure temperature of air, channel, and settling tank contents.

6. Collect necessary samples for analysis (primarily raw sewage, MLSS, and effluent).

7. Check DO of channel.

8. Check chlorine residual and set feed rate.

9. Wash down the walkways and building walls.

10. Haul sludge (when required).

11. Cut grass or shovel snow as necessary.

Table A6 presents the construction costs of the Dawson plant both at the time of the actual bid (1971) and using an EPA sewage treatment plant index which reflects second quarter, 1976 cost levels. Table A7 gives operation and maintenance costs. Power costs are primarily for the electric motors which drive the various items of process equipment. The total connected horsepower is 53 Hp; however, not all of the equipment is in service continuously. Heating costs are for an oil-fired furnace and hot-air blower system within the control building and laboratory. This is the only structure in the plant site

Table A6

Dawson Construction Costs
(From Efficient Treatment of Small Municipal Flows at Dawson, Minn., EPA Technology Transfer, Capsule Report, 625/2-77-015 [USEPA, 1977].)

Approximate Unit Process or Item Costs	Bid Date November 30, 1971 @ EPA STP Index = 166	Estimated Costs @ EPA STP Index = 255
Raw Sewage Pumps and Related Items	$ 18,000	$ 28,000
Channel Aerators and Related Items	100,000	153,000
Clarifiers and Housing	65,000	100,000
Return Sludge Pumping System	12,000	18,000
Control Building and Laboratory	43,000	66,000
Other (Chlorination, Chemical Feed System)	12,000	18,000
	$ 250,000	$ 383,000

that is equipped with a permanent heating system. Because of freezing problems in the clarifier building, portable heater units are sometimes used during the winter. This cost is included under miscellaneous supplies and replacement parts. Miscellaneous supplies and replacement parts include items such as furnace repairs, replacement of a hydraulic hose on the aerators, grease and oil, cleaning supplies, lawn mower parts, and similar items. No major repairs were made on the process equipment items. Transportation costs reflect the cost for disposing of sludge on nearby fields. The sludge is disposed of in liquid form if weather conditions permit.

Keffer, W. J., Winter Performance of Secondary Wastewater Treatment Facilities, Region VII Report (U.S. Environmental Protection Agency, 1977).

Region VII of the USEPA has collected and summarized a great deal of detailed wastewater plant performance data; this information includes results of multiple days of analysis on 24-hour composite influent and effluent samples. These performance summaries include data collected through all seasons of the year in both high and low precipitation periods.

Table A7

Dawson Operation and Maintenance Costs
(From Efficient Treatment of Small Municipal Flows at Dawson, Minn., EPA Technology Transfer, Capsule Report, 625/2-77-015 [USEPA, 1977].)

Cost Item	Raw Sewage Pumping	Channel & Aerators	Clarifier No. 1	Clarifier No. 2	Return Sludge Pumping	Control Bldg. & Laboratory	Other	Totals
Power	0.5	3.6	0.1	0.1	1.2	0.5	0.2	6.2
Heating Costs						0.7		0.7
Chlorine							1.0	1.0
Misc. Supplies & Replacement Parts						2.0	0.6	2.6
Labor	0.5	1.5	1.0	1.0	1.2	6.0	2.3	13.5
Transportation							0.2	0.2
Subtotals	1.0	5.1	1.1	1.1	2.4	9.2	4.3	24.2

*Average flow of 174,650 gallons per day.

**Cents/1,000 gallons treated.

The latest summary (Table A8) includes 249 facilities which have been inspected and sampled since 1973. The summary shows that actual field performance of "secondary" facilities is not meeting the definition of secondary treatment. The 226 secondary plants on the list are, on the average, exceeding the BOD_5 level of 30 mg/L by 27.3 percent and the suspended solids level of 30 mg/L by 44.7 percent. The most important single factor contributing to this inadequate performance level is undoubtedly inadequate skills and available time of the wastewater plant operators. Other significant contributing factors are toxic materials in the wastewater influent and organic overloads from food-processing facilities.

The following provides information on one important subset of the overall performance summary -- data from 30 facilities sampled during the winter of 1975 through 1976.

Several types of "secondary" treatment works, especially trickling-filter plants and oxidation ditches, are becoming more commonly used. A field data collection survey of selected facilities was therefore designed to provide performance information on these systems.

There are only a few oxidation ditch-type facilities operating in Region VII; therefore the survey was designed with these facilities as logistical focal points. Generally, other types of secondary plants were selected at random from those within a 75-mile radius of the oxidation ditch facilities. The surveyors tried to select facilities which cover the expected range of age, flow volume, operator competence, and industrial waste problems; however, based on actual numerical levels of each type of facility, the general results of the survey do not indicate effluent quality for a truly representative cross section of the region.

Thirty facilities were selected for 3 days of influent and effluent 24-hour composite sample collections between January 10 and March 2, 1976 (see Table A9). Plant flow rates ranged from 30,000 gpd to 17.4 mgd. Each facility was sampled for three consecutive 24-hour daily influent and effluent composites for BOD_5, COD, NFS, Total P, NH_3-N, TKN, NO_2-NO_3-N, and influent and effluent water temperatures.

Table A10 summarizes data from the 30 facilities. Generally, only the oxidation ditch subset was meeting the secondary treatment effluent definition.

Figures A10 and A11 show removal efficiencies for BOD_5 and NFS -- the conventional performance-measuring parameters. The best performance of both activated sludge (compressed air) and oxidation ditch facilities approaches 99 percent removal; as expected, performance of both types of trickling filter was somewhat poorer. The wide range of performance, especially for activated sludge facilities, shows the inadequacies of treatment personnel to optimize the performance of complex wastewater plants.

Figure A12 shows that nitrogen conversion efficiencies are the most sensitive measure of optimal performance. The data show that well-operated activated sludge and oxidation ditch facilities can be expected to produce effluents with less than 0.5 mg/L of ammonia, even at water temperatures under $5^{o}C$. It is also apparent that most plants are not doing this and that even

Table A8

Municipal Sewage Plant Performance Summary
(From Efficient Treatment of Small Municipal Flows at Dawson, Minn.,
EPA Technology Transfer, Capsule Report, 625/2-77-015 [USEPA, 1977].)

COLUMN 1	2	3	4			5				6			7				8		
PLANT TYPE	STATE	NO. OF PLANTS	INFLUENT BOD5			EFFLUENT BOD 5				INFLUENT COD			EFFLUENT COD				INFLUENT MFS		
			AVG	RANGE	S.D.	AVG	RANGE	S.D.	%REM	AVG	RANGE	S.D.	AVG	RANGE	S.D.	%REM	AVG	RANGE	S.D.
Primary Sedimentation		23	416			300			27.0	1032			534			48.2	512		
Lagoons		80	173			41			76.3	378			131			65.3	160		
Activated Sludge		41	200			44			78.0	516			121			76.5	206		
Trickling Filters		89	104			33			82.1	426			100			76.5	197		
Oxidation Ditch		7	291			20			93.1	742			69			90.7	238		
Package STP'S		9	102			48			73.6	400			122			69.5	177		
		249																	
All Secondary Plants		226	186.2			38.2			79.4	434			14.7			73.5	186.0		

COLUMN 1	9				10			11				12			13			
PLANT TYPE	EFFLUENT MFS				INFLUENT NH3-N			EFFLUENT NH3-N				INFLUENT TOTAL P			EFFLUENT TOTAL P			
	AVG	RANGE	S.D.	%REM	AVG	RANGE	S.D.	AVG	RANGE	S.D.	%REM	AVG	RANGE	S.D.	AVG	RANGE	S.D.	%REM
Primary Sedimentation	116			77.3	16.4			16.8			—	9.3			8.0			13.9
Lagoons	53			66.8	13.6			6.2			54.4	9.2			5.4			41.3
Activated Sludge	46			77.7	17.3			11.2			35.2	11.5			7.8			32.1
Trickling Filters	35			82.2	18.5			8.6			53.5	10.4			8.8			15.3
Oxidation Ditch	14			94.1	22			5.1			76.[illegible]	12.5			8.9			28.0
Package STP'S	53			78.0	[illegible].8			7.1			55.0	9.9			7.1			28.2
All Secondary Plants	43.4			76.6														

Table A9

1975-76 Winter Survey Plant Size Ranges by Type
(From W. J. Keffer, Winter Performance of Secondary Wastewater Treatment Facilities [USEPA, Region VII, 1977].)

lows (MGD)	Trickling Filter	Oxidation Ditch	Activated Sludge	Total
<0.10	3	1	---	4
0.11 to 0.25	2	4	3	9
.26 to 1.00	3	2	3	8
.00 to 10.0	6	---	1	7
>10.0	2	---	---	2
TOTAL	16	7	7	30*

Three facilities originally scheduled for inclusion in the survey were dropped because the secondary portion of the facility was out of operation at the time of the sampling effort.

small departures from optimal treatment efficiencies can greatly increase effluent NH_3-N concentrations. It should be noted that the poor performance of two of the oxidation ditches in this area are caused by specific problems, such as a frozen sludge return line and poor design of the ditch, which caused dead spots and septic conditions in turn areas.

Figure A13 is a somewhat unconventional composite removal display; however, it is recommended for appraising plant performance, since it includes all the parameters of normal concern and can best identify where operational and design deficiencies are causing poor performance. Twenty of the thirty selected facilities are not averaging 85 percent removal. Of the ten facilities which are removing 85 percent under tough climatic conditions, five are oxidation ditches, four are activated-sludge plants, and one is a covered trickling-filter plant.

The oxidation ditch facilities surveyed represent more than 75 percent of these plants known to be operating in EPA Region VII. Operations range from 8 hours per day for 5 days a week to an estimated 8 hours per week. As the data show, these plants seem to be least affected by the caliber of operator competence.

The uncovered trickling-filter plants showed a wide range of operator attention and capability. The covered trickling-filter facilities ranged from small fixed operation units to units where two people work 8 hours each day to optimize facility operation.

The activated sludge facilities sampled range from generally unattended facilities to well-operated units with surface aerators in square basins and covered secondary clarifiers to the most modern facility with two-stage nitrification. Range of performance characteristics was greatest in the activated-sludge subset.

Table A10

Plant Performance 1975 - 1976 Winter Data
(From W. J. Keffer, Winter Performance of Secondary Wastewater Treatment Facilities [USEPA, Region VII, 1977].)

Type Plant	Oxidation Ditch		Uncovered Trickling Filter Plants		Covered Trickling Filter Plants		Activated Sludge		Total Survey	
Parameter	mg/L	% Rem	mg/L	% Rem	mg/L	% Rem	mg/L	% Rem	mg/L	% Rem
BOD_5	9.7 ± 6.9	95.0	31.1 ± 12.2	77.2	86.5 ± 88.6	68.9	28.3 ± 37.2	84.7	42.1 ± 60.3	80.6
COD	54.5 ± 38.9	88.4	95.6 ± 37.8	72.4	211.4 ± 201.1	66.8	66.1 ± 65.7	83.9	116.1 ± 133.8	77.1
NFS	11.5 ± 6.8	95.0	35.3 ± 13.6	82.2	49.1 ± 47.1	76.3	30.9 ± 38.0	81.0	32.8 ± 35.3	83.1
NH_3-N	5.63 ± 7.1	71.7	14.9 ± 8.9	43.4	40.8 ± 45.1	24.6	7.9 ± 8.3	54.3	18.9 ± 29.5	47.9
H_2O Temp., °C	7.1		9.5		10.4		9.0		9.1 ± 2.7	
Number of Plants	7		7		9		7		30	

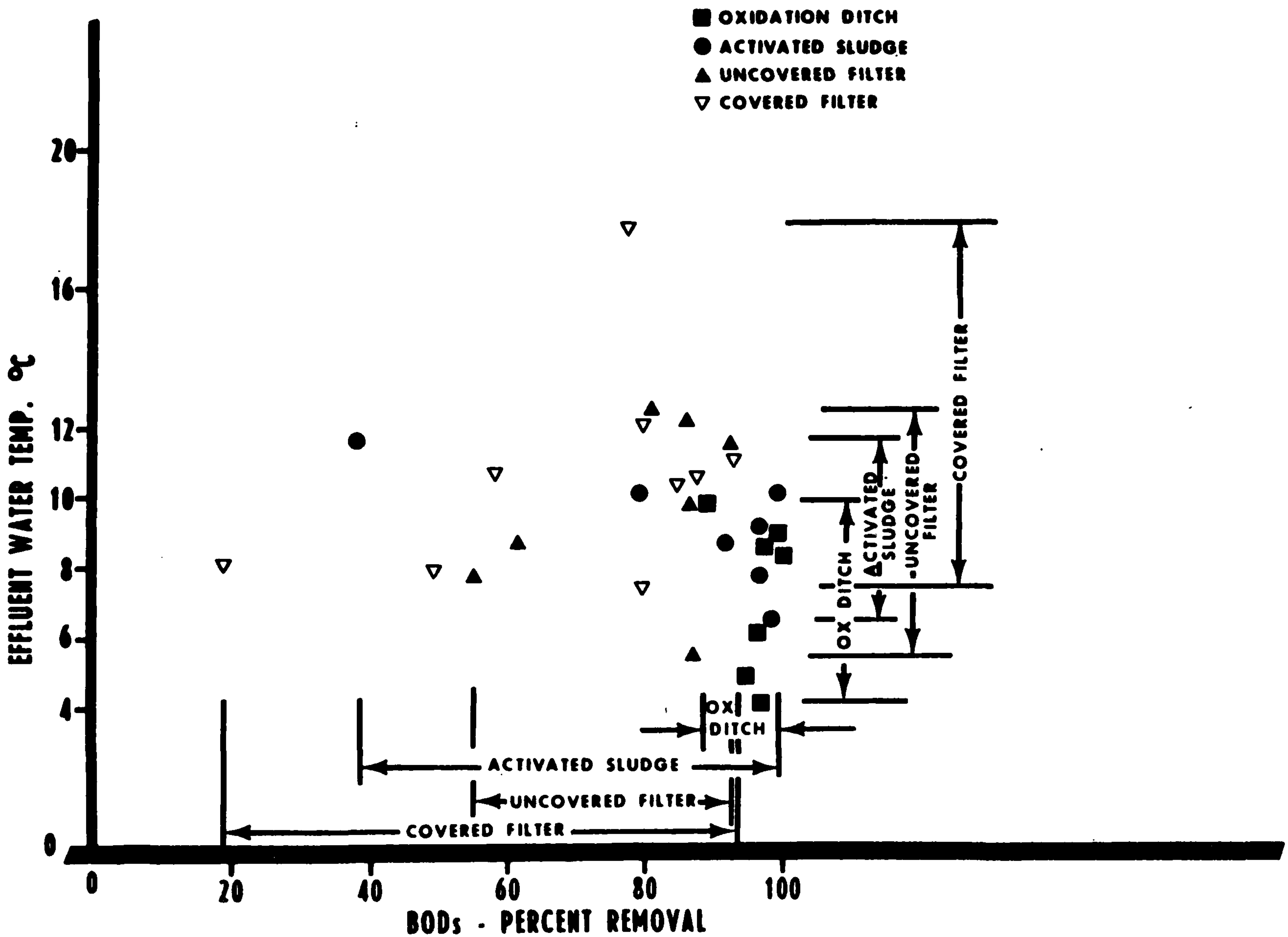

Figure A10. Removal efficiencies for BOD_5. (From W. J. Keffer, Winter Performance of Secondary Wastewater Treatment Facilities [USEPA, Region VII, 1977].)

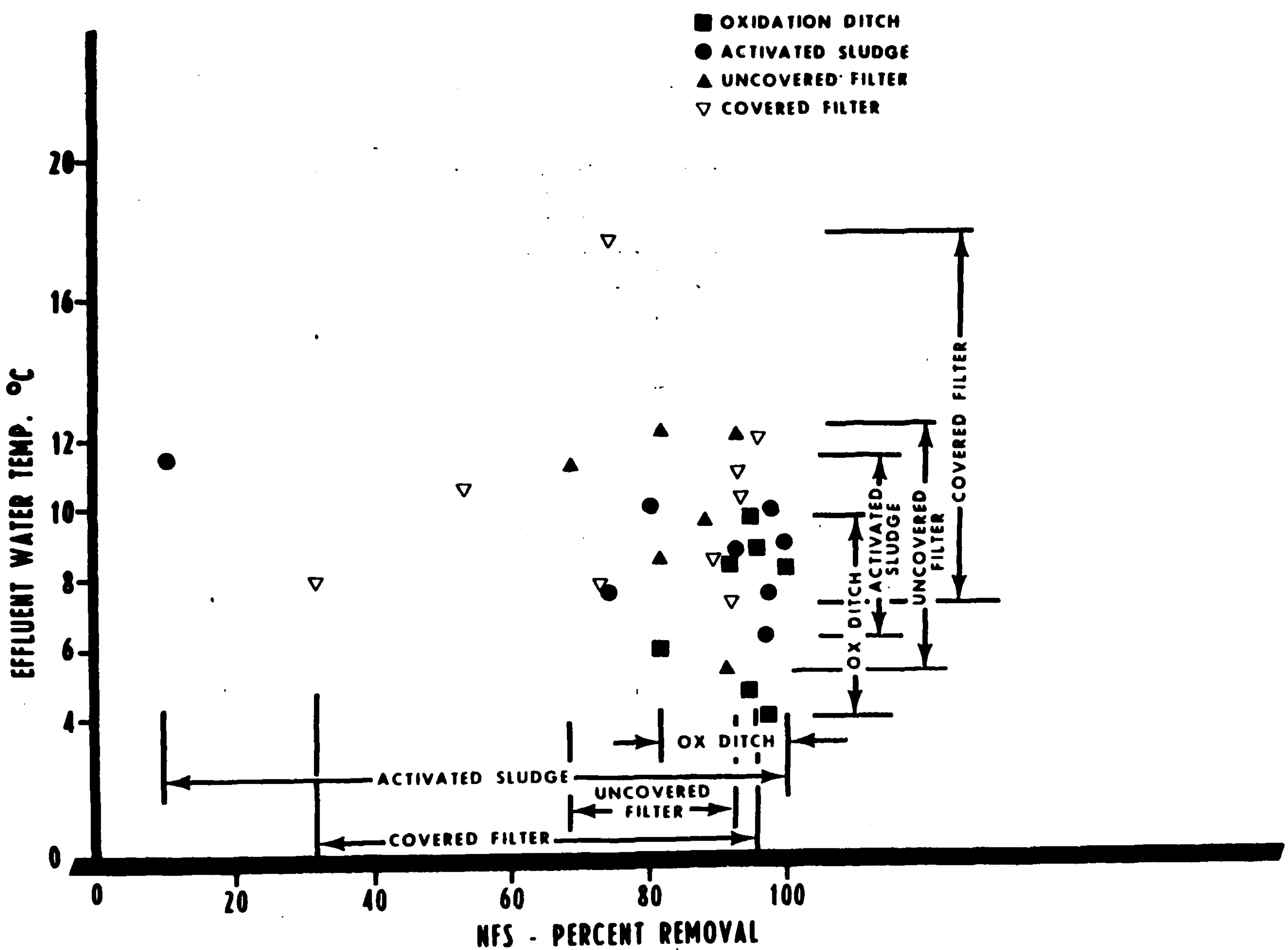

Figure A11. Removal efficiencies for NFS. (From W. J. Keffer, Winter Performance of Secondary Wastewater Treatment Facilities [USEPA, Region VII, 1977].)

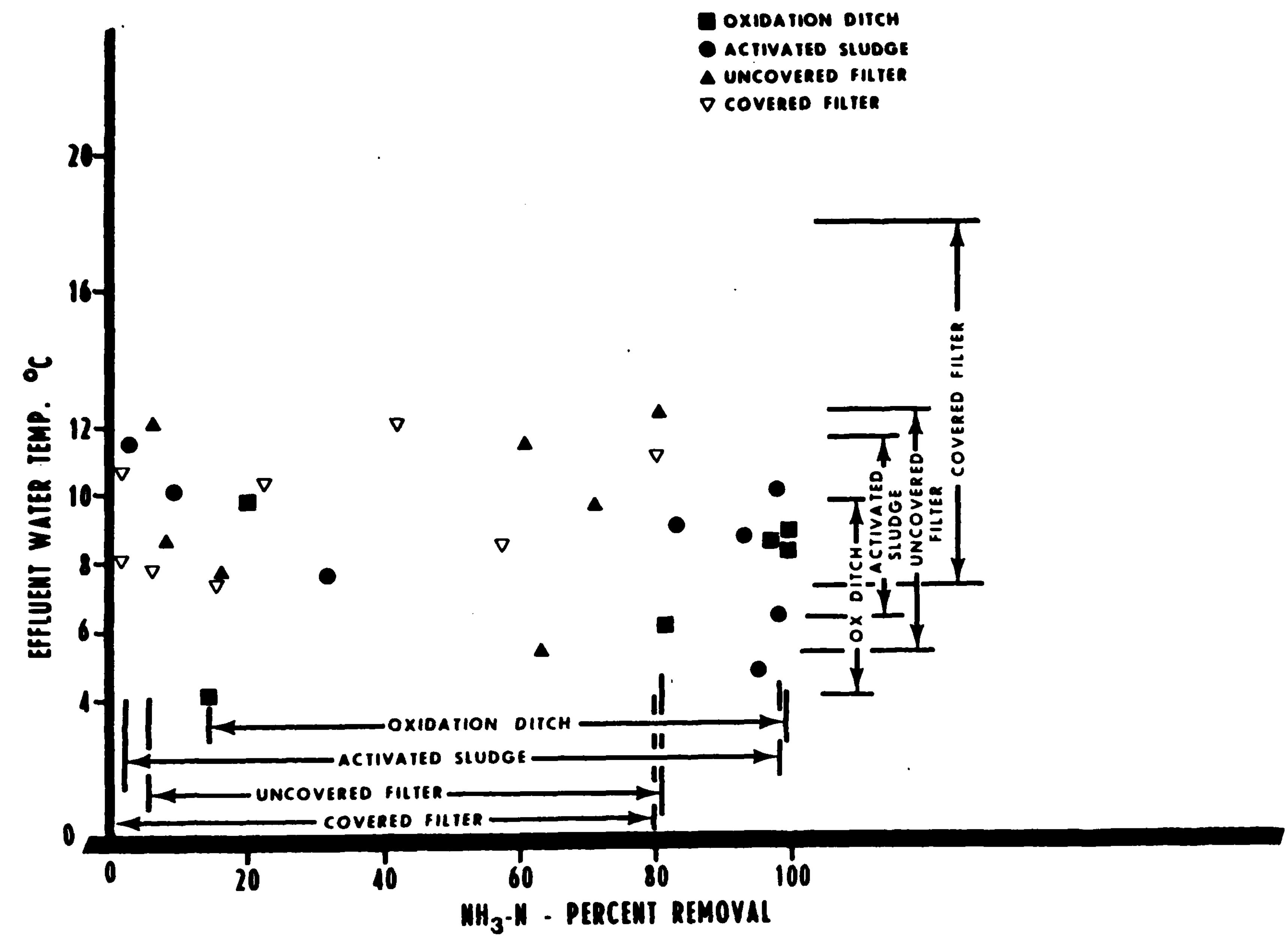

Figure A12. Ammonia-nitrogen conversion efficiencies. (From W. J. Keffer, Winter Performance of Secondary Wastewater Treatment Facilities [USEPA, Region VII, 1977].)

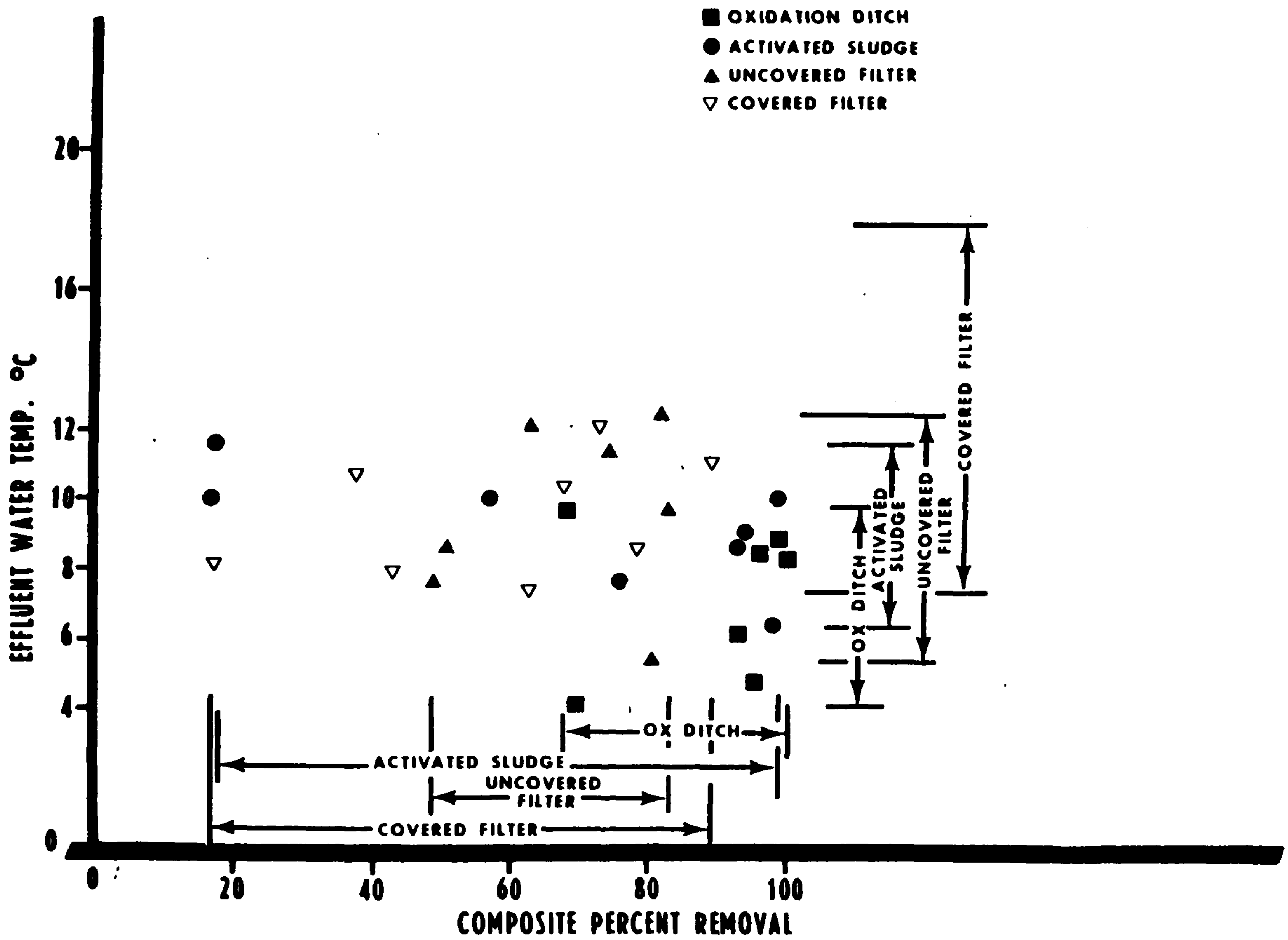

Figure A13. Composite removal efficiencies for BOD_5, NFS, and NH_3-N. (From W. J. Keffer, Winter Performance of Secondary Wastewater Treatment Facilities [USEPA, Region VII, 1977].)

The failure to optimize operations at the facilities sampled apparently results from two main problems. These problems also tend to compound the adverse effect observed on plant effluent.

1. Generally low skill level of operators and inadequate available time to optimize facility operation.

2. Failure of design to anticipate industrial loads, and poor monitoring and plant operation to handle industrial wastes received.

The data in this survey point up the need for closer communications between in-house program elements whose actions affect each other, increased O&M efforts, and a need for further study in selected areas, especially ammonia removal.

Engineering News Record (October 5, 1978), pp 61-62.

In 1978, only two Carrousel process systems had been installed in the United States -- one at the A. C. Lawrence Leather Co., Inc., in Winchester, NH, and the other one at Campbellsville. The Winchester plant has a 500,000 gpd capacity. During a 2-week testing period, the plant achieved 99.4 percent BOD removal and a 95.3 percent ammonia removal, despite large quantities of emulsified wool grease and animal fat in the wastewater that were difficult to treat.

The Campbellsville plant treats the combined city wastewater and the waste from a clothes manufacturing plant (4.2 mgd). The combined wastewater has a BOD averaging 352 mg/L, which totals 12,445 lb/day. Effluent requirements for the area call for 98 percent removal of pollutants. Detention time for the wastewater varies from 20 to 40 hours.

Compared to a conventional sewage treatment scheme, the Carrousel process is supposed to reduce a plant's cost from $8.0 million to $3.5 million. It uses 20 percent less electricity than any other comparable activated sludge system and produces only about 70 percent of the excess sludge by solid weight.

The economic advantage of a system like Campbellsville's may top out at around 10 mgd; at that size or greater, the cost of the concrete required could exceed the mechanical costs of other schemes.

Beer, C., "Looped Reactors for the Activated Sludge Process," Clear Waters (September 1978), pp 22-27.

This article describes the salient features and design criteria of several types of oxidation ditch systems. The loading parameters used by Pasveer (0.05 kg BOD_5/kg of mixed liquor SS and 0.2 kg BOD_5/m^3 of aerator space) are still used today. However, primary settling is no longer used. The oxidation ditch achieves a better effluent and uses less energy than extended aeration; however, it does produce more sludge.

Single-Loop Plants Using Horizontal Rotor-Aerators

For daily flows greater than about 250,000 gal, 42-in.-diameter brush aerators are used on channels with vertical walls. Rotor length is commonly 4 to 30 ft, design channel depth 4 to 12 ft, and blade inversion 4 to 12 widths. The largest unit is 8 mgd design flow with an 8-ft side water depth.

For small flows, horizontal rotors with a 27- to 30-in. diameter are used (manufactured by Lakeside Passavant and by Cherne Ind., Inc.). Both brush-type and cage rotors are available. The Cherne aerators (perforated paddles) are mounted on floats and equipped with variable-speed drives. Cage rotors (Lakeside) are 27 in. in diameter and are used in ditches about 1.4 m deep. Most of the small oxidation ditch plants in the United States are equipped with cage rotors (see Figure A14).

The EPA and Lakeside have concluded that a 20/20 effluent can be produced reliably in a looped reactor plant, using brush or cage rotors and the oxidation ditch process, and without employing DO control measures or special operating care. Complete nitrification is often achieved, but suppliers of brush and cage rotors usually do not emphasize N removal. Sludge production

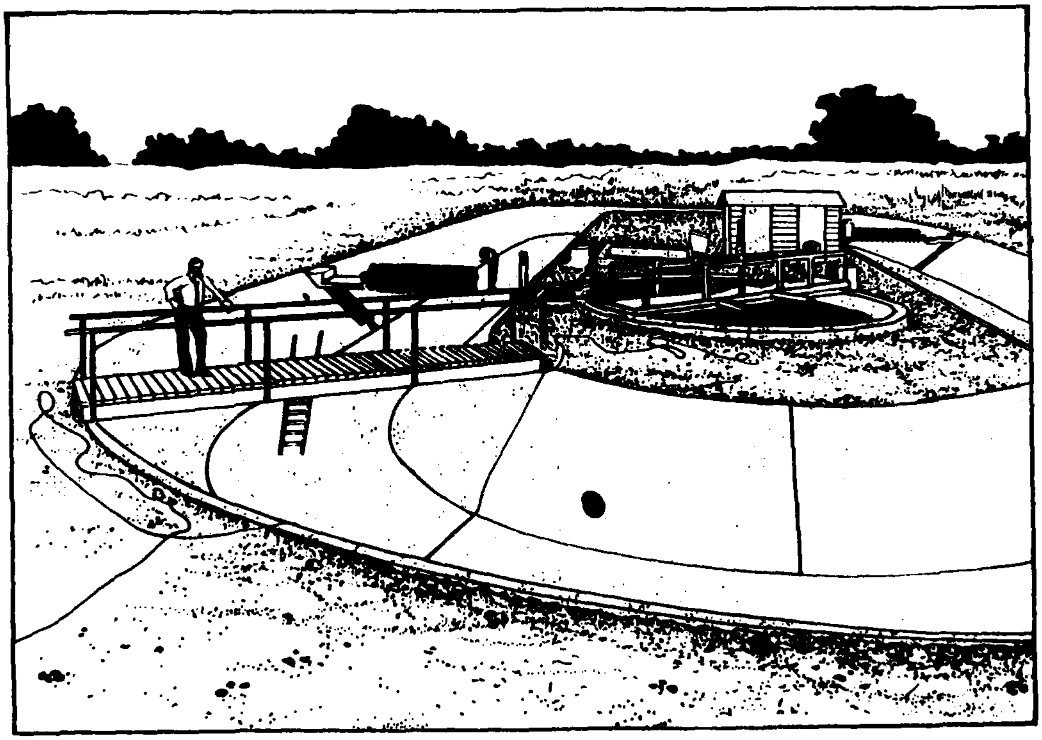

Figure A14. Oxidation ditch plant with cage rotors. (From Lakeside Equipment Corporation.)

is 0.55 to 0.6 mg/mg BOD_5 removed. Volatile content of the sludge ranges from 60 to 70 percent; the higher volatility occurs in the winter.

The Dawson, NM plant (see pp 126-139) is equipped with Cherne aerators and a measure of flow control. A protective housing is used for the aerators. The average effluent BOD_5 is 3 mg/L, and the average SS is 8 mg/L for the first 300 days of operation. This plant uses two settlers in series.

Orbal Plants

The orbal system is similar to the "folding" design procedure used in the Carrousel plants. The number of aeration disks per channel decreases from channel to channel, proceeding inward. The disk aerators run at about 52 rpm. According to the manufacturer (Envirex), 6 mgd is the maximum economical plant size. In 1975, 34 orbal plants were operating in the United States.

Jet-Aerated Channels

In comparison with other looped reactor systems, the Pentech-Houdville system has several advantages: the depth and width of the channel are independent of each other so greater channel depths may be used; and the adverse cooling effect of surface aerators is avoided, and return sludge and raw sewage are mixed very efficiently with the mixed liquor by the jets. The deepest channel in use is 24 ft.

Carrousel Plants

Dwars, Heedrik en Verhey B. V. of the Netherlands have adapted slow-moving, large-diameter, vertical-surface aerators to the oxidation ditch process and introduced the "folded" design of looped reactors. These two measures greatly improve effluent quality and allow the design of large plants which are commercially competitive with the traditional activated-sludge plants (Figure 4, p 25).

The pollution control strategy adopted by the Netherlands in 1971 helped promote the use of Carrousels. This strategy includes discharge fees based on a formula containing the factor 2.5 BOD_5 + 4.57 TKN. BOD_5 removal and nitrification are required, but not nitrogen removal. However, nitrogen removal efficiency of about 50 percent is being achieved in municipal Carrousels. It has been observed that BOD_5 removal increases with increasing nitrogen removal. A popular channel cross section is 4 x 8 m; however, cross sections of 5 x 10 m have also been used. There is a fixed relationship between channel width and channel depth. Also, large aerators must be used for deep-channel reactors; a 5-m-deep channel requires an aerator of 4-m diameter. The smallest plant flow to which the Carrousel system can be adapted is 100,000 gal/day. There is no upper limit on the size of Carrousels.

Surface aerators are usually placed at every other 180-degree turn of the channel. There are no guide walls within the channel in these areas. At the turns not occupied by aerators, semicircular guide walls are used to reduce turbulence losses in the conduit.

The travel time between aerators is very important in achieving substantial nitrate respiration because it does not occur unless DO is reduced to about 0.5 mg/L. To provide for good admixing of the influent flow with the channel content, the untreated process water is usually brought into the aeration zone. Under this plan, nitrate respiration will be mostly in the form of endogenous nitrate respiration. Nitrogen removal may be somewhat enhanced by putting part of the influent into an anoxic zone, thereby providing substrate nitrate respiration. Stringent DO control, achieved by controlling the immersion of the aerators, is also helpful for nitrogen removal.

Data on Carrousels indicate that an effluent of about 5 mg/L BOD_5, accompanied by complete nitrification, is possible. Sludge production is usually about 0.6 kg of VSS per kg of BOD_5 removed, but is higher in plants operated for nitrogen removal. The performance of Carrousels in the United States will probably be somewhat lower because greater fluctuations in hydraulic load can be expected.

Activox Plants

These plants are a new type of looped channel aeration. A 1.4-million-gallon liquid capacity plant is treating the wastes of a sugar refinery at Chalmette, LA. The plant uses two channels of nearly oval shape, in parallel or in series. Vertical surface aerators mounted on horizontal bridges that cross the channel radially near the 180-degree turn are also used. Each channel is provided with one aerator. The channel configuration at the aerator's location differs markedly from the corresponding channel shape of the Carrousels. The Activox reactor is asymmetric with respect to its longitudinal center line.

Cox, G. C., et al., "Production of High Quality Effluents Using Carrousel and High Rate Filtration Processes," Colloque International, Traitement Combine Des Eaux Usees Domestiques et Industrielles (16-19, Mai, 1978).

The Ash Vale Carrousel plant (see pp 143, 153) has an aeration tank of reinforced concrete with overall dimensions of 84.5 x 29 m subdivided by walls. The tank has a water level depth of 3.4 m in the aeration zones and 2.4 m in the channels. It contains two vertical-shaft aerator sets with rise and fall gear boxes; this allows the depth of immersion to vary according to the DO content of the Carrousel mixed liquor. The DO is monitored and controlled by a probe placed just downstream of one aeration zone.

The recirculation time for a plug of material entering the Carrousel is about 15 minutes. With a retention time of 30 hours, based on crude sewage flow, an average of 120 circuits of the tank would be made before the mixed liquor passes to the final settlement tank. This rapid alternation between oxygenation and anoxic conditions is believed to be responsible for the very high degree of nitrification and denitrification provided by this process.

One advantage of using the plant for denitrification is that it can provide quite substantial power savings (see Figure A15).

The first 24 hours provide close-limit automatic DO control; the second 24 hours give conditions of constant maximum aerator immersion and hence

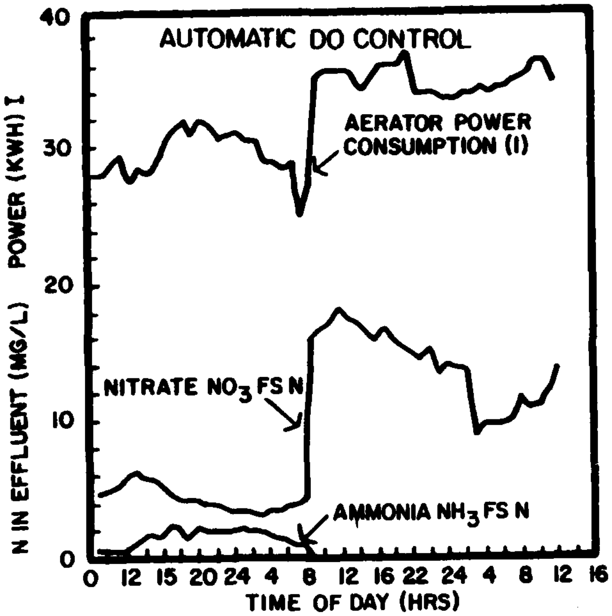

Figure A15. Power, nitrate, and ammonia variation. (From G. C. Cox, et al., "Production of High Quality Effluents Using Carrousel and High Rate Filtration Processes," Colloque International, Traitement Combine Des Eaux Usees Domestiques et Industrielles [16-19, Mai, 1978].)

maximum DO input. During the first 24 hours, nitrates were around 5 mg/L and ammonia at above 2 mg/L; the power consumption varied with the diurnal load. The second 24-hour period showed a marked increase in nitrate to about 12 mg/L, while ammonia levels were less than 0.1 mg/L. The power consumption for the first half of this survey was 25 percent lower, when good denitrification occurred; however, in the second period, there was a much poorer degree of denitrification. Therefore, it can be concluded that power savings up to 25 percent can be obtained by using the close-limit DO control system. A higher degree of denitrification can also be achieved at the same time with only a slight deterioration in effluent ammonia levels.

Ettlich, W. F., A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, EPA-600/2-78-051 (U.S. Environmental Protection Agency, 1978).

Figures A16 through A24, which were taken from this report, show the oxidation ditch performance, reliability and costs of several plants surveyed by the EPA during their study.

Johnstone, D. W. M., et al., "Settlement Characteristics and Settlement Tank Performance in the Carrousel System," paper presented in London for the U.K. Institute of Water Pollution Control (1978).

A Carrousel plant was commissioned at Ash Vale (Thames Water, London) in January 1976. The designed dry weather flow of this plant is 4545 m^3/day with a peak flow equal to three times the dry weather flow. The plant consists of screens and grit channels as well as an oxidation ditch volume of 5750 m^3 and two aerators. Aeration is followed by a 31-m-diameter settlement tank and a

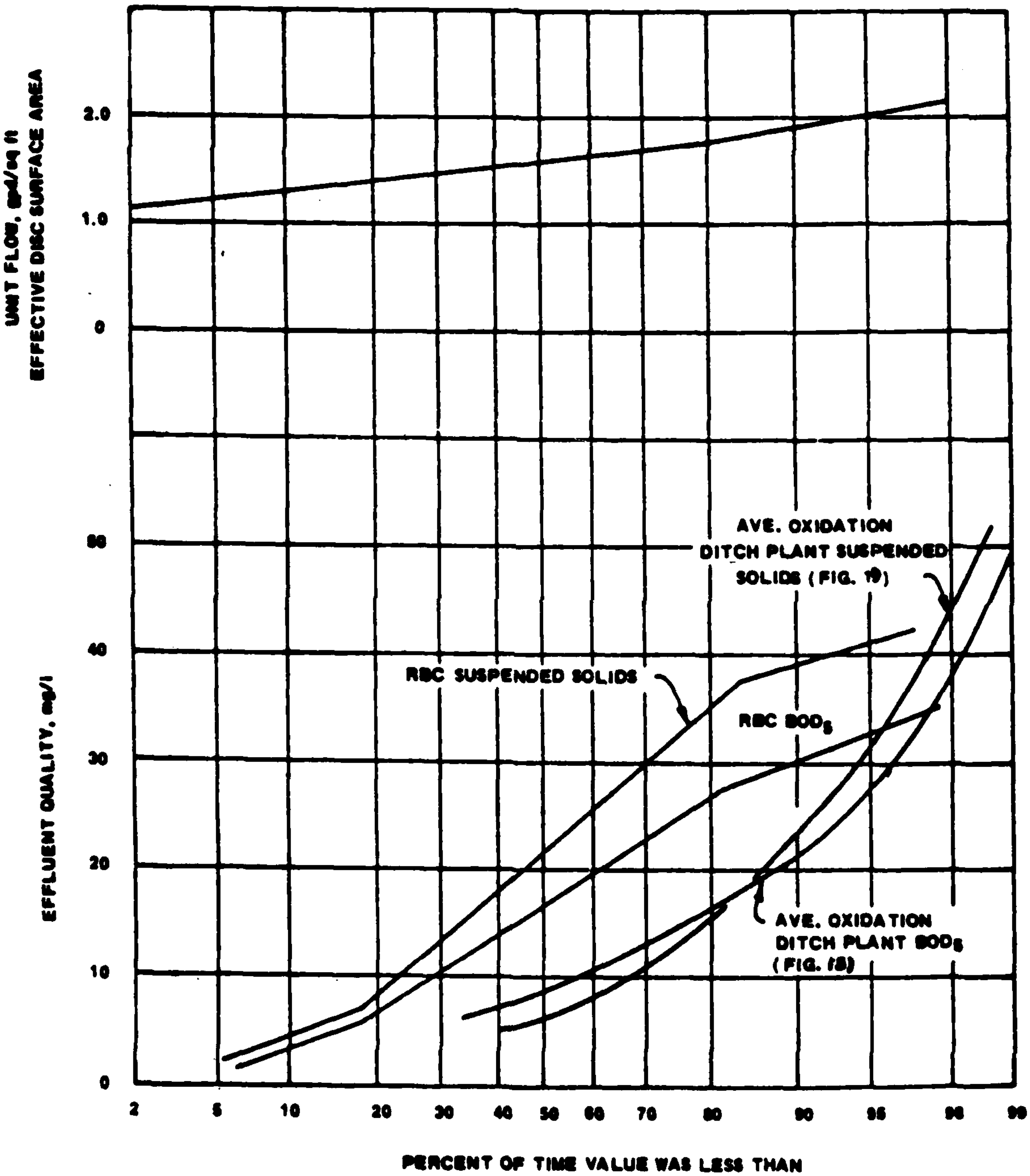

Figure A16. RBC effluent quality, Gladstone, Michigan. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

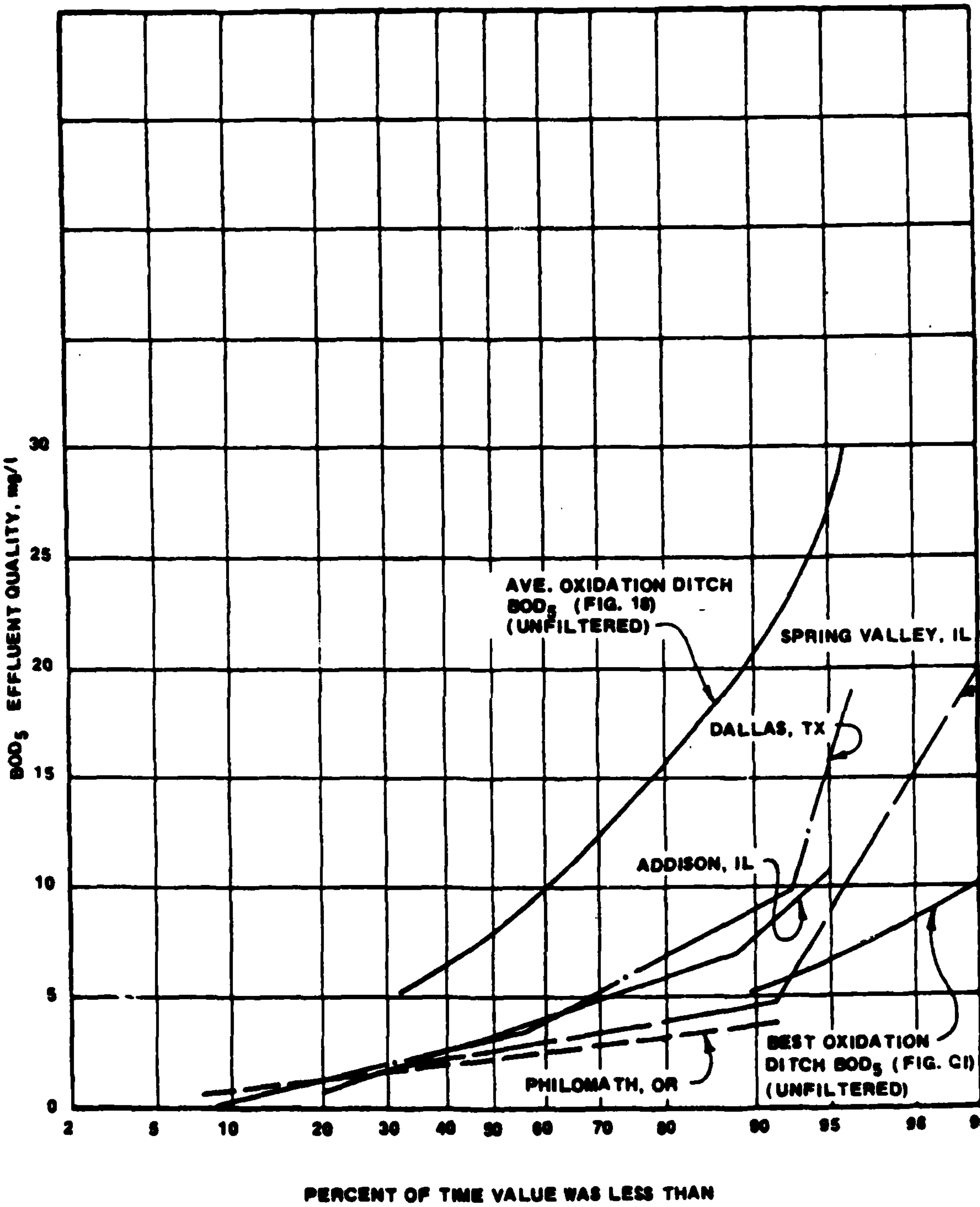

Figure A17. Filtered activated sludge plant BOD_5 quality based on four plants. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

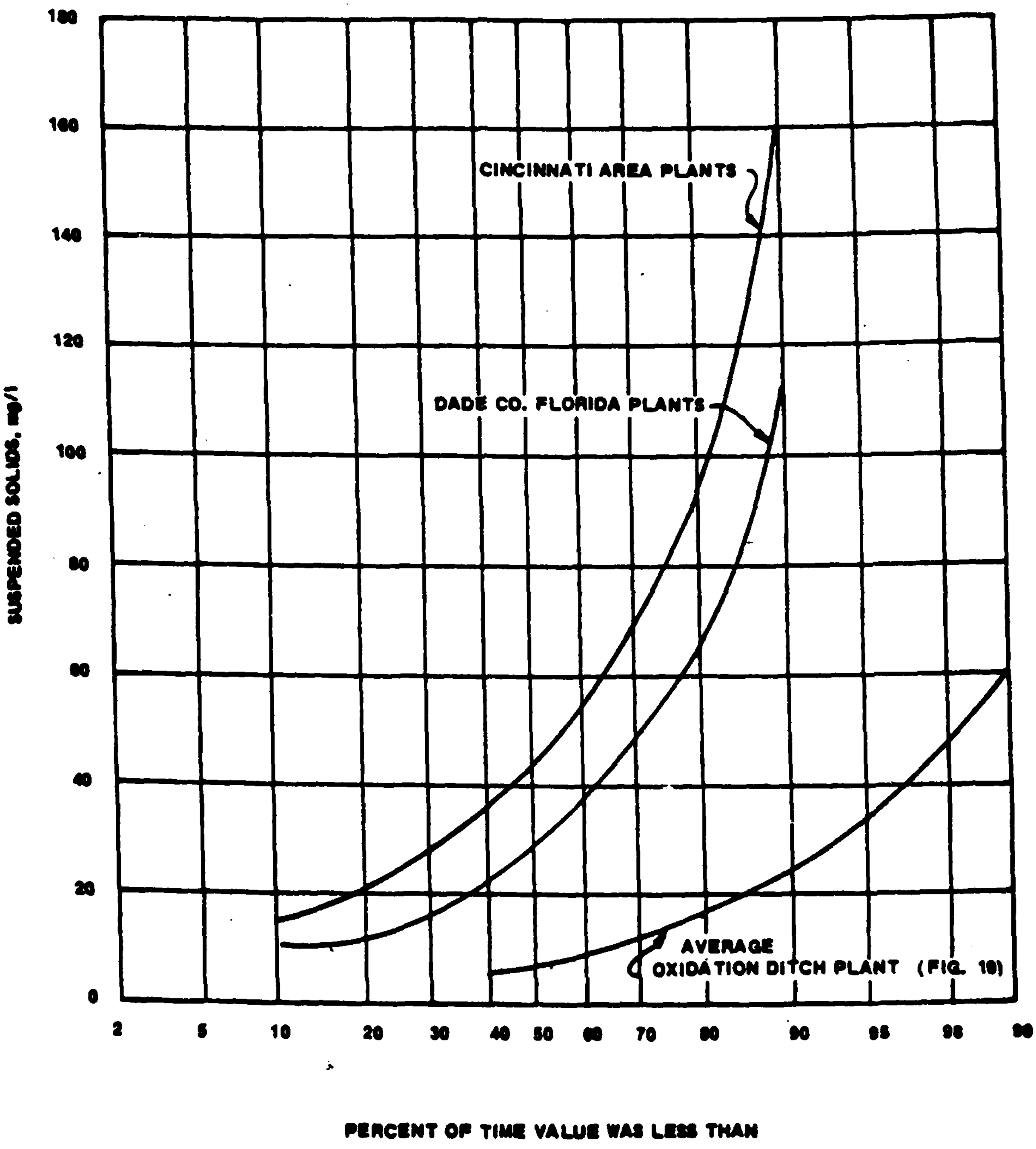

Figure A18. Activated sludge package plant reliability, suspended solids. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

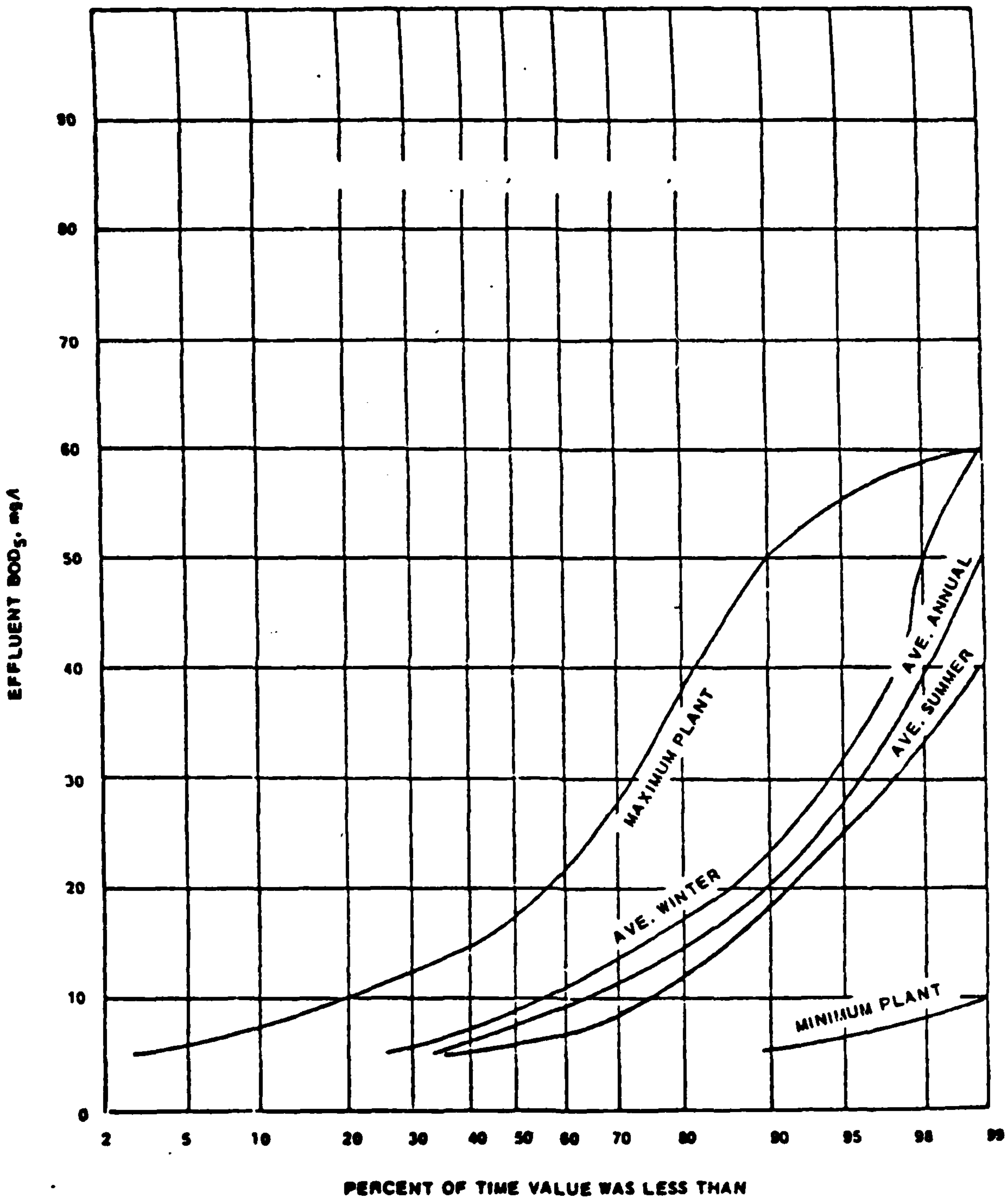

Figure A19. Oxidation ditch plant BOD_5 reliability. (From W. F. Ettich, <u>A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes</u>, 600/2-78-051.)

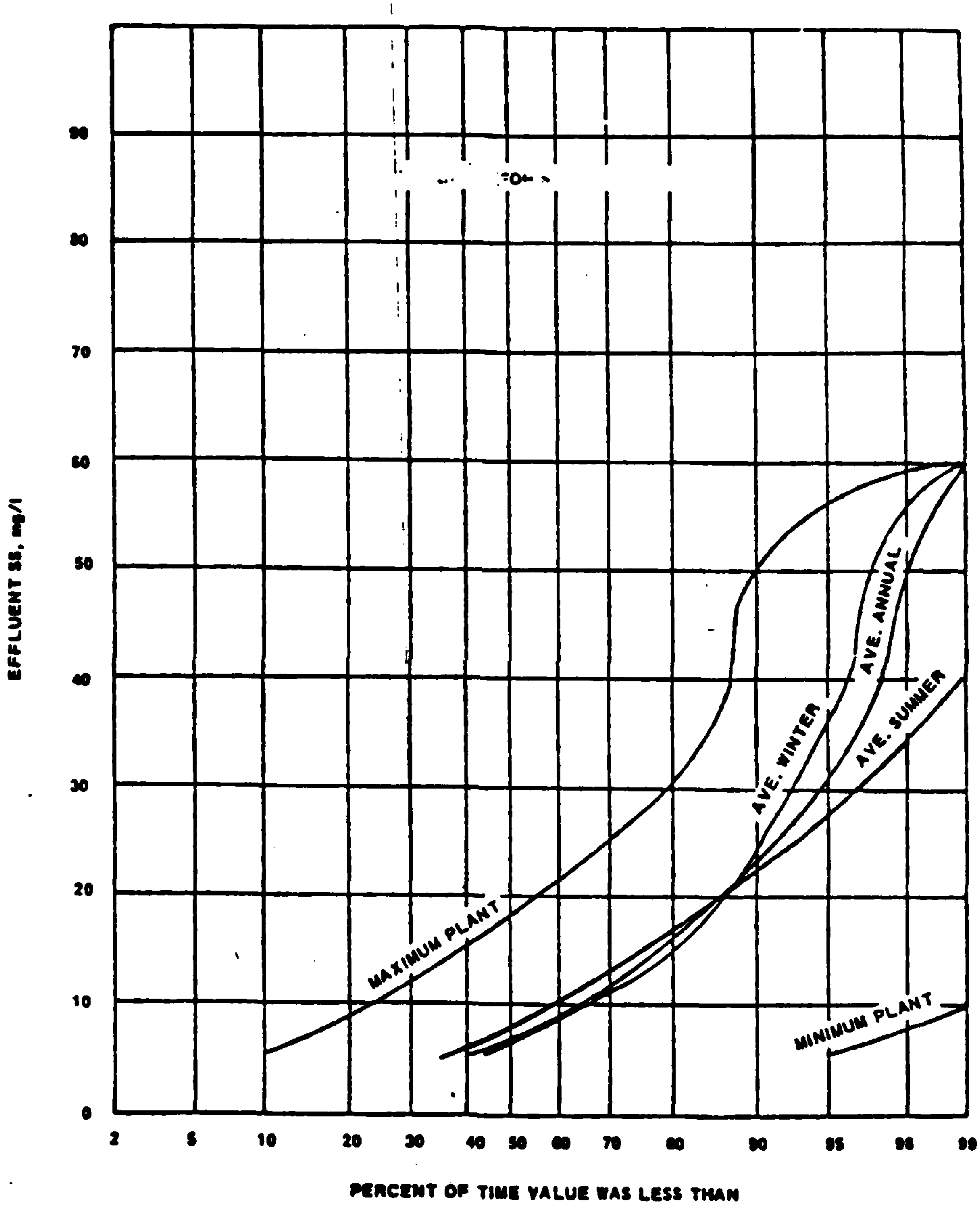

Figure A20. Oxidation ditch plant suspended solids reliability. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

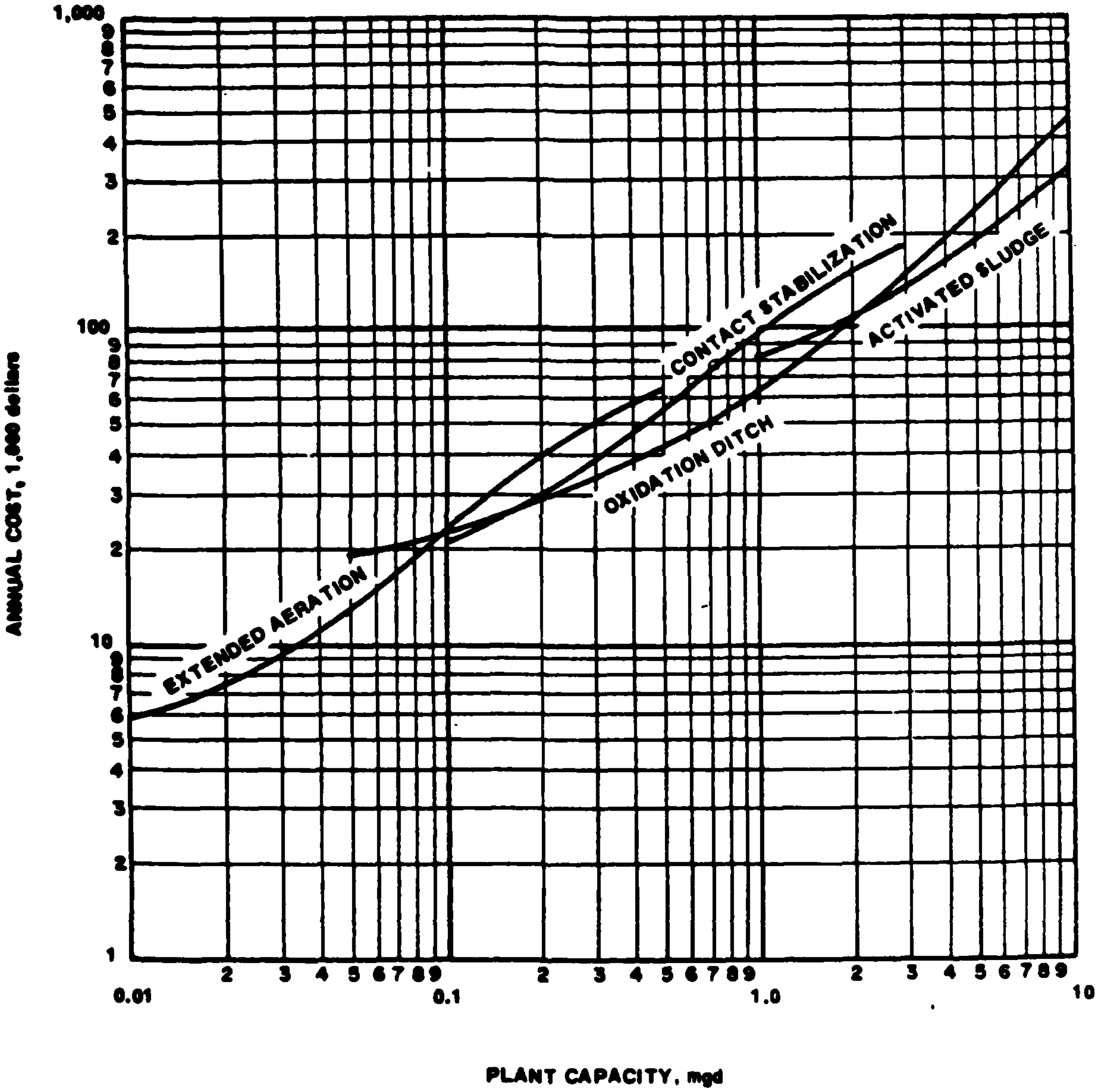

Figure A21. Biological treatment process operation and maintenance cost, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

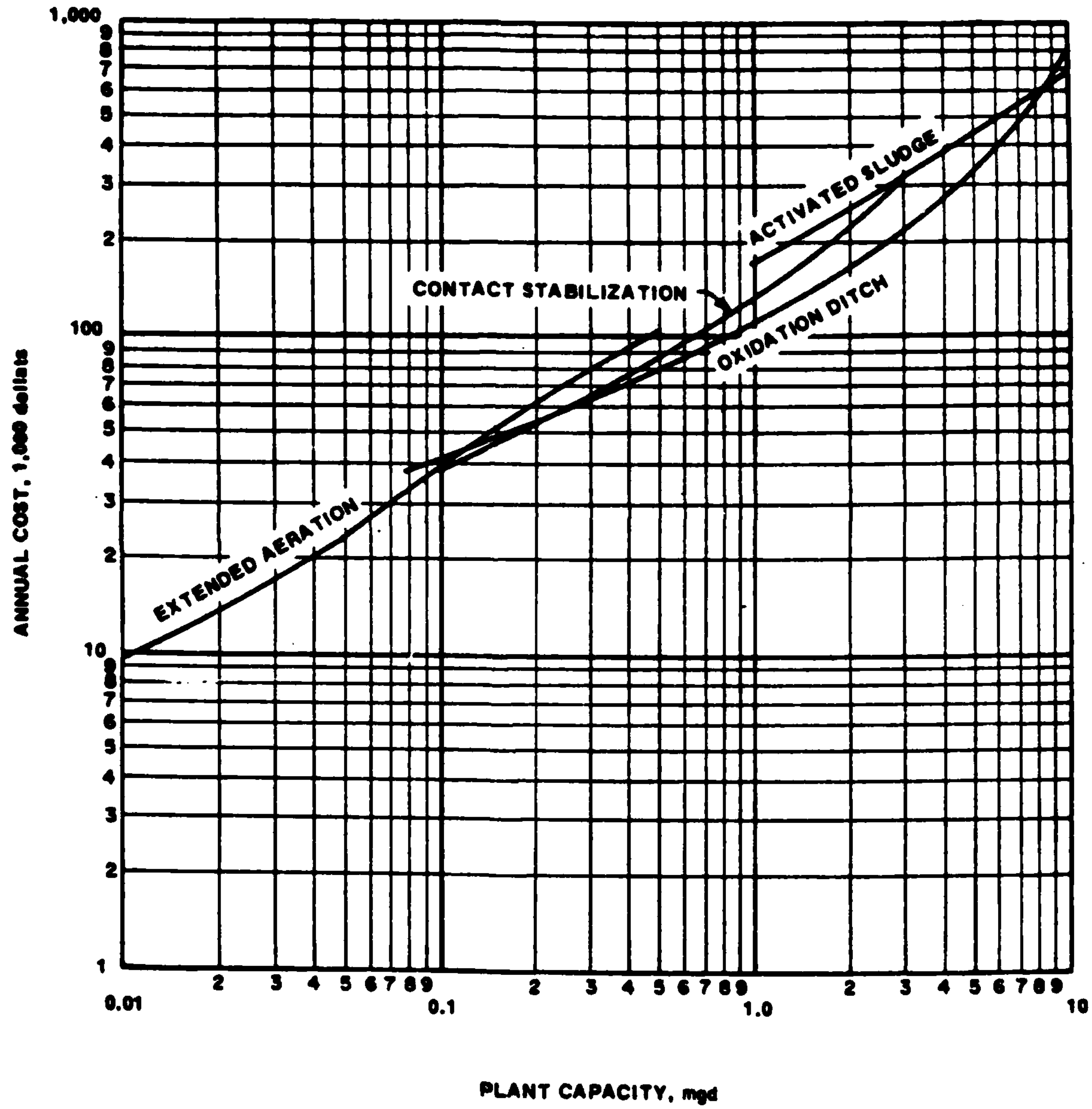

Figure A22. Biological treatment process total annual cost, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

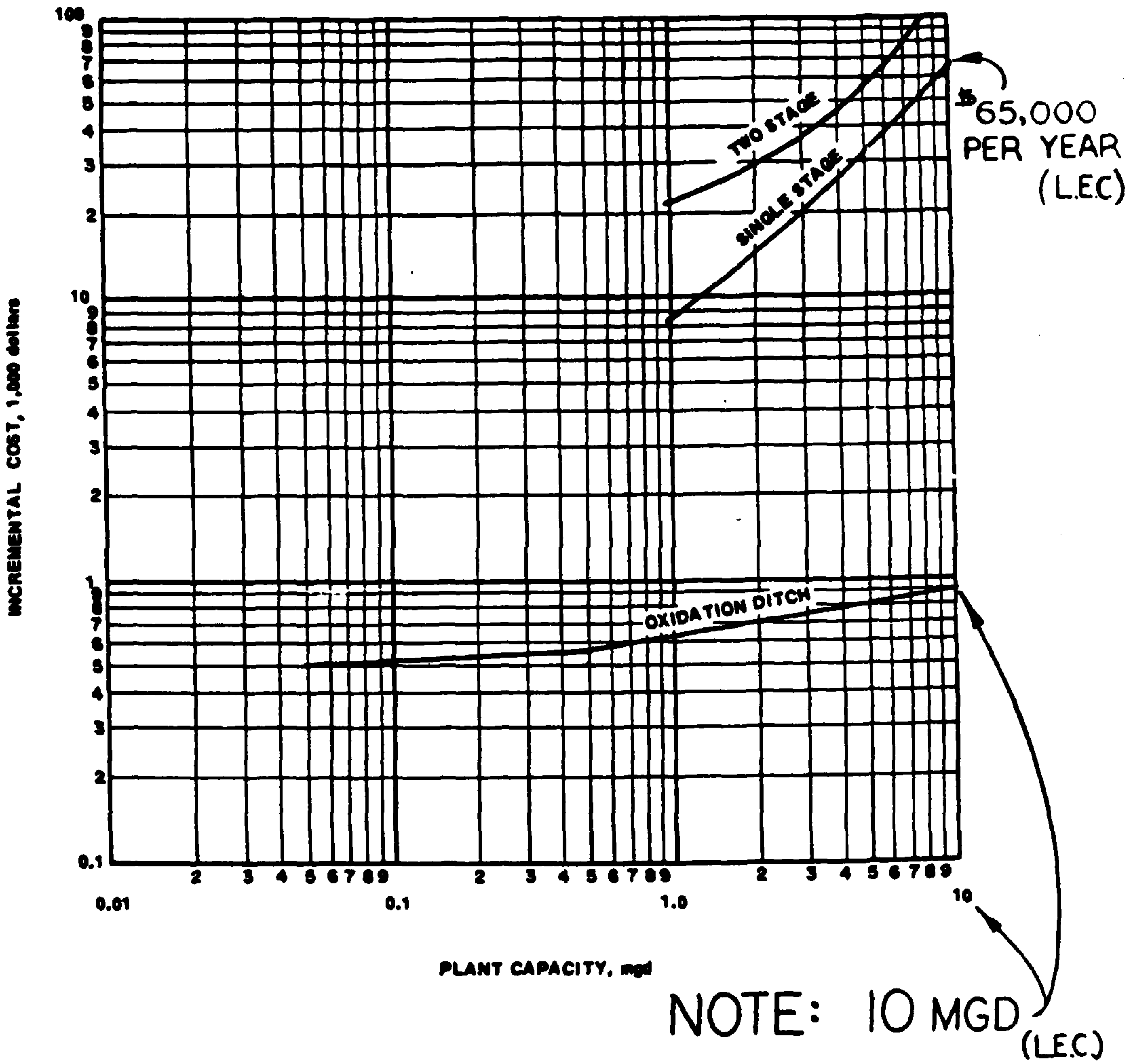

Figure A23. Incremental operation and maintenance cost for biological nitrification, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

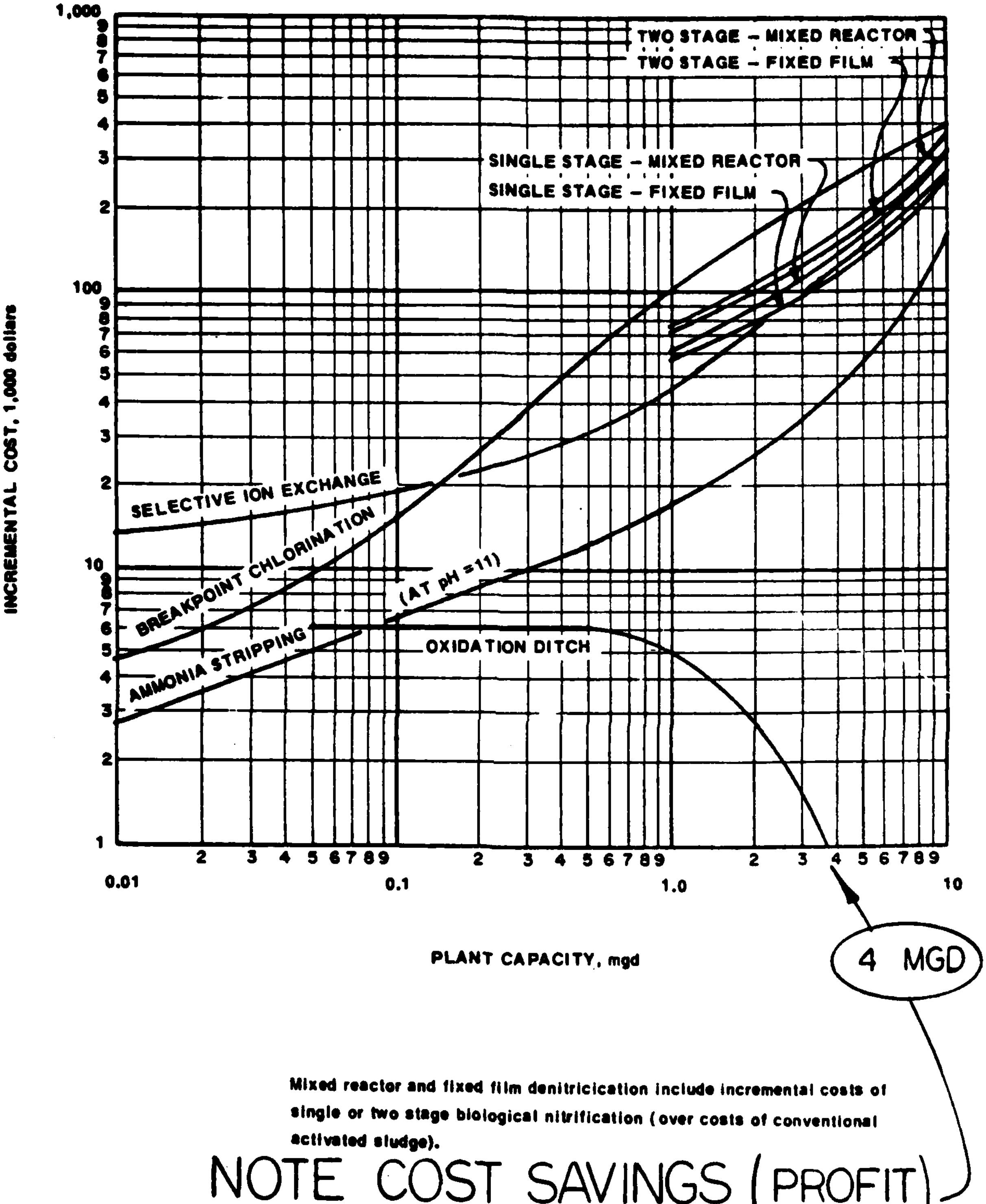

Figure A24. Incremental operation and maintenance cost for biological and physical-chemical denitrification, 1976. (From W. F. Ettich, A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes, 600/2-78-051.)

sludge thickening tank with a volume of 318 m^3. BOD removal was found to be very successful, with an average effluent BOD of 4.0 mg/L from an average influent BOD of 245 mg/L. Ammonia removal ranged from 30 mg/L to 3.0 mg/L. Effluent SS concentration was an average of 3.5 mg/L.

Initially, there was poor SS settling in the final settlement tank, with evidence of filamentous growth. Introducing the total sewage load and total volume of return sludge to one side of the Carrousel system would only increase the plug-flow characteristics. There is now considerable evidence that an anoxic zone at the beginning of such a modified plug-flow system improves settlement.

A Carrousel plant at Cinencester was commissioned in May 1977. Its dry weather flow is 4545 m^3/day with a peak flow of six times the DWF. There are comminutors (25 mm maximum size) and a "pista" grit trap ahead of the Carrousel ditch. The ditch has a 6200-m^3 volume with two aerators, followed by two 32-m-diameter final settlement tanks and two 220-m^3 sludge-thickening tanks. The sludge age for the Cinencester plant is 24 to 31 days (as opposed to 12 to 16 for the Ash Vale plant); this results in a much smaller sludge production of 0.7 to 0.9 kg/kg of BOD removal (compared to 1.0 to 1.2 kg/kg BOD removal for the Ash Vale plant). This will give better settling over several years of service.

The Ash Vale Carrousel system is continually operated at low residual DO levels with large anoxic zones; the DO levels at Cinencester are higher, and the anoxic zones much smaller because there is a different control limit on the aerators. Consequently, the Ash Vale plant shows a very high degree of denitrification; however, the Cinencester plant operates as a nitrifying plant without significant denitrification.

Golding, R. F., et al., "The Pentech JAC Oxyditch,™" paper presented at the 52nd Annual Meeting, Central States, WPC Association, Inc. (May 1979).

Deep basins are possible today because of the application of the jet aeration principle; this provides significant construction and space use economies. Whereas shallow basins use large land areas for a given flow, deeper channels can be placed in a small area of an existing plant. This avoids having to expand the total plant area. JAC Oxyditch designs are presently in operation which range from 0.2 to 12 mgd in various configurations and have operating depths up to 24 ft.

Viranaghavan, T., et al., "Oxidation Ditch Plus Alum Take Phosphorus Away," Water and Sewage Works (1979), pp 54-56.

An oxidation ditch treatment plant was chosen for McAdam, New Brunswick, Canada, on the basis of economics and simplicity of operation. Also, results of an Ontario study demonstrated the feasibility of phosphorus removal in an oxidation ditch treatment plant using alum or ferric chloride. Table A11 lists the salient features of the plant. The designed average flow for the treatment plant is 290,000 gpd, with peak flow at 740,000 gpd. The plant performance summarized in Table A12 shows excellent BOD and P removal.

Table A11

Features of McAdam Plant

(Reprinted with permission from Water and Sewage Works, "Oxidation Ditch Plus Alum Take Phosphorus Away" [1979], by T. Viranaghavan, et al.)

UNIT	CAPACITY AND SIZE	REMARKS
Above-ground package pumping station.	Two pumps, each capable of 250 and 500 Gpm, using two-speed motors.	Station located up-stream of existing septic tank.
Force main	8-in diameter, 600 ft long	
Bar screen	Screen channel 10 ft long, 1 ft 6 in wide and 2 ft deep with a manually cleaned bar screen.	
Parshall flume	6-in throat width	Equipped with float-activated, in-stream flow meter.
Chemical addition tank	5 ft x 5 ft x 7 ft 6 in	Liquid alum added, return sludge, sludge holding tank supernatant, and drying-bed underdrainage lines terminate here.
Oxidation ditch	Capacity is 290,000 gal. Race-track design, 260 ft long; trapezoidal section: 13 ft wide at the bottom, 23 ft wide at liquid level and 5 ft deep (liquid depth); side slopes: 1:1; concrete lined; two 12-ft cage rotors, each driven by 7½-hp motor.	Berms or cuts in the situ material sloped to 1.5:1 and then steepened to 1:1 by adding a compacted layer of pit run gravel; 6-in reinforced concrete slab cast on this layer; perforated drains installed to remove groundwater seepage and leakage through lining (safety feature).
Clarifier	38-ft diameter; 10-ft SWD. Equipped with mechanical scraper.	Two sludge pumps (submersible) each providing 300 Gpm for (a) returning the sludge to the oxidation ditch, and (b) pumping the sludge to the sludge tank or the sludge drying beds; sludge chamber designed deep enough for gravity flow from sludge holding tanks where elevation in the holding tank no longer permits gravity flow to the beds.
Chemical storage and feed equipment	FRP tank: 5,000-gal capacity.	Alum dose paced flow-measuring device installed in the Parshall flume.
Treatment building	Preengineered 72 ft x 58 ft	Covers bar screen, Parshall flume, alum storage, and feed equipment, metering facilities, chlorine room, office, and laboratory and clarifier.
Sludge holding tank	110,000-gal capacity; 38-ft diameter and 16 ft deep (liquid depth)	Has a 5-hp floating aerator for partial aerobic digestion for minimizing odors.
Sludge drying beds	Four, each 31 ft x 100 ft	Handle 458 lb (300 lb biological plus 158 lb chemical) per day.
Chlorination	Contact tank: 7500-gal capacity (15 minutes at peak flow)	Dose paced by the flow-measuring unit at the outlet chamber of clarifier.

Table A12

Characteristics of Raw Wastewater and Treatment Plant Effluent*
(Reprinted with permission from Water and Sewage Works, "Oxidation Ditch Plus Alum Take Phosphorus Away" [1979], by T. Viranaghavan, et al.)

Date D	Date M	Date Y	Flow (Gpd)	MLSS in Ditch	Raw Wastewater ph	Raw Wastewater BOD	Raw Wastewater COD	Raw Wastewater TOC	Raw Wastewater SS	Raw Wastewater TKN	Raw Wastewater Total PO. (as P)	Plant Effluent ph	Plant Effluent BOD	Plant Effluent COD	Plant Effluent TOC	Plant Effluent SS	Plant Effluent TKN	Plant Effluent Total PO. (as P)
19	7	77†	259,000		7.25	36	132		38		2.40	7.1	9.4	47.4		26		0.79
3	8	77**		2,400	6.0	110	450	88	190	48	4.70	5.8	10	53	13	24	16	0.47
3	8	77**		2,300	6.4	130	350	100	170	37	5.70	5.8	10	40	13	15	27	0.43
3	8	77**		2,300	6.0	78	230	120	94	27	3.70	5.7	10	45	12	23	15	0.43
4	8	77**		2,400	5.8	96	450	130	210	85	8.0	5.6	10	41	n9	39	13	0.37
4	8	77**		2,500	5.7	140	400	100	180	42	6.30	5.5	10	38	10	35	3.9	0.40
Avg. 2-6				2,400		111	376	108	169	48	5.70		10	45	13	27	15	0.42

* All concentrations ex-ept pH are in mg/l.

† 24-hr composites.

** 4-hr composites of influent and grab samples of effluent and oxidation ditch mixed liquor.

The total cost of the project including land, legal engineering, and interim financing was $900,000. The annual O&M costs for the treatment plant are about $40,000.

Wiskow, R., "Oxyditch Concept Proven in Pioneer Municipal Plant," Public Works (April 1979).

The St. Charles, MN, treatment plant (see Figure A25) was designed to handle a design flow of 0.8 mgd. With an average BOD of 338 mg/L, the loading is 2255 lb BOD/day. The SS loading is based on 200 mg/L, which amounts to 1330 lb/day.

The plant is essentially two mirror-image treatment systems in parallel. The dual system affords standby capacity, with ample capacity for large flows. Included are two individual 0.4-mgd race tracks (deep channel) which use the proprietary Pentech JAC Oxyditch concept. Directional mix units are used which contain separate air and mixed liquid conduits; air and sewage are discharged uniformly from each jet, producing a fine-bubble plume. Compressed air is supplied by two 1385-scfm blowers located in a building between the two race tracks. The aeration system can transfer close to 5 lb O_2/brake horsepower-hour.

Each clarifier is covered with a fiberglass dome which protects the structure from the severe northern climate; it also provides a significant housekeeping benefit by limiting unsightly algal growth around the clarifier effluent launders. Return sludge is pumped directly from each clarifier to the respective oxidation channels with the timetable determined by mixed liquor SS buildup. Provision is also made for tertiary treatment before chlorination. Three 12-ft-diameter, prefabricated-steel, dual-media filters reduce the effluent SS concentration. The waste sludge is stored in a 92,000 cu ft holding basin. A loading station with a pump transfers the sludge to tank trucks for final disposal on nearby land. Table A13 shows the operational results in 1978 for this plant.

Applegate, C. S., et al., "Total Nitrogen Removal in a Multi-Channel Oxidation System," Jour. Water Pollution Control Fed., Vol 52 (1980), pp 568-577.

Some design features of the oxidation ditch/activated sludge plant are ideal for creating conditions which encourage complete nitrogen removal:

1. The closed-loop design, unique in the ability to maintain solids in suspension with very little applied power

2. Multiple-series aeration channels, which allow isolation of zones favorable for nitrification and denitrification

3. The multiple-channel configuration, which prevents short-circuiting of wastewater and assures complete treatment

4. The disk aerator, which allows maximum control of oxygen input either by changing the number of disks or by changing the disk submergence

5. The concentric channel arrangement, which allows maximum flexibility for raw feed and return sludge addition to one or more channels.

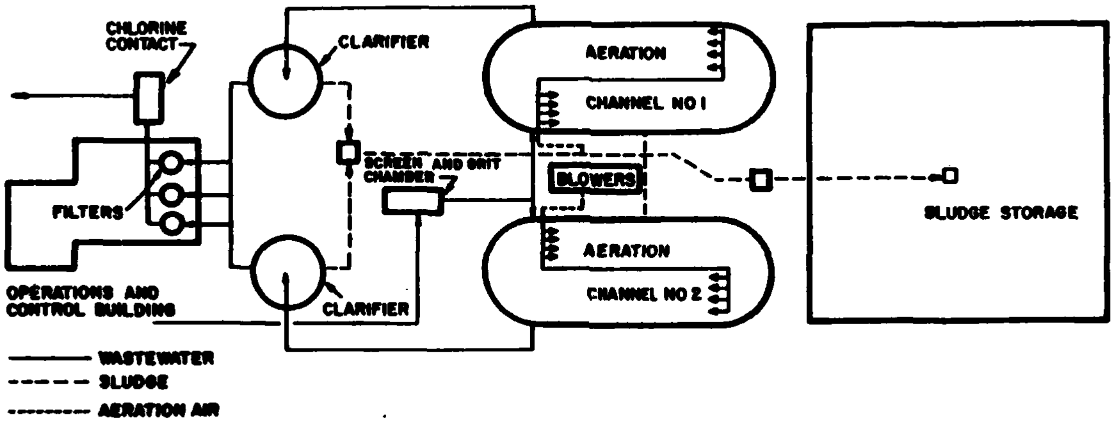

Figure A25. Treatment unit arrangement and flow distribution. (From R. Wiskow, "Oxyditch Concept Proven in Pioneer Municipal Plant," <u>Public Works</u> [April 1979].)

Table A13

Operation Results in 1978 for St. Charles, MN, Plant
(From R. Wiskow, "Oxyditch Concept Proven in Pioneer Municipal Plant," <u>Public Works</u> [April 1979].)

	February	March	April	May	June
Average Flow, mgd	0.4	0.44	0.42	0.4	0.57
BOD, mg/L:					
Influent average	252	201	238	206	221
Influent range	170-378	100-320	227-259	173-238	144-258
Effluent average	2	3	2	4	5
Effluent range	1-4	2-6	1-5	2-7	4-8
Average removal, percent	99	99	99	98	98
Suspended Solids, mg/L:					
Influent average	161	132	129	128	81
Influent range	59-376	94-242	114-150	108-141	99-128
Effluent average	2	2	2	4	5
Effluent range	0.5-4	0.4-14	0.1-5	1-11	2-10
Effluent Dissolved Oxygen, mg/L:					
Average	11.4	11.3	11.9	10	9
Minimum	10	8.7	9.8	7.6	6.4
Maximum	13.2	12.4	16.4	10.8	10.4

Intentional denitrification will allow recovery of up to 63 percent of the oxygen used for nitrification. Because 4.6 kg of oxygen are required per kilogram of ammonia-nitrogen oxidized, combined nitrification-denitrification could be done with just 1.7 kg of additional oxygen per kilogram of nitrogen. Use of raw wastewater as the carbon source allows the oxygen credit to be used for BOD reduction.

Nitrification can destroy 7.14 kg of alkalinity per kilogram of ammonia-nitrogen removed. In some waters with low influent alkalinity, this loss is reflected in depressed pH, which could result in process upset. Denitrification offsets one-half of the alkalinity destroyed by nitrification.

The oxidation ditch/activated sludge plant in Huntsville, TX, has been on-line since 1974. Treating only domestic wastewater, the system was designed for an initial 0.035 m^3/s (0.8 mgd) flow with expansion capacity to 0.70 m^3/s (1.6 mgd) using the same aeration basin. All wastewater flow is pumped to the plant through three main lift stations. Two parallel bar screens, grit chambers, and comminutors preceding a Parshall flume for flow measurement make up the headworks. Without additional pretreatment, the flow enters the aeration basin consisting of four oval, concentric, endless aeration channels in series arranged as in Figure A26.

The total aeration volume is 3030 m^3 (800,000 gal), with 44 percent of the total volume allocated to the outer channel, 28 percent to the second, 17 percent to the third, and 11 percent to the inner channel. The outer channel can be isolated for aerobic digestion of waste-activated sludge or used as the first channel in series with the inner channel for extended aeration operation. Influent from the headworks may be introduced to any of the first three channels. After passing sequentially from the outer to the inner channel, mixed liquor leaves the aeration basin over an adjustable weir located at the center island.

Mixed liquor is propelled in the channels and aerated by a mechanism consisting of several disks arranged on four horizontal shafts. Each aerator shaft is powered by a separate motor and gearbox at 56 rpm. As shown in Figure A26, two aerator shaft assemblies span all four channels, and two span only the first two channels. Oxygen input can be varied over a wide range by changing the number of disks on each shaft or by changing the disk submergence in the mixed liquor.

Clarification is provided by a single 18.3-m (60 ft)-diameter center-feed clarifier. Two pumps are available for return sludge from the clarifiers; normally one pump is operated at its capacity of about 0.028 m^3/s (450 gal/min). Return sludge is pumped to the first aeration channel; however, there is flexibility to introduce it into the second channel if desired. Waste-activated sludge is dewatered on sludge-drying beds. Clarifier effluent is chlorinated and discharged to East Sandy Creek, a tributary of the San Jacinto River.

The South Huntsville Sewage Treatment Plant has demonstrated nitrogen removal (up to 91 percent) through endogenous nitrate respiration in a single sludge system. Very stable organic and nitrogen removals were obtained with no special operator attention. Important factors contributing to the effectiveness of the plant are:

1. Multiple aeration channels in series allow oxygen concentration control in each. Favorable conditions can therefore be created for carbonaceous removal, ammonia oxidation, and nitrate reduction.

2. Splitting the feed to the first two aeration channels enhances nitrogen removal.

3. Removal efficiency for the extended aeration mode of operation is: total COD, 91 percent; SS, 95 percent; NH_3-N, 99 percent; and total N (total TKN + oxid.-N), 76 percent. Removal efficiency for the split-feed mode of operation is: total COD, 89 percent; SS, 98 percent; NH_3N, 98 percent; and total -N, 91 percent.

If the feed is not split, all the nitrogen in the effluent is in the nitrate form. The nitrate nitrogen in the effluent represents nitrogen released from biomass formed in the first channel. In contrast, the effluent after split-feed treatment shows some residual ammonia-nitrogen, organic nitrogen, and nitrate nitrogen. Both nitrification and denitrification occur in each channel, resulting in better use of the entire reactor volume. Rotating disk aerators can provide adequate channel mixing while maintaining minimum aeration for low oxygen concentration. Oxygen input can easily be controlled by either changing the number of disks or controlling the disk submergence.

Besides the obvious benefits from improved effluent quality, complete nitrogen removal allows partial recovery of alkalinity and oxygen used for nitrification. Also, low nitrate concentration in the mixed liquor prevents sludge flotation in the final clarifier.

Aufiers, F. G., "Advanced Wastewater Treatment: Stowe, Vermont." Jour. New England Water Pollution Control Assoc., Vol 15, No. 2 (1981), pp 149-151.

The Stowe, VT, plant (see Figure A27) has two oxidation ditches. Each has a volume of 99,600 gal, providing 28 hours of detention time at the 0.175 mgd design flow. Mixed liquor overflows the ditch effluent weir, is dosed with alum or sodium aluminate for precipitation of phosphorus, then settled in one of two 21-ft-diameter clarifiers. Clarifier effluent is polished in an automatic backwash filter, disinfected with chlorine, dechlorinated with sulfur dioxide, and discharged. Part of the settled sludge is returned to the oxidation ditches. The remainder is aerobically digested, dewatered on a belt filter, and disposed of at the town landfill. The treatment facility cost $1,514,000 and is manned by two full-time employees. One is a class II Vermont operator, and the other is a maintenance assistant. The plant is manned 8 hours per day, 5 days per week, with intermittent coverage on weekends.

Boettcher, G., "A Construction Grants Program Success Story," Public Works (1981), pp 35-37.

An oxidation ditch advanced secondary wastewater treatment plant was built in 1979 for West Plains, MO. The basic design parameters for this facility are:

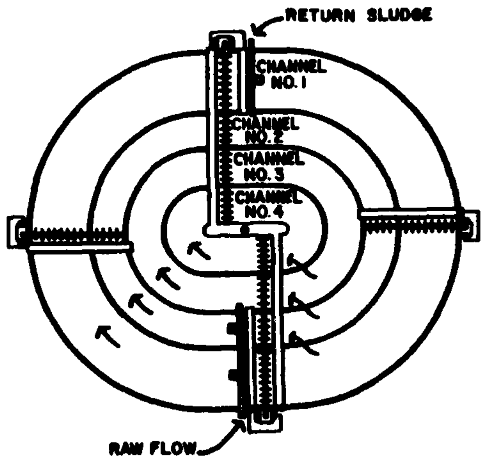

Figure A26. Oxidation ditch aeration basin. (From C. S. Applegate et al., "Total Nitrogen Removal in a Multi-Channel Oxidation System," J.W.P.C.F., Vol 52 [1980], pp 568-577.)

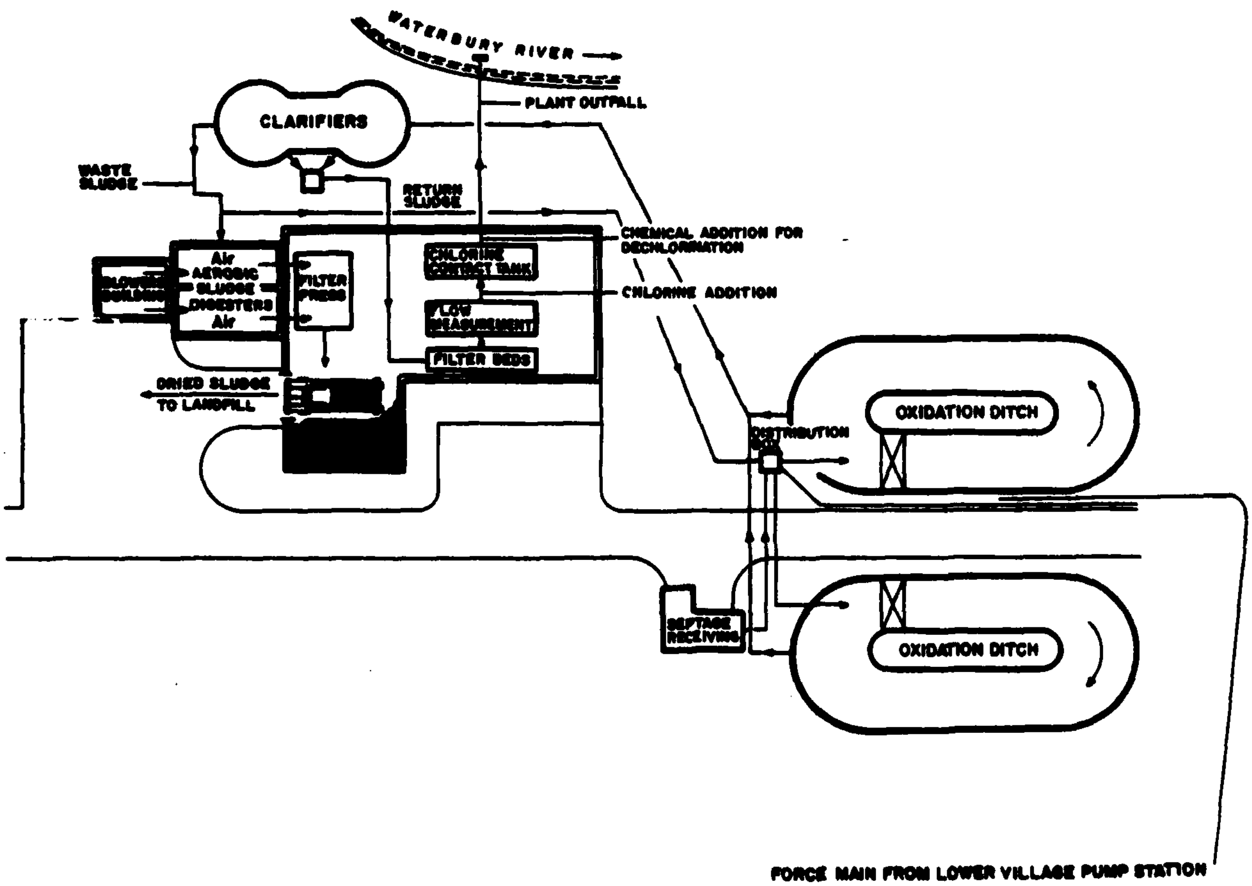

Figure A27. Process schematic for Stowe, VT, Plant. (From F. G. Aufiers, "Advanced Wastewater Treatment: Stowe, Vermont," Journal New England Water Pollution Control Association, Vol 15, No. 2 [1981], pp 149-151.)

Design period............	20 years
Design population equivalent.............	25,000
Design average flow......	2.5 mgd
Design peak flow.........	6.25 mgd
Design BOD loading.......	4250 lb/day

Component loadings at the above design flows are:

Oxidation ditch volume...	2,354,816 gal
Oxidation ditch aeration.	415.5 lb/hr
Clarifiers..............	442 gpd/sq ft
Sludge pumps.............	695 gpm
Chlorinator..............	200 lb/day (max)

The oxidation ditch has a four-zone arrangement, as shown in Figure A28.

The site work cost $230,000, and the plant construction was $3,150,000 for a total project cost of almost $3,500,000. The construction cost was high because 460 piles were needed for foundation works. An initial light organic loading resulted in a slow start because the activated sludge matured very slowly. Fifteen months after start-up, the average effluent BOD was 3 mg/L for the last two months, while SS averaged 1 mg/L.

Chambers, J. V., "Improving Waste Removal Performance Reliability of Wastewater Treatment System Through Bioaugmentation," Proc. 36th Industrial Waste Conference, Purdue University (1981), pp 631-643.

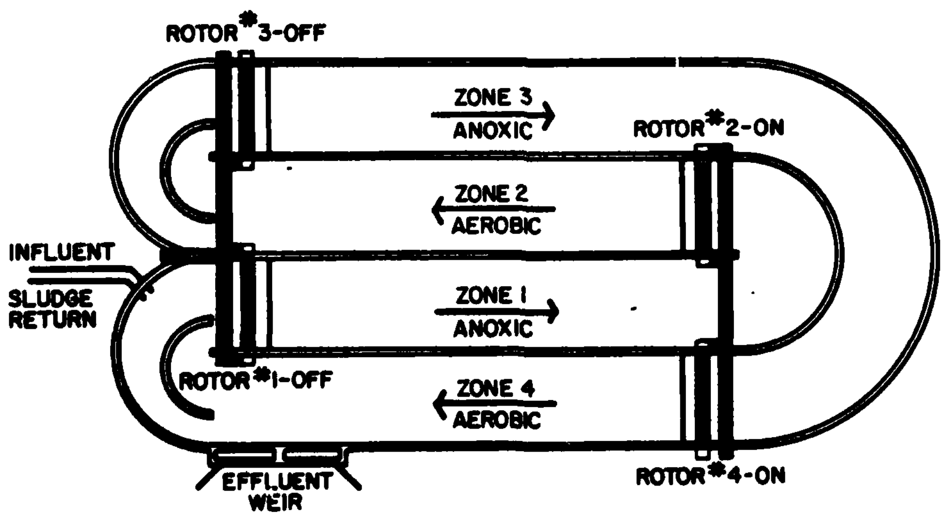

Figure A28. Four-zone arrangement of oxidation ditches. (From G. Boettcher, "A Construction Grants Program Success Story," Public Works, 1981, pp 35-37.)

An oxidation ditch plant which treats daily wastewater has problems with bulking sludge; this leads to poor effluent quality because of excessive loss of suspended solids in the discharge. The sludge volume index is 150 to 375, and the BOD removal efficiency ranges from 85 to 92 percent; the plant receives a hydraulic loading of 75,000 to 105,000 gpd. The organic loading consistently exceeds 100 lb of BOD per day. Detention times range from 2 to 2.7 days with a food-to-microorganism ratio (F/M) of 0.2 to 0.3 maintained. Sludge age is about 8 days.

The returned sludge is chlorinated. This is followed by a bioaugmentation program in which the oxidation ditch is seeded with a commercially available bacteria DBD-Az (Flow Laboratories, Inglewood, CA). After 3 weeks of seeding, there is evidence of a population shift. A stabilized microbial population is established by the fourth week and maintained for 9 months. Detention times remain around 2.5 days with an established F/M ratio of 0.1 to 0.15. Sludge age is increased to 16 days, and BOD removal efficiency is increased to 98 percent or more. The bioaugmentation program is then withdrawn for 12 months; during this time, the bacteria population is thinned out, and BOD removal efficiency decreases. Finally, the bioaugmentation program is resumed, and the desired bacteria population is reestablished. BOD removal efficiency returns to the 98 ± 1.5 percent level for the system. Occasional bulking occurs, but is corrected quickly by chlorinating the returned sludge stream.

Sanks, R. L., et al., "Operator Training is Key to Oxidation Ditch Start-up and Operation, Jour. Water Pollution Control Fed., Vol 53 (1981), pp 444-450.

Optimal operation is most likely when there is close cooperation between the designer and the operator. The O&M manual, hands-on training, cooperative experimentation, monitoring of results, and continued designer advice are interrelated parts of a good program. Engineering contracts should be written to include the cost of such work over a full season of operation.

To promote operational ease and flexibility of design, O&M manuals should be written during the design stage. They should be available for use before start-up; later they should be modified to reflect the best operation of the plant.

Based on experience with various training programs, it is evident that one of the best ways to train operators is by closely supervised hands-on operation. Eight to twelve days of this type of training spread over several months is equal to many weeks of seminars or unsupervised experience; such training can produce competent operators for small plants in only a few months. In the short run, hands-on training seems expensive. However, for a plant at Colstrip, MT, hands-on training costs were less than 6 percent of the first year's total outlay for operation and amortization.

Modern methods of operation (such as those based on sludge retention time) are not difficult for the well-trained operator. These methods provide for excellent control of activated sludge plants with reasonable labor costs. Total operating and laboratory time for the Colstrip plant is 40 manhours per week.

Cold climates create special problems for oxidation ditches. These problems occur because of freezing and are aggravated by intermittent operating conditions and cooling caused by long detention times. Nevertheless, if certain operating procedures are followed, these problems can be overcome even under severe conditions.

Lecompte, A. R., Deep Channel Secondary Effluent Treatment (Kimberly-Clark Corp., 1981).

A mean velocity of 1 ft/sec is desirable for mixing. Both pump and blower horsepower provide the horizontal thrust. A design velocity calculation accounts for friction, plume, and turning head losses. The unidirectional flow conserves momentum. The result is a range of mixing horsepowers of 35 to 50 Hp per million gallons. Conventional designs are 100 to 200 Hp/millon gallons.

Using the industry (PEMA) method for evaluating submerged aeration devices for standard oxygenation capacity, values of 3.2 to 4.8 lb O_2/wire Hp-hr (4.3 to 6.4 lb O_2/kWh) have been measured. However, this standard rate must be translated into operational values by incorporating α and β.

Cerwick, J. A.. "Innovative Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the 1981 Water Pollution Control Federation Conference, Detroit, MI (October 1981).

This article reports the results of a demonstration project using a newly developed Burns and McDonnell treatment process. The demonstration facility is within the existing Little Blue Valley Waste Treatment Plant. Figure A29 is a flow schematic of the demonstration plant. It is basically an oxidation ditch plant except that it uses a relatively short detention time and an intrachannel clarifier. The intrachannel clarifier returns sludge automatically; this eliminates the need for sludge return pumping and other mechanical devices associated with a conventional clarifier. The result is a considerable reduction in capital and O&M costs.

The clarifier area in the oxidation ditch consists of a section of the ditch which is blocked upstream and downstream from the normal ditch velocity. The area is located in a deepened section of the ditch with a semi-open bottom that allows the ditch flow to pass beneath it. The clarifier section (see Figure A30), consists of inverted "V" section components placed transverse to the direction of flow. The "V" sections:

1. Prevent velocity currents from entering the quiescent zone.

2. Allow mixed liquor to pass upward through the section.

3. Allow sludge to fall downward into the mixed liquor flowing beneath.

In the demonstration study, a maximum screened wastewater flow of 1.3 mgd flows by gravity to the oxidation ditch. Flow can be varied by changing weir elevations. The 0.32 million gallon capacity oxidation ditch has a nominal depth of 6 ft with a 14-ft bottom and side slopes of 1.67:1. The ditch is lined with hand-placed concrete for erosion control. A Lightning down-pumping submerged turbine aerator with a 6-ft-diameter "U" tube discharge is used.

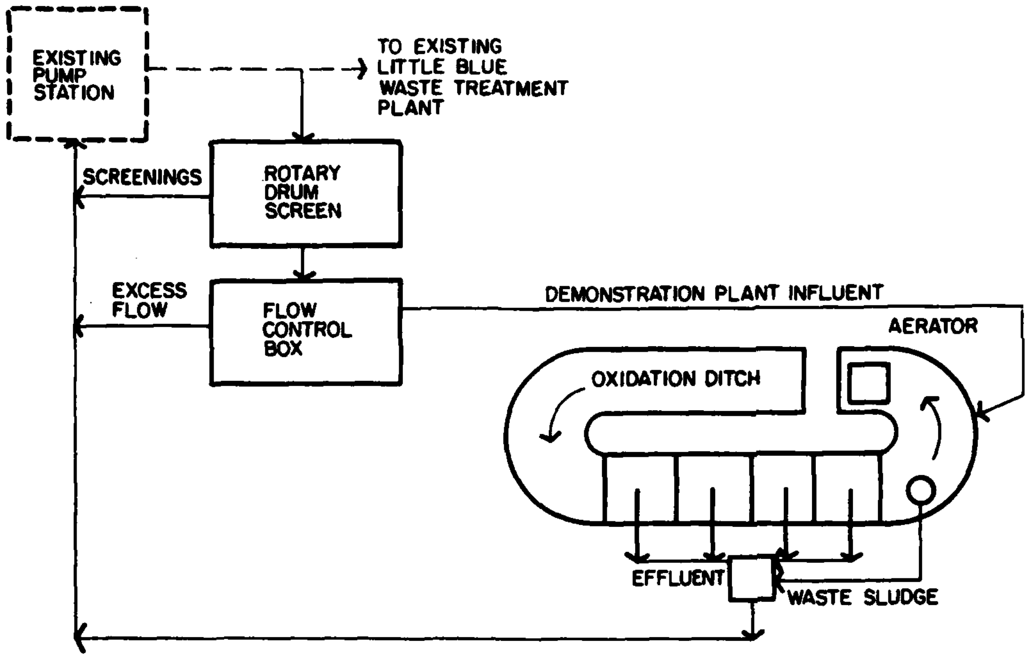

Figure A29. Little Blue Valley Sewer District Demonstration Plant Flow Schematic. (From J. A. Cerwick, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

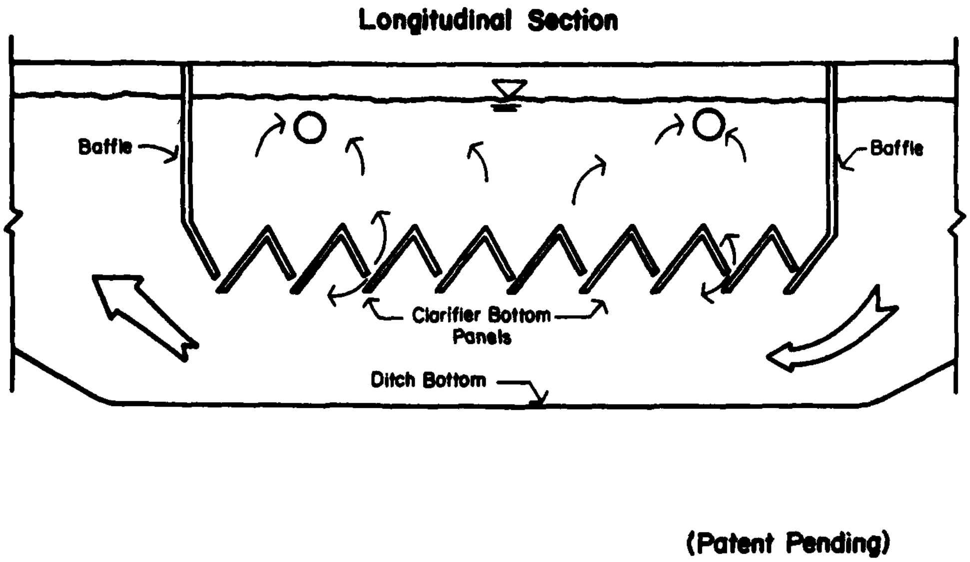

Figure A30. Intrachannel clarification device. (From J. A. Cerwich, "Innovative Treatment Scheme Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

Air is supplied by a 40-hp centrifugal blower, which injects the air at a relatively shallow depth and pumps it downward. The clarification system is installed opposite the aerator. Effluent is drawn from near the surface of the clarifiers through orifices in 6-in. effluent pipes. A Flyst's submersible waste sludge pump is used at the downstream end of the clarification system at the lowest point of the ditch. The pump is used to waste sludge and dewater the facility. As shown in Figure A30, the waste sludge, together with the treated effluent, are pumped back to the influent pumping station wet well of the existing treatment plant.

The ditch velocities vary between 0.5 fps near the inside curve sections to 1.4 fps immediately downstream from the "U" tube discharge. It is estimated that the average ditch velocity is about 1.0 fps. No sludge deposition is detected with this velocity. Dissolved oxygen levels of 2.0 mg/L and 1.0 mg/L downstream and upstream, respectively, of the aerator are adequate in satisfactory BOD removal.

The 5-day BOD reduction has been very good throughout the test period, with a detention time of 20 hr. However, there is a change in quality in comparison to the BOD reduction of the adjacent interim activated-sludge plant. As the detention time is shortened, BOD reduction varies greatly. This is because digester supernatant is periodically discharged to the influent pump station from the overloaded aerobic digester associated with the adjacent interim plant. Changes of digester operation increase the BOD removal efficiency consistently even at detention times as low as 6 hr (average effluent BOD between 10 and 25 mg/L). The viability of the intrachannel clarification concept is proved in this demonstration project. A suspended solids concentration of less than 30 mg/L is achieved consistently, with average levels varying between 5 and 30 mg/L.

Estimated costs projected to 1.0 and 40 mgd plant capacities between four different biological treatment schemes are presented in Tables A14 through A18.

The Burns & McDonnell extended aeration treatment process is the oxidation ditch with intrachannel clarifier, operated with a detention time of about 24 hours; their high-rate treatment process is the same process operated with a detention time of about 6 hours. The cost figures for these processes should be examined with reservation because they are estimated costs at best. These estimated costs should not be compared with the real costs reported by other facilities until the Burns & McDonnell processes can demonstrate steady performance over a longer time and the following three questions can be resolved:

1. What is the efficiency of the aeration and mixing system using a combination of a down-draft pump and a blower in conjunction with a U-shaped draft tube?

2. Why is the short detention (6 hr) in the ditch aeration a high rate complete-mix operation, rather than an extended aeration oxidation ditch operation?

3. Why is there a large liquid volume and low solid concentration in the wasted sludge stream?

Table A14

Estimated Costs for a 40 mgd Plant
(From J. A. Cerwick, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

Conventional Complete Mix		Oxidation Ditch		Burns & McDonnell Treatment Process (Extended Aeration)		Burns & McDonnell Treatment Process (High Rate)	
Primary Clarifiers	2,210,000	Oxidation Ditch	6,660,000	Burns & McDonnell Aeration Basin	7,190,000	Burns & McDonnell Aeration Basin	3,730,000
Complete Mix	4,280,000	Clarifiers	2,140,000				
Secondary Clarifiers	2,180,000	Recycle Pump Station	1,450,000				
Recycle Pump Station	1,460,000						
Aerobic Digester	2,780,000						
Total	12,910,000		10,250,000		7,190,000		3,730,000
Percent Savings with Burns & McDonnell High Rate	71%		64%		48%		

NOTE: Costs for this comparison were developed with the use of COMPUTER ASSISTED PROCEDURE FOR THE DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET). This computer program was prepared for the Department of the Army and the U.S. Environmental Protection Agency.

Table A15

Total Capital Costs for a 40 mgd Plant
(From J. A. Cerwick, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

Conventional Complete Mix		Oxidation Ditch		Burns & McDonnell Treatment Process (Extended Aeration)		Burns & McDonnell Treatment Process (High Rate)	
Preliminary	2,000,000	Preliminary	2,000,000	Preliminary	2,000,000	Preliminary	2,000,000
Primary Clarifiers	2,210,000	Oxidation Ditch	6,660,000	Burns & McDonnell Aeration Basin	7,190,000	Burns & McDonnell Aeration Basin	3,730,000
Complete Mix	4,280,000	Secondary Clarifiers	2,140,000	Gravity Thickener	190,000	Gravity Thickener	300,000
Secondary Clarifiers	2,180,000	Recycle Pump Station	1,450,000	Belt Filters	2,230,000	Belt Filters	2,230,000
Recycle Pump Station	1,460,000	Gravity Thickener	190,000	Mobilization	610,000	Mobilization	460,000
Aerobic Digester	2,780,000	Belt Filters	2,230,000	Clear & Grub & Site	730,000	Clear & Grub & Site	550,000
Gravity Thickener	280,000	Mobilization	770,000	Site Electrical & Control	2,000,000	Site Electrical & Control	1,460,000
Belt Filters	2,230,000	Clear & Grub & Site	870,000	Yard Piping	760,000	Yard Piping	580,000
Mobilization	770,000	Site Electrical & Control	3,670,000	Outfall	1,000,000	Outfall	1,000,000
Clear & Grub & Site	870,000	Yard Piping	1,510,000	Lab & Maintenance & Administration Building	1,570,000	Lab & Maintenance & Administration Building	1,570,000
Site Electrical & Control	3,670,000	Outfall	1,000,000	Profit/Overhead	4,000,000	Profit/Overhead	3,040,000
Yard Piping	1,510,000	Lab & Maintenance & Administration Building	1,570,000				
Outfall	1,000,00	Profit/Overhead	5,040,000				
Lab & Maintenance & Administration Building	1,570,000						
Profit/Overhead	5,720,000						
Total	32,530,000		29,100,000		22,280,000		16,920,000
Percent Savings with Burns & McDonnell High Rate	48%		42%		24%		

NOTE: Costs for this comparison were developed with the use of COMPUTER ASSISTED PROCEDURE FOR THE DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET). This computer program was prepared for the Department of the Army and the U.S. Environmental Protection Agency.

Table A16

Estimated Costs for a 1 mgd Plant
(From J. A. Cerwick, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

Conventional Complete Mix		Oxidation Ditch		Burns & McDonnell Treatment Process (High Rate)	
Primary Clarifiers	171,000	Oxidation Ditch	161,000	Burns & McDonnell Aeration Ditch	256,000
Complete Mix	187,000	Clarifiers	170,000		
Secondary Clarifiers	170,000	Recycle Pump Station	108,000		
Recycle Pump Station	108,000				
Aerobic Digester	177,000				
Total	813,000		439,000		256,000
Percent Savings with Burns & McDonnell High Rate	69%		42%		

NOTE: Costs for this comparison were developed with the use of COMPUTER ASSISTED PROCEDURE FOR THE DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET). This computer program was prepared for the Department of the Army and the U.S. Environmental Protection Agency.

These questions were discussed with a representative of the EPA and one of the Burns & McDonnell staff. Their responses are incorporated in the following sections.

Aeration and Mixing System

Previously, the demonstration project used a Lightning down-pumping submerged turbine aerator with a 6-ft-diameter "U"-tube-type discharge rated at 60 hp. To maintain the mixing requirement and the liquid velocity in the ditch, only 35 hp were needed. Added to the 25 hp required for the blower, a total of 60 hp were needed for proper operation. The energy requirement seemed excessive because much of it was spent to maintain the flow velocity and turbulence for mixing. Now, a new aeration and mixing system is being tested. A Flyst paddle mixer at low speed, 3 1/2 hp, is used along with the 25-hp blower. An air diffuser pipe grid with fine bubble diffusers is placed immediately downstream from the Flyst mixer, as shown in Figure A31. Since the total horsepower is only 28 1/2 hp, a significant savings of energy is anticipated if successful operation of the treatment system with this mixing and aeration device can be demonstrated. There should be about a 25 percent savings of power using this new system when compared with other conventional aeration systems (rotors, turbines, etc.). Test data will be available soon. At this time, the Flyst paddle slow mixer is having many mechanical problems and some major development work. Power savings cannot be proven.

Table A17

Total Capital Costs for a 1 mgd Plant
(From J. A. Cerwick, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

Conventional Complete Mix		Oxidation Ditch		Burns & McDonnell Treatment Process (High Rate)	
Preliminary	100,000	Preliminary	100,000	Preliminary	100,000
Primary Clarifiers	171,000	Oxidation Ditch	161,000	Burns & McDonnell Aeration Basin	256,000
Complete Mix	187,000	Secondary Clarifiers	170,000	Gravity Thickener	61,000
Secondary Clarifiers	170,000	Recycle Pump Station	108,000	Belt Filters	1,035,000
Recycle Pump Station	108,000	Gravity Thickener	61,000	Mobilization	63,000
Aerobic Digester	177,000	Belt Filters	1,035,000	Clear & Grub & Site	76,000
Gravity Thickener	61,000	Mobilization	65,000	Site Electrical & Control	202,000
Belt Filters	1,035,000	Clear & Grub & Site	114,000	Yard Piping	13,000
Mobilization	65,000	Site Electrical & Control	251,000	Outfall	62,000
Clear & Grub & Site	114,000	Yard Piping	118,000	Lab & Maintenance & Administration Building	198,000
Site Electrical & Control	251,000	Outfall	62,000	Profit/Overhead	453,000
Yard Piping	118,000	Lab & Maintenance & Administration Building	198,000		
Outfall	62,000	Profit/Overhead	536,000		
Lab & Maintenance & Administration Building	198,000				
Profit/Overhead	618,000				
Total	3,435,000		2,979,000		2,519,000
Percent Savings with Burns & McDonnell High Rate	27%		15%		

NOTE: Costs for this comparison were developed with the use of COMPUTER ASSISTED PROCEDURE FOR THE DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET). This computer program was prepared for the Department of the Army and the U.S. Environmental Protection Agency.

Table A18

Annual Operation and Maintenance Cost Savings
(From J. A. Cerwick, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

	Conventional Complete Mix	Oxidation Ditch	Burns & McDonnell Treatment Process (Extended Aeration)	Burns & McDonnell Treatment Process (High Rate)
40 mgd Plant				
Preliminary	92,200	92,200	92,200	92,200
Primary Clarifiers	61,200			
Aeration Basin	582,500	1,153,100	1,153,100	582,500
Secondary Clarifiers	66,600	64,500		
Recycle Pump Station	135,200	132,200		
Aerobic Digester	485,600			
Gravity Thickener	19,900	9,900	9,900	22,200
Belt Filters	173,900	173,900	173,900	173,900
Total	1,617,100	1,625,800	1,429,100	870,000
Percent Savings	46%	46%	39%	
1 mgd Plant				
Preliminary	2,600	2,600		2,600
Primary Clarifiers	7,000			
Aeration Basin	28,000	34,300		28,000
Secondary Clarifiers	7,300	7,200		
Recycle Pump Station	9,900	9,700		
Aerobic Digester	24,100			
Gravity Thickener	2,600	1,500		2,800
Belt Filters	21,900	21,900		21,900
Total	103,400	77,200		55,300
Percent Savings with Burns & McDonnell High Rate	47%	28%		

NOTE: Costs for this comparison were developed with the use of COMPUTER ASSISTED PROCEDURE FOR THE DESIGN AND EVALUATION OF WASTEWATER TREATMENT SYSTEMS (CAPDET). This computer program was prepared for the Department of the Army and the U.S. Environmental Protection Agency.

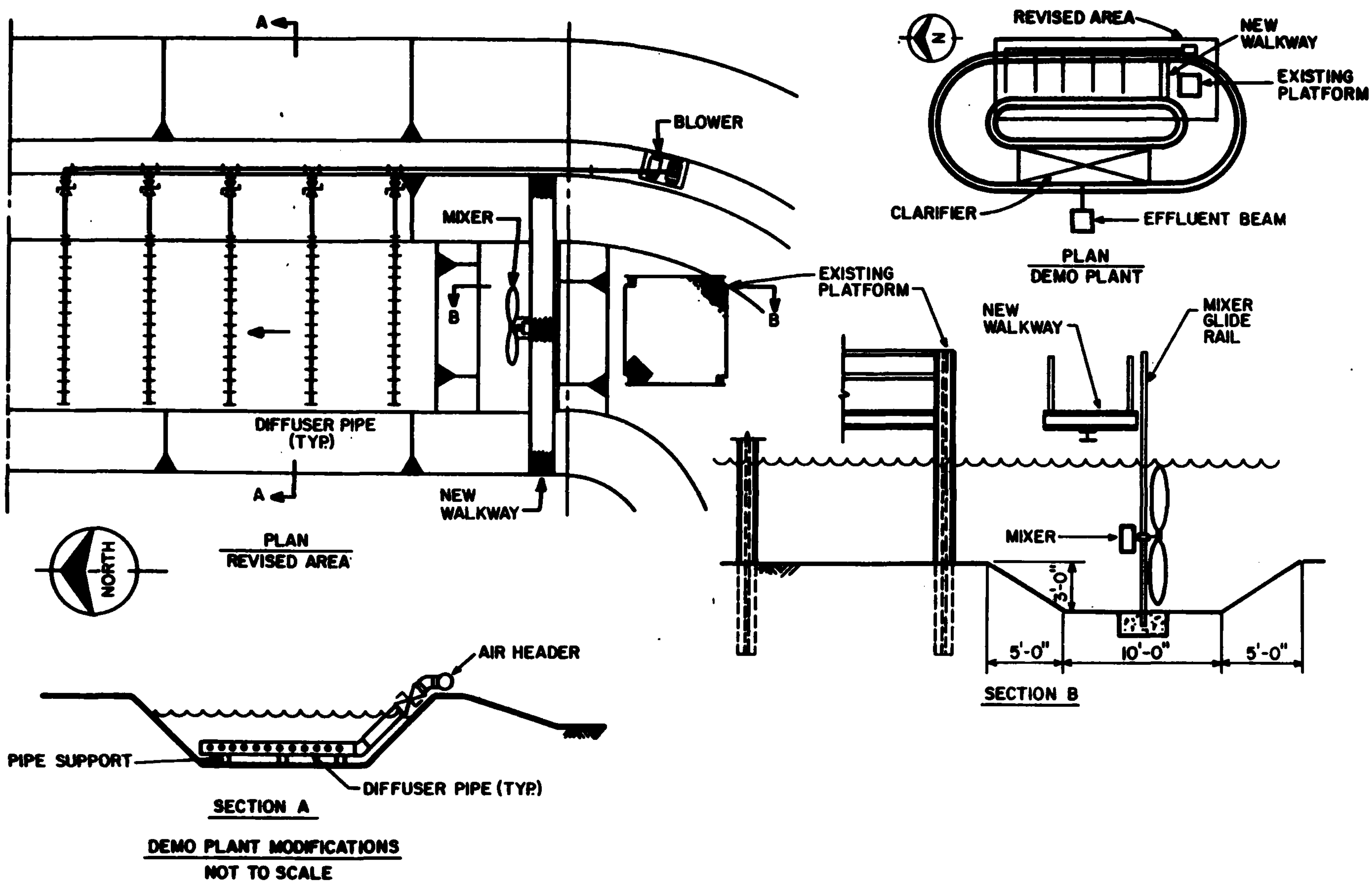

Figure A31. Demonstration plant modifications. (Information for this figure was obtained through personal communications with Burns and McDonnell Corp.)

Short Aeration Time

Normally, an oxidation ditch plant is designed for about a 24-hr hydraulic detention time. This extended aeration mode of operation gives the plant the capacity to equalize possible surge loads to minimize fluctuations in effluent quality. The original Burns & McDonnell treatment process (extended aeration) with the intrachannel design used the same concept. During testing, the hydraulic detention time was shortened to 6 to 8 hr. As the detention time was shortened, more drastic changes in BOD reduction were noted. The BOD treatment efficiency became much more consistent later when the surge discharge of digester supernatant into the plant was eliminated by making operating changes to the digester operation; however, this practice may not be feasible in other treatment plants. A plant with shortened detention time is no longer an oxidation ditch in the traditional sense; closeloop reactor is a better terminology. Without the long duration time, the benefit of equalizing the flow, which insures a consistent effluent quality, is lost. Also a short detention will not allow nitrification to occur, while nitrification is expected in all oxidation ditches operated in the extended aeration mode. Therefore, the cost savings of the Burns & McDonnell High Rate Process should be weighed against the loss of equalization. A decision should be made during design which option (shortened detention time or regular detention time) is more cost-effective in providing the same effluent quality at all times.

Sludge Wasting

Throughout the demonstration project, the wasted sludge is withdrawn with a Flyst sludge pump placed on the ditch channel downstream from the intrachannel clarifier (Figures A32 and A33). The wasted sludge concentration is essentially the same as the sludge concentration in the ditch anywhere else. So far, the maximum concentration withdrawn has been about 3000 mg/L. This is a very diluted sludge concentration when compared with sludge withdrawn from the bottom of a conventional clarifier. Basically, the mean cell residence time or sludge age is hydraulically controlled. This type of control is simpler, but it does have a drawback. The wasted sludge is low in concentration -- about one-third of what normally occurs if sludge is wasted from the bottom of a conventional clarifier. The wasted sludge, which has the same concentration as the mixed liquor suspended solid concentration, needs a clarifier to concentrate its solid content before it can be handled by a sludge digester. It has been suggested that the diluted wasted sludge be discharged to a sludge lagoon for settling and digesting. No matter how it is done, the cost of handling this larger liquid volume of diluted sludge is more expensive than the more concentrated and smaller liquid volume of conventional settled sludge. At the present time, the diluted wasted sludge is simply discharged to the neighboring full-scale treatment plant. However, it is essential that this problem be addressed so that a more realistic cost estimation of this new process can be obtained.

Conclusion

The aeration and mixing system, the concept of sludge wasting by withdrawing the mixed liquid instead of a concentrated sludge, and particularly the cost-saving aspects of these aeration-mixing and intrachannel facilities need confirmation from full-scale plant operational experience. However,

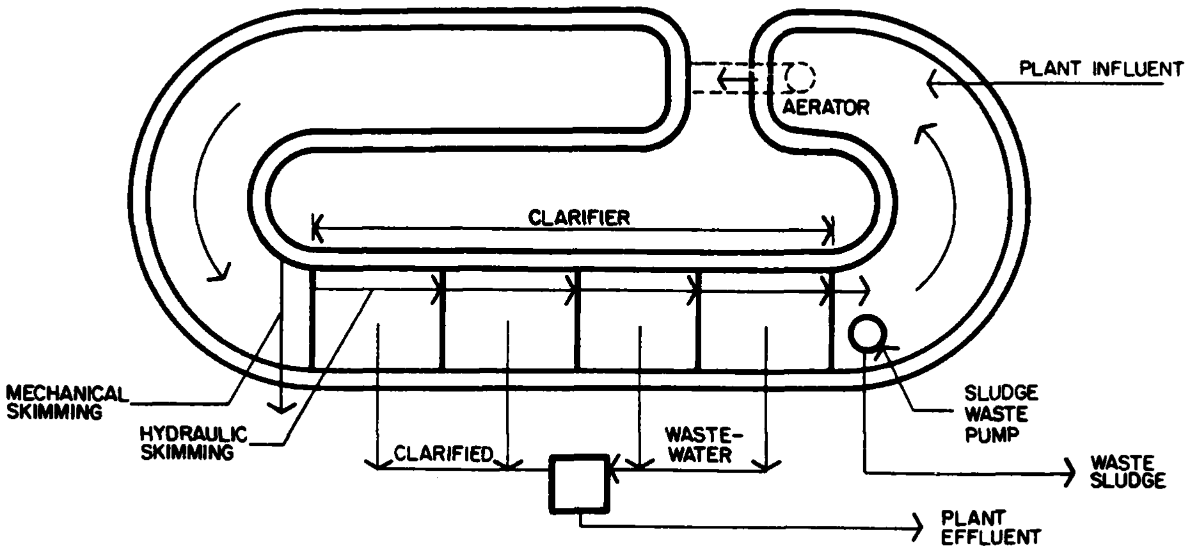

Figure A32. Treatment process with intrachannel clarification device. (From J. A. Cerwich, "Innovative Treatment Scheme Offers Cost Advantages and Operating Simplicity," paper presented at the Water Pollution Control Federation Conference, Detroit, Michigan [October 1981].)

these facilities may be less costly, require less O&M, and find wide application to sewage treatment at Army installations.

Oxidation Ditch Plant Visits

The following information was obtained from visits to three active oxidation ditch plants.

Advanced Wastewater Treatment Facility, Stowe, VT

The basic design data for this plant were provided by the plant superintendent and is shown in Table A19.

The present flow ranges from 82,000 to 90,000 gpd, with a peak flow of 177,480 gpd; both are way below the design flows. The peak flow is mainly the result of groundwater infiltration into the old sewer line. Each oxidation ditch has a 12 ft long Passavant blade rotor of 10-hp, which has a maximum submergence of 8 3/4 in. Because of the flows, a 6-in. submergence is now used. A timer control puts the aerator on a 5-min on and 5-min off cycle. This helps maintain an effluent DO concentration of 1.5 to 3.0 mg/L. However, the aerator is turned on continuously at nights when the ambient temperature is below 0°F. At very low flows, when the detention time is longer than 24 hr, the rotor can freeze during very cold weather. This damages the rotor blades, which must sometimes be replaced. Currently, 10 to 16 hr/yr are required to straighten and repair damaged blades and to service gear box and outside bearing assembly.

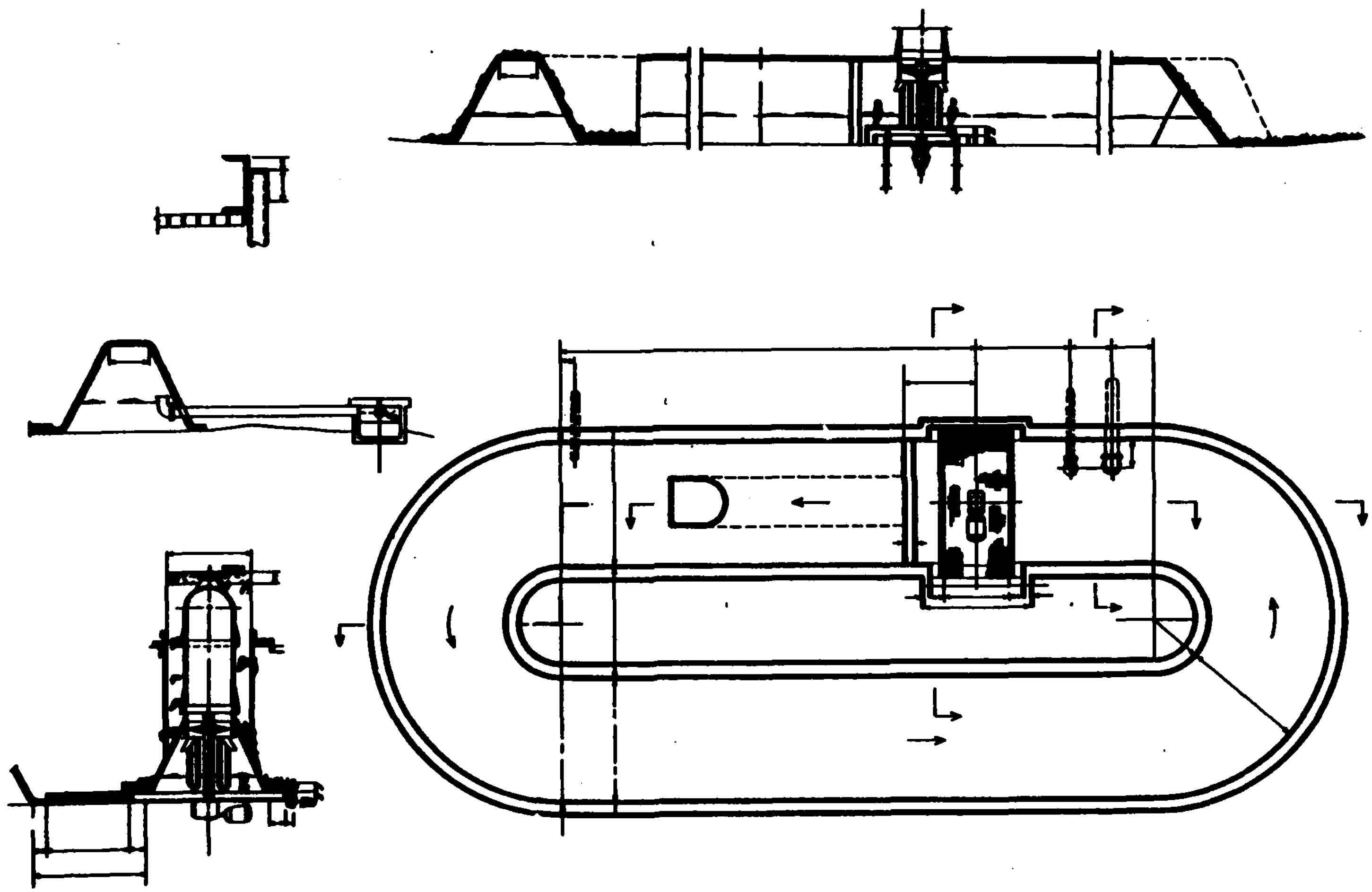

Figure 33. Blue Grass Army Depot oxidation ditch.

Table A19

Stowe, Vermont, Advance Wastewater Treatment Plant
Basic Design Data
(Information for this table was obtained through a personal conversation with Stowe Wastewater Treatment Plant.)

Design Population (Year 2000)	
Permanent	750
Transient	750
Design Wastewater Flows	
Average Daily Flow, gallons per day	173,000
Peak Flow, gallons per day	779,000
Wastewater Loadings	
BOD; pounds per capita per day	0.22
Suspended solids, pounds per capita per day	0.25
Phosphorous, milligrams per liter	16
Screen	
This unit (with bypass channel and manually cleaned screen) is located at the Lower Village Pump Station.	
Oxidation Ditches (2)	
Capacity per Unit, gallons	99,600
Length, 117 ft; width, 52 ft; depth, 4.5 ft	
Detention time at average peak Day Flow, hours	27.6
Final Clarifiers (2)	
Capacity per Unit, gallons	25,900
Diameter, 21 ft; depth, 10 ft	
Detention time at average peak Day Flow, hours	7.2
Surface Overflow Rate at Average Day Flow, gallons per day per square ft	575
Filter	
Length, 20 ft; width, 6 ft; area, 120 sq ft	
Filtration rate, gallons per minute per square ft	1.0
Flow Measurement (by Parshall Flume)	
Throat width, in.	3
Range of Flow, gallons per day	0-700,000
Chlorine Contact Tanks (2)	
Capacity per Unit, gallons	4,530
Length, 8.5 ft; width, 9.5 ft; depth, 7.5 ft	
Dechlorination	
By sulfur dioxide addition to plant effluent.	
Aerobic Sludge Digesters (2)	
Capacity per Unit, gallons	34,560
Length, 30 ft; width, 14 ft; depth, 11 ft	
Waste sludge, pounds per day	210
Per Capita Volume, cubic ft	4.9
Sludge Dewatering (by BeltFilter Press)	
Waste Sludge Loading, pounds per day (dry solids)	160
Chemical Feed Systems	
Chlorine and sulfur dioxide (see above) polymer, sodium aluminate	
Septic Waste Receiving Facility	
Capacity, gallons	2,500
Length, 8.75 ft; width, 8.75 ft; depth, 4.5 ft	
Sludge Production	
Waste sludge to Aerobic Digesters, pounds per day	219
Digested Sludge to Dewatering Facility, pounds per day	160
Dewatered Sludge to Ultimate Disposal, cubic yards per day	1.0

According to the operator, there are fewer filamentous forms of bacteria and less variation in SS concentrations in the oxidation ditch than in the activated-sludge process. When a significant storm occurs (4 to 5 hr), the operator shuts down the aerator. This allows the suspended solids to settle and keeps excessive biological solids from being washed out, which could overload the clarifiers. On the average, the sludge age is 24.2 days. The returned sludge flow is generally 100 to 200 percent of the influent flow. Foaming problems occur only occasionally and seem to correct themselves when the solid concentration changes.

The two clarifiers are housed in a heated building. Settled sludge is withdrawn from each clarifier once every day. The clarified effluent is sent through an automatic backwashing sand filter. However, the sand filter is often overloaded with solids. This causes filter failure and requires extensive cleaning before the filter operation can be restored. The chlorinated effluent carries a residual chlorine of 0.5 to 1.5 mg/L concentration, averaging 0.7 to 0.8 mg/L. No dechlorination is required, although a dechlorination facility is provided. The aerobic sludge digesters are located in an unheated building with a double-thickness fiberglass roof. The belt filter press has never worked since the plant was put into operation in February 1980. Instead, the digested sludge is trucked away to local farmers at the expense of the equipment manufacturer.

The plant accepts septage and both raw and chemically preserved sewage. A 3000-gal capacity storage tank is used for the septage dumping. From the storage tank, the septage is fed slowly into the oxidation ditch at night at a rate of 500 gal/15 min when the sewage flow is low. The storage tank has preclorination and preaeration capabilities, but these have never been used.

Neither oxidation ditch has a drain. The operators find this very inconvenient, since a lot of time and much pumping are required to lower the water level for repair and maintenance.

The plant has two operators. The superintendent is a grade 4 operator, and the other is a grade 1-P certified by the State of Vermont. Both operators work 5-day weeks with occasional weekend duties. A third operator may be needed in the summer to take care of the extensive yard work. In addition to plant operation, and maintenance and repair work, the operators perform daily analysis of sewage temperature, DO, pH, MLVSS, settleable solids, concentrations of returned sludge, wasted sludge, digested sludge, chlorine residue, and the turbidity of the clarified effluent. Coliform number is determined once a week, BOD once or twice a month, and P and COD once weekly. All these measurements take about 5 hr/day. So far, the filtered effluent contains only 2 mg/L of SS and 5.6 mg/L of BOD. This is significantly lower than the state standards of 30 mg/L for both SS and BOD.

A. C. Lawrence Leather Co. Oxidation Ditch Treatment Plants, Winchester, NH

The main activity of the A.C. Lawrence Leather Co. is the processing of sheepskins. A primary treatment system was installed in 1974, and the oxidation ditch was added 1976-1977.

The present average flow is 330,000 gpd, and the peak flow is about 576,000 gpd. A traveling screen removes fibrous material from the raw influent. The sewage pH is adjusted to 9.0 or higher by adding lime, followed by the addition of cationic and anionic polymers to promote coagulation-flocculation. An air flotation tank that follows removes the grease, particulates, and chromium. The coagulation-flocculation step is preceded by equalization in a 180,000-gal tank with no aeration or mixing.

The sludge from the flotation tank, together with the wasted sludge from the oxidation ditch, are pumped into two fiberglass storage tanks. Twice a day, about 1500 gal of the stored sludge is sent to a filter press for processing. The filtered sludge is trucked away for disposal.

The oxidation ditch is a Carrousel design by Envirotech. The ditch is 7.5 ft deep with a liquid volume of 391,000 gal. Two aerators, each with 20 hp rated output, are used with adjustable submergence, varying from 25.5 to 30 cm. The average MLSS in the ditch is 7000 mg/L. The ditch influent contains 0.15 to 2.75 mg/L DO, and its effluent is consistently less than 0.5 mg/L DO.

The oxidation ditch has a problem with massive floating biological solids covering its entire surface. The cause of the problem is not yet known. High residual grease content, high pH, or infestation of filamentous organism growth -- either individually or collectively -- can contribute to the floating sludge problem. The floating sludge can reduce DO in the ditch, although it is not known if this is the result of very high BOD loadings. There is a tentative plan to increase the capacity of the two aerators. Although the ditch effluent is low in DO, the clarifier effluent discharged to the river has an average DO of 7.0 mg/L, because there is a step-aerator at the outfall.

The clarifier is 45 ft in diameter. It is not covered, but experiences no problems during the winter. The returned sludge contains from 8000 to 10,000 mg/L solids and is returned to the oxidation ditch at a rate equivalent to 100 percent of the influent flow.

The plant requires two operators working 8-hr days, plus one operator at nights, supplemented by one operator working on Saturdays. Two 24-hr composite samples per month are collected. The samples are sent to a commmercial laboratory for BOD, total SS, chromium, ammonia-nitrogen, and fecal coliform analyses. Other than occasional high-effluent SS and chromium values, the state effluent standards are met.

Oxidation Ditch Plant at Blue Grass Army Depot, Lexington, KY

Figure A33 shows the plan and sectional views of the Blue Grass Army Depot oxidation ditch.

The plant has a design flow capacity of 17,000 gpd and serves five or six outlying buildings, most of which are workshops, including one laundry shop. These buildings had been served by an Imhoff tank, which is no longer used. According to the design engineer, the present flow is estimated to be only one-third or less of the design flow. The plant capacity was chosen not by the projected future flow, but rather on the smallest practical size of an oxidation ditch that can be built. The design detention time is 18 to 24 hr, depending on the flow and liquid depth in the ditch (the effluent pipeline from the ditch is at a fixed elevation). Because of the low flow, the detention time may now be significantly longer than 24 hours.

The influent goes through either a comminutor or bar screen, a manhole, and then to the oxidation ditch. The downdraft induction aerator unit (Mixing Equipment Co., Inc., Rochester, NY), powered by a 10-hp motor, draws air and wastewater down a 36-in. inside-diameter draft tube which discharges the wastewater about 22 ft downstream near the bottom of the ditch. The average depth of wastewater in the ditch is 4 ft. This aerator-mixing unit is now providing 1 to 1.4 ft/sec surface velocity. DO concentration in the ditch is always 3 to 4 mg/L. However, one section of the ditch -- between the bulkhead near the aerator and the outlet of the draft tube -- serves no purpose at all in the treatment process. This section is more or less a dead space with stagnant flow. Solids settle down in the ditch, floating scum accumulates, and DO is almost 0 mg/L. Thus, it is apparent that this section should be eliminated.

The wastewater is discharged to a 7-ft, 6-in. clarifier housed in an unheated building. A 4-in. sludge pump activated by a timer can return the settled sludge back to the ditch or waste it. Currently, no sludge is wasted. The plant was completed late in 1981, so the solid is building up slowly; it will probably take several more months of operation without wasting before the MLSS can be built up to the recommended level of 4000 to 8000 mg/L. Effluent from the clarifier is going through a V-notch weir chamber with a float-actuated, flow-measuring device; it then goes through a chlorination manhole, and is finally discharged to a creek nearby. The plant is operated by three full-time staffs on split shifts. Their duties also include the operation of a 300,000 gpd primary sewage treatment plant in the Depot in the area central to the residents, as well as a water treatment plant (coagulation-sedimentation-filtration plant). The NPDES permit for the oxidation ditch plants has not yet been issued. At the present time, the mixed liquor SS concentration varies in the low hundreds of milligrams per liter, with an occasional high of 1200 mg/L. Treatment performance is not yet stabilized. Effluent BOD is usually from 20 to 40 mg/L, but occasionally reaches highs of 109 and 375 mg/L. Once the mixed liquor SS buildup reaches the desirable levels (4000 to 8000 mg/L), the treatment performance should be steady and ought to meet secondary treatment standards. Effluent samples in the first few months of operation show a concentration of rather low NH_3-N (less than 0.5 mg/L), and concentrations of NO_2 and NO_3 that are generally less than 1 mg/L. Analyses also show SO_4 concentrations from 20 to 30 mg/L and PO_4-P concentrations from 0.5 to 2.0 mg/L.

Since the aerator-mixer unit is on 24 hr/day and presently provides a high DO concentration in the oxidation ditch, the operators plan to install a timer so that it can be operated on a preprogrammed on-off cycle. This will reduce power costs, since only 1 to 2 mg/L of DO are needed in the ditch.

The simplest construction method was used for the ditch concrete work. A concrete spray gun was used and the concrete was smoothed by hand-trowel, which does not require a high level of workmanship. The total cost of the treatment plant is $158,000, including all materials, equipment, installation, engineering fees, overhead, and administration costs. Plant construction took about 8 months; some time was lost because a couple of major pieces of equipment could not be delivered on time.

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