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Upgrading Lagoons

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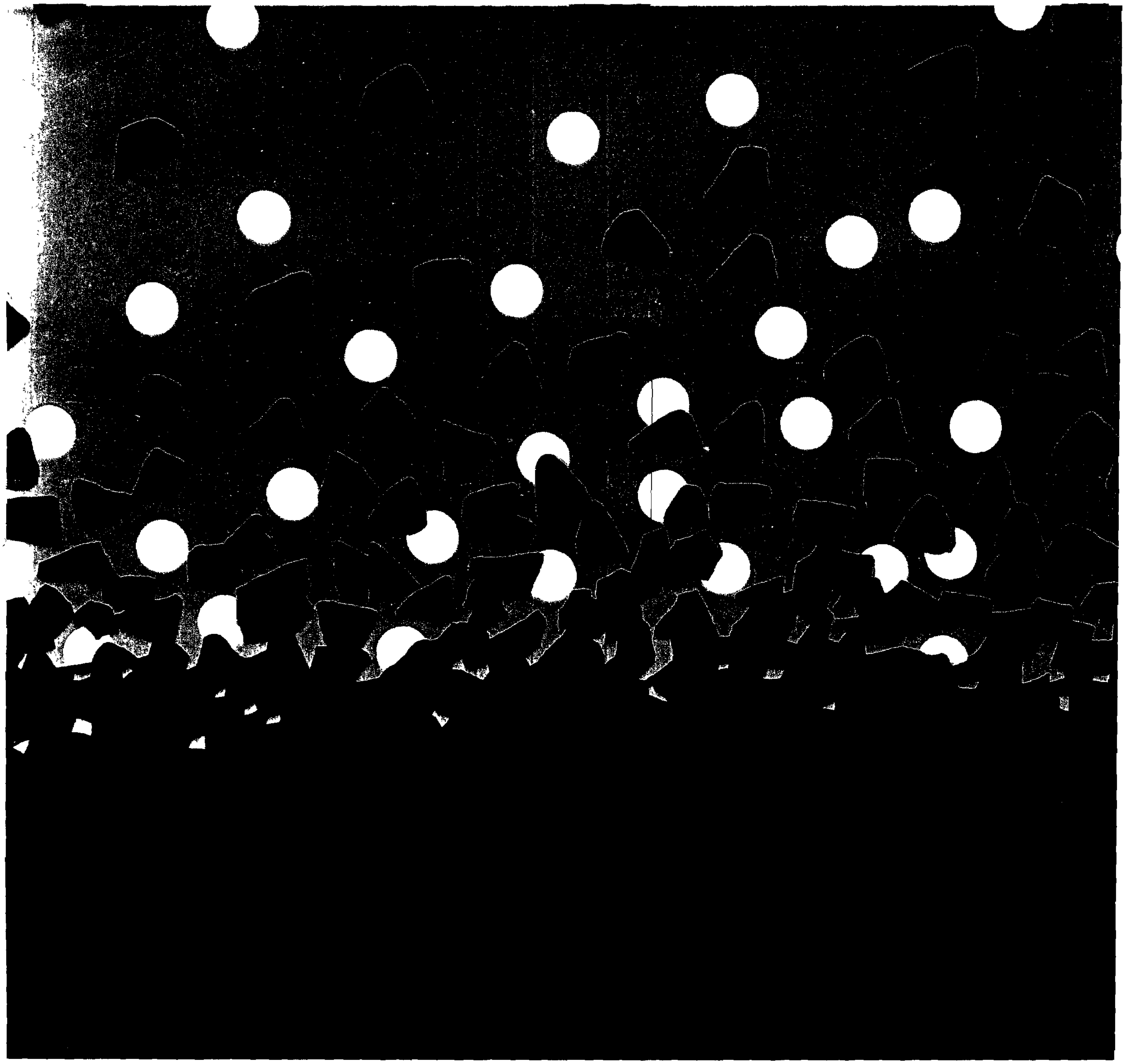

EPA-625/4-73-001b

UPGRADING LAGOONS

ENVIRONMENTAL PROTECTION AGENCY ● Technology Transfer

August 1973

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This seminar publication contains materials prepared for the U.S. Environmental Protection Agency Technology Transfer Program and presented at Technology Transfer design seminars throughout the United States. It is a revised version of a publication originally prepared in August 1973.

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NOTICE

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CONTENTS

Chapter I

LAGOONS IN WASTE TREATMENT

Lagoons are one of the most commonly employed secondary waste-treatment systems. In 1968, treatment systems in the general category of "stabilization ponds" constituted 34.7 percent of the 9,951 secondary treatment systems operating in the United States. Stabilization ponds served 7.1 percent of the 85,600,000 people served by secondary treatment plants. These ponds usually serve small communities; 90 percent were in communities with 10,000 persons or less.[1]

TYPES OF LAGOONS

Waste-treatment lagoons can be divided conveniently into five general classes (table I-1) according to the types of biological transformations taking place in the lagoon.[a] Two of these classes, high-rate aerobic ponds and facultative ponds, are also called oxidation ponds.

High-Rate Aerobic Ponds

In high-rate aerobic ponds, algae production is maximized by allowing maximum light penetration in a shallow pond. These ponds are generally only 12-18 inches in depth and are intermittently mixed. The main biological processes are aerobic bacterial oxidation and algal photosynthesis. Organic loadings range from 60 to 200 pounds 5-day biochemical oxygen demand (BOD_5) per acre per day. Usually 80-95 percent of the waste organic matter is converted to algae.

Facultative Ponds

Facultative ponds are perhaps the most numerous of the pond systems and are deeper than high-rate aerobic ponds, having depths of 3-8 feet. The greater depth allows two zones to develop: an aerobic surface zone and an anaerobic bottom layer. Oxygen for aerobic stabilization in the

Table I-1.—*Types of lagoons*

Type	Depth, feet	Loading, lb BOD_5/acre/day	BOD removal or conversion, percent
High-rate aerobic pond	1 to 1.5	60 to 200	80 to 95
Facultative pond	3 to 8	15 to 80	70 to 95
Anaerobic pond	Variable	200 to 1,000	50 to 80
Maturation pond	3 to 8	<15	Variable
Aerated lagoon	Variable	Up to 400 lb/acre/day	70 to 95

[a]For a complete review of the technology and art of this form of treatment, see references 2 through 4.

surface layer is provided by photosynthesis and surface reaeration, while sludge in the bottom layer is anaerobically digested. Loadings generally range from 15 to 80 pounds BOD_5 per acre per day, and BOD_5 removal from 70 to 95 percent, depending on the concentration of algae in the effluent. BOD_5 removals as high as 99 percent have been obtained.

Anaerobic Ponds

Organic loads are so high in anaerobic ponds that anaerobic conditions prevail throughout. BOD_5 loadings are generally in the range of 200-1,000 pounds BOD_5 per acre per day, and BOD_5 removals are limited to about 50-80 percent. Anaerobic ponds are usually followed by aerobic or facultative ponds to reduce the BOD_5 in the effluent.

Maturation or Tertiary Ponds

The maturation, or tertiary, pond generally is used for polishing effluents from conventional secondary processes, such as trickling filtration or activated sludge. Settleable solids, BOD_5, fecal organisms, and ammonia are reduced. Algae and surface aeration provide the oxygen for stabilization. BOD_5 loadings are generally less than 15 pounds BOD_5 per acre per day, but may be higher.

Aerated Lagoons

Aerated lagoons derive most of their oxygen for aerobic stabilization by mechanical means, either air diffusion or mechanical aeration. Photosynthetic oxygen generation usually does not play a large role in the process. Up to 95 percent BOD_5 removals are obtainable, depending on detention time and the degree of solids removal.

OPERATING PROBLEMS

With increasingly stringent effluent requirements, waste-treatment lagoons, like any other waste-treatment process, may require modification to meet all objectives. The problems that occur with individual ponds, however, may not be common to all.

Organic Matter in Effluents

An algal-bacteria symbiosis operates in both aerobic and facultative ponds. Under aerobic conditions, bacteria degrade organic matter according to the following simplified transformation:

$$\underset{\text{(organics)}}{CH_2O} + O_2 \xrightarrow{\text{bacteria}} CO_2 + H_2O \qquad \text{(I-1)}$$

Under anaerobic conditions, the equation is:

$$2CH_2O \xrightarrow{\text{bacteria}} CH_3COOH \xrightarrow{\text{bacteria}} CO_2 + CH_4 \qquad \text{(I-2)}$$

Algae, in turn, reuse the carbon (as carbon dioxide) to form algal biomass:

$$CO_2 + 2H_2O + \text{energy} \xrightarrow{\text{algae}} \underset{\text{(algae)}}{CH_2O} + O_2 + H_2O \qquad \text{(I-3)}$$

While these equations oversimplify the transformations, they show the recycling of carbon in ponds. Unless the algae are removed, or the carbon is removed through methane fermentation in an anaerobic sludge layer, little organic reduction may occur.[5]

The fate of algae discharged to receiving waters has received relatively little attention, possibly because severe problems have not developed in most instances. Two studies have shown, however, that for two differing aquatic environments the algae did constitute a BOD load on the receiving waters and decreased the dissolved oxygen (DO) levels.[6,7] In these cases, the algae from the pond effluent were in an unfavorable environment for either their maintenance or growth, and they decayed (as in equations I-1 and I-2).

Secondary treatment requirements developed by the EPA limit treatment-plant effluent BOD and suspended solids (SS) concentrations to less than 30 and 45 mg/l on a monthly and weekly average, respectively. Figure I-1 presents average effluent qualities for three types of lagoons.[8] None have BOD or SS concentrations of less than 30 mg/l, and the facultative lagoon, the type most commonly used, has an average SS concentration of 70 mg/l. Figure I-1 clearly indicates that additional

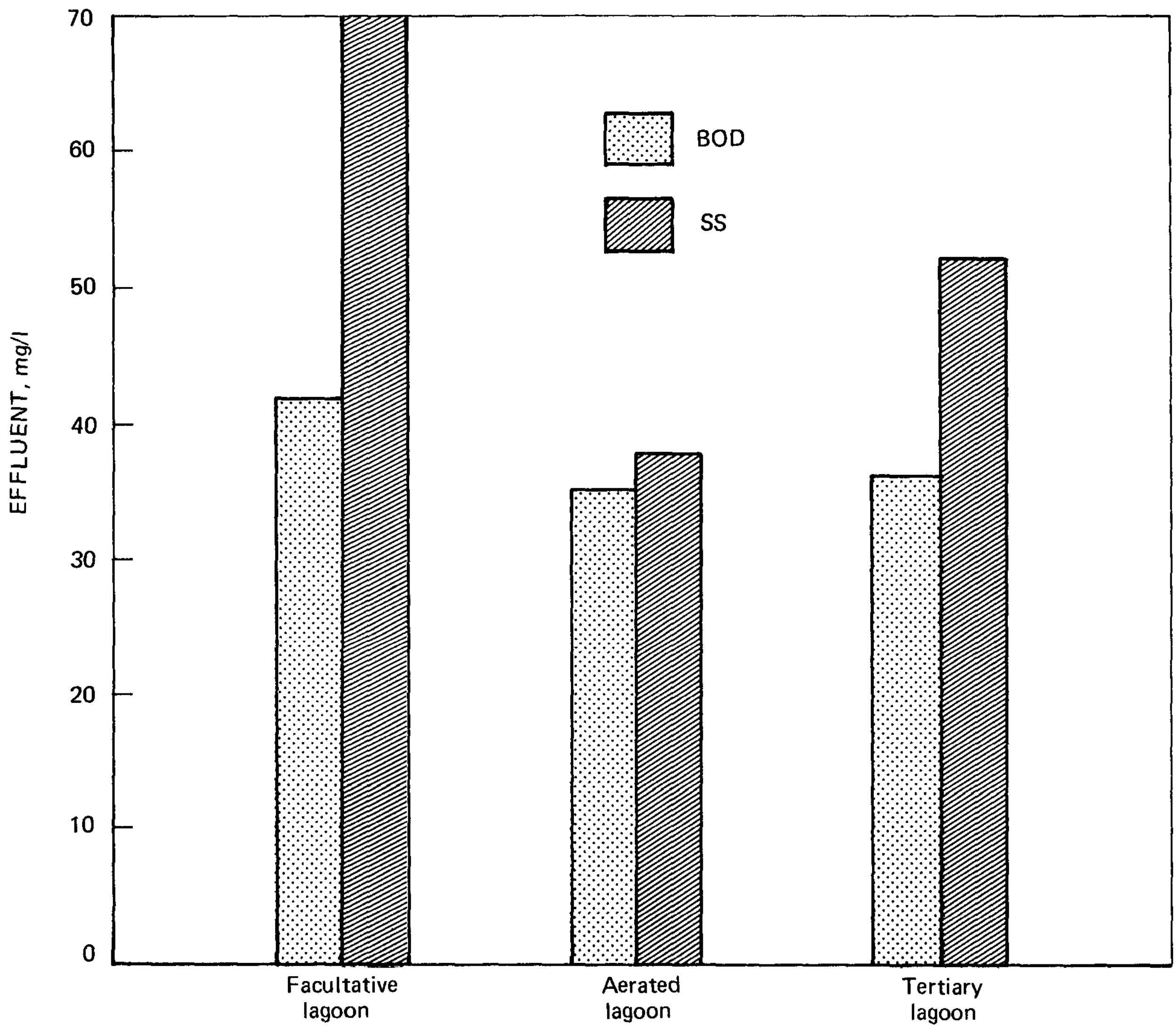


Figure I-1. Performance of lagoon systems.[8]

treatment will usually be necessary to enable pond systems to meet the secondary treatment requirements.

Aerated-lagoon effluents, while not containing large amounts of algae, may contain biological solids resulting from the conversion of a portion of the BOD_5. One aerated-lagoon application achieved only 70 percent BOD_5 removal; the insertion of a final clarifier in the process allowed 90 percent BOD_5 removal because of solids removal.[9]

Odors

That lagoons may occasionally emit odors is shown by the very common State requirements concerning lagoon location, i.e., requirements that lagoons should be located as far from existing or future residential or commercial development as is practical or reasonable. Anaerobic ponds particularly tend to have odor problems due to hydrogen sulfide formation, although some methods have been developed for odor control.

Noxious Vegetative Growths

Without maintenance and good design, aquatic growths may develop in ponds. Deeper ponds (deeper than 3 feet) will discourage rooted growths, and proper levee maintenance can handle shoreline problems. If not suitably controlled, noxious plants can choke off hydraulic operation and create large accumulations of floatable debris. The debris usually becomes septic and creates odors and conditions detrimental to photosynthetic activity.

Seasonal Performance Variations

In most locales of the United States there are seasonal changes in both available light and temperature. Typically, in the winter algae activity diminishes. Biological activity may also slow; methane fermentation in facultative ponds may practically cease.[5] Thus, in winter BOD_5 removals may be low. In Michigan, no discharge is permitted until the spring thaw when increased biological activity causes a lower effluent BOD_5.[10]

Despite operating problems, which certainly have not occurred with every lagoon application, lagoons have been providing economical treatment at thousands of locations for decades. Low capital cost, simplicity of operation, and low operation and maintenance costs have favored lagoon treatment. Considering both more stringent water-quality criteria and environmental constraints posed by encroaching suburbanization, however, many lagoons will have to be upgraded in both treatment efficiency and mode of operation.

REFERENCES

[1] Federal Water Quality Administration, "Municipal Waste Facilities in the United States," No. CWT-6, Washington, D.C., 1970.

[2] *Second International Symposium for Waste Treatment Lagoons*, Missouri Basing Engineering Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[3] Missouri Basin Engineering Health Council, "Waste Treatment Lagoons—State of the Art," EPA-WQCRS 17090EHX, Washington, D.C., July 1971.

[4] U.S. Environmental Protection Agency, Office of Water Programs, "Wastewater Treatment Ponds," Technical Bulletin, supplement to *Federal Guidelines: Design, Operation, and Maintenance of Wastewater Treatment Facilities*, EPA 430/9-74-011, Washington, D.C., Mar. 1974.

[5] W. J. Oswald, A. Meron, and M. D. Zabat, "Designing Waste Ponds to Meet Water Quality Criteria," *Second International Symposium for Waste Treatment Lagoons*, pp. 186-194, Missouri Basin Engineering Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[6] R. C. Bain, P. L. McCarty, J. A. Robertson, and W. H. Pierce, "Effects of an Oxidation Pond Effluent on Receiving Water in San Joaquin River Estuary," *Second International Symposium for Waste Treatment Lagoons*, pp. 168-180, Missouri Basin Engineering Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[7] D. L. King, A. J. Tolmsoff, and M. J. Atherton, "Effect of Lagoon Effluent on a Receiving Stream," *Second International Symposium for Waste Treatment Lagoons*, pp. 159-167, Missouri Basin Engineering Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[8] G. Barsom, U.S. Environmental Protection Agency, Office of Research and Development, *Lagoon Performance and the State of Lagoon Technology*, EPA-R2-73-144, Washington, D.C., June 1973.

[9] L. A. Esvelt and H. H. Hart, "Treatment of Fruit Processing Waste by Aeration," *J. Water Pollut. Cont. Fed.*, *42*, 7, 1305-1326, 1970.

[10] Maurice M. Richmond, "Quality Performance of Waste Stabilization Lagoons in Michigan," *Second International Symposium for Waste Treatment Lagoons*, pp. 54-62, Missouri Basin Engineering Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

Chapter II

UPGRADING LAGOONS THROUGH PROCESS MODIFICATION

Many of the techniques available for upgrading lagoons treating primary and secondary effluents have already been incorporated in designs at one or more locations—often in the original construction and not as a modification. A well-designed pond will incorporate physical features that minimize upsets, maintenance, and nuisances, and maximize operational flexibility, stability, and biochemical oxygen demand (BOD) removal. Physical design features that should be considered include configuration, recirculation, feed and withdrawal variations, pond transfer inlets and outlets, dike construction, supplementation of oxidation capacity, and algae removal. These features will be discussed in this chapter.

The discussion that follows will center on lagoons treating primary or secondary effluents. Many of the waste-treatment lagoons in the United States, particularly in the Midwest, treat raw sewage. One of the ways to upgrade such lagoons is to add primary or primary-plus-secondary treatment ahead of the ponds.

POND EFFICIENCY VERSUS POND LOADING

It is fairly well established that pond-process performance is affected by both areal BOD loading[a] and detention time.[1,2] Typical data for canning wastes are shown in figure II-1. A similar,

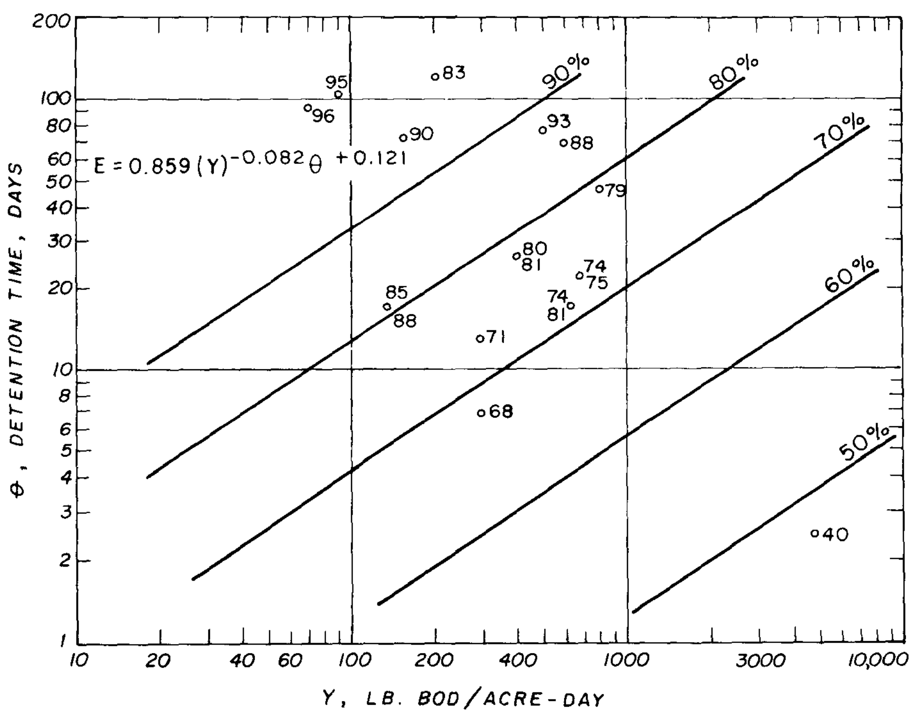


Figure II-1. BOD-removal relationship for ponds treating cannery wastes.[1]

[a]Except for aerated lagoons, where areal BOD loading is not an appropriate design criterion.

but not necessarily identical, empirical relationship would apply to domestic wastes. Figure II-1 shows that pond performance can be improved by three techniques.

- Increased detention time will increase BOD removal and can be accomplished by deepening the pond. The most probable cause of improvement would be increased algae sedimentation.
- Decreased areal BOD loading will increase the BOD removal by decreasing the carbon to be processed (and recycled to algae). This decreased loading can be accomplished by pretreatment; e.g., placing a primary sedimentation unit before the pond in a system formerly using only raw-sewage ponds.
- Decreased aeral BOD loading and increased detention time can be accomplished by increasing the number of ponds in the system.

POND RECIRCULATION AND CONFIGURATION

Pond recirculation involves interpond and intrapond recirculation as opposed to mechanical mixing in the pond cell. The effluents from pond cells are mixed with the influent to the cells. In intrapond recirculation, effluent from a single cell is returned to the influent to that cell. In interpond recirculation, effluent from another pond is returned and mixed with influent to the pond (see fig. II-2).

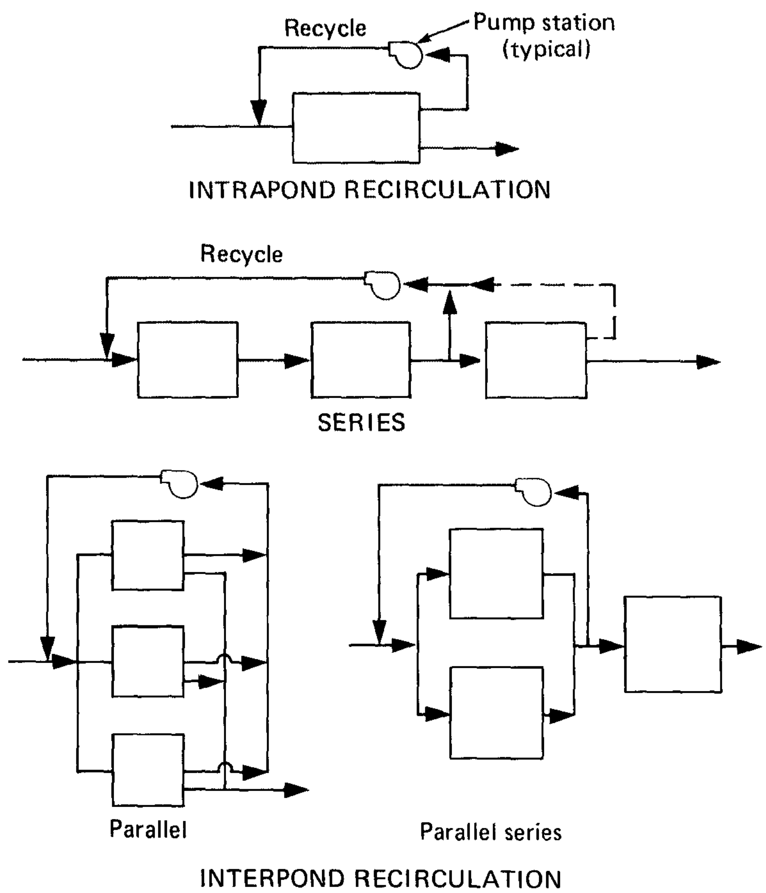


Figure II-2. Common pond configurations and recirculation systems.

Both methods return active algal cells to the feed area to provide photosynthetic oxygen for satisfaction of the organic load. Intrapond recirculation allows the pond to gain some of the advantages that a completely mixed environment would provide if it were possible in a pond. It helps prevent odors and anaerobic conditions in the feed zone of the pond.

Both interpond and intrapond recirculation can affect stratification in ponds, and thus gain some benefits ascribed to pond mixing, which is discussed later. Pond recirculation is not generally as efficient as are mechanical systems in mixing facultative ponds. Both pond mixing and pond recirculation are incorporated in the Sunnyvale, Calif., example (ch. IV, case 1).

Three common types of interpond-recirculation systems (series, parallel, and parallel series) are shown in figure II-2. Others have been suggested but seldom used.

One objective of recirculation in the series arrangement is to decrease the organic loading in the first cell of the series. While the loading per unit surface is not reduced by this configuration, the retention time of the liquid *is* reduced. The method attempts to flush the influent through the pond faster than it would travel without recirculation. The hydraulic retention time of the influent and recycled liquid in the first, most heavily loaded pond in the series system is:

$$T = \frac{V}{(1 + r)F} \qquad \text{(II-1)}$$

where V is the volume of pond cell, F is the influent flow rate, r, or R/F, is the recycle ratio, and R is the recycle flow rate.

Another advantage of recirculation in the series configuration is that the BOD in the mixture entering the pond is reduced, and is given by the expression:

$$S_m = \frac{S_{in}}{1 + r} + \left(\frac{r}{1 + r}\right) S_3 \qquad \text{(II-2)}$$

where S_m is the BOD of the mixture, S_3 is the effluent BOD from the third cell, and S_{in} is the influent BOD. Thus, S_m would be only 20 percent of S_{in} with a 4:1 recycle ratio, as S_3 would be negligible in almost all cases. Thus, the application of organic load in the pond is spread more evenly throughout the ponds, and organic loading and odor generation near the feed points are less. Recirculation in the series mode has been used to reduce odors in those cases where the first pond is anaerobic.[3]

The parallel configuration more effectively reduces pond loadings than does the series configuration, because the mixture of influent is spread evenly across all ponds instead of the first pond in a series. Recirculation has the same benefits in both configurations.

For example, consider three ponds, either in series or parallel. In the parallel configuration, the surface loading (pounds BOD_5 per acre per day) on the three ponds is one-third that of the first pond in the series configuration. The parallel configuration, therefore, is less likely to produce odors than the series configuration.

Recirculation usually is accomplished with high-volume, low-head propeller pumps. Figure II-3 presents a simplified cross section of such an installation. In this design, the cost and maintenance problems associated with large discharge flap gates are eliminated by the siphon discharge. An auxiliary pump with an air eductor maintains the siphon. Siphon breaks are provided to insure positive backflow protection.

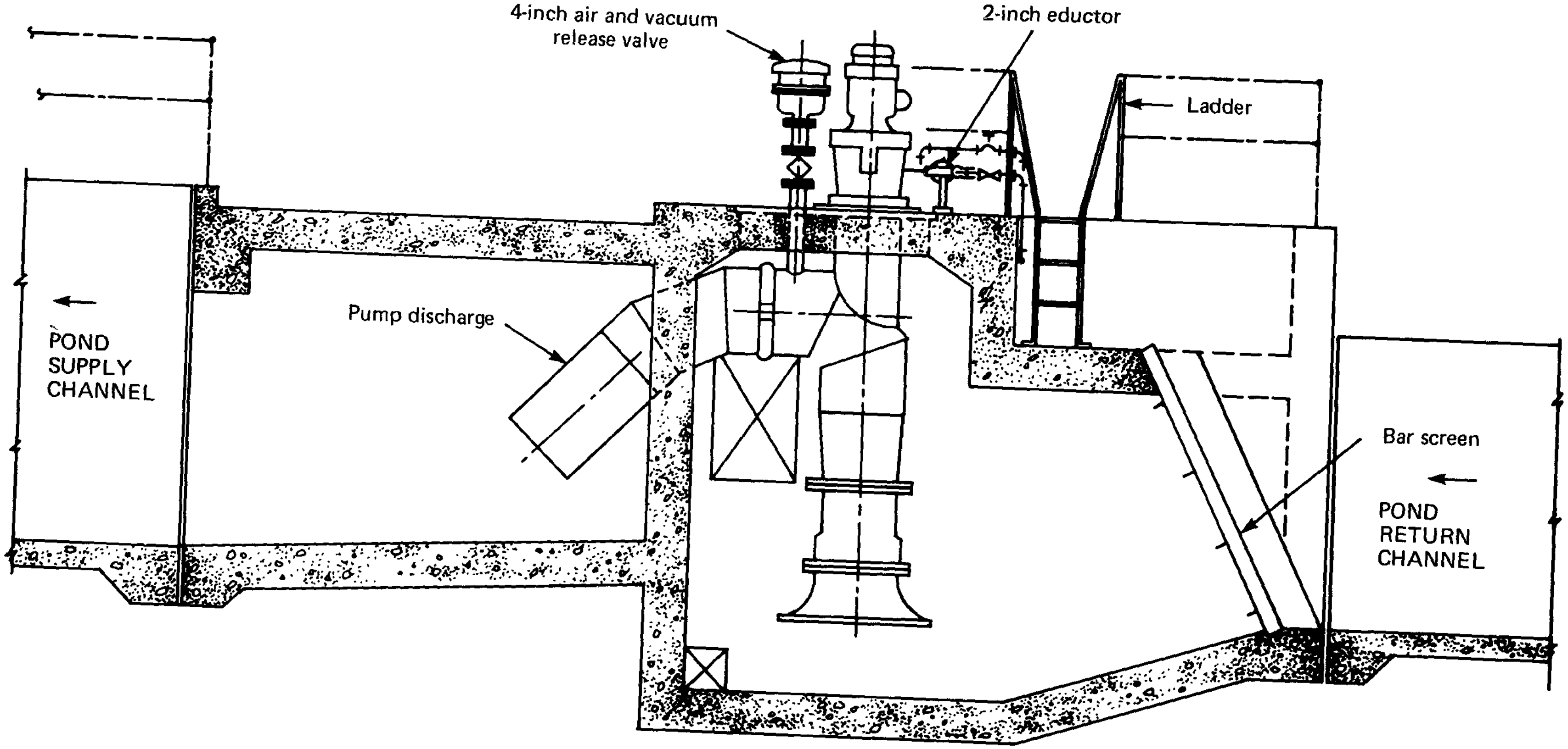


Figure II-3. Cross section of a typical recirculation pumping station.

Pumping stations of this type can be designed to maintain full capacity with minimal increase in horsepower even when the inlet and discharge surface levels fluctuate over a 3-4-foot range. Multiple- and/or variable-speed pumps are used to adjust the recirculation rate to seasonal load changes.

Pond configuration should allow full use of the wetted pond area. Transfer inlets and outlets should be located to eliminate dead spots and short circuiting that may be detrimental to photosynthetic processes. Wind directions should be studied, and transfer outlets located to prevent dead pockets where scum will tend to accumulate. Pond size need not be limited, as long as proper distribution is maintained.

FEED AND WITHDRAWAL

Opinion in the literature is nearly unanimous that ponds should be fed by a single pipe, usually toward the center of the pond. Such design should be used for raw-sewage treatment by ponds. It has been found that with primary or secondary effluent, a single point of entry into a pond tends to overload the pond in the feed zone, allowing odors to develop. Brown and Caldwell often employs a multiple-entry and single-exit approach to distribute evenly the organic load throughout the pond cell (see fig. II-4). One form of multiple inlet, used for ponds as large as 20 acres, uses inlet head loss to induce internal pond circulation and initial mixing. The inlet pipe, laid on the pond bottom, has multiple ports or nozzles all pointing in one direction and at a slight angle above the horizontal. Port head loss is designed for about 1 foot at average flow, resulting in a velocity of 8 feet per second. This velocity induces sufficient mass pond movement to permit the pond outlet to be located near the inlet. A second outlet, with low head loss and controlled by an overflow weir, accommodates peak wet-weather flow.

The multiple-entry, multiple-exit approach has been used in the Stockton, Calif., ponds (case 2, ch. IV). This system was developed to discourage the development of stagnant surface areas within the pond that can cause development of blue-green algae mats. Such mats can emit odors.

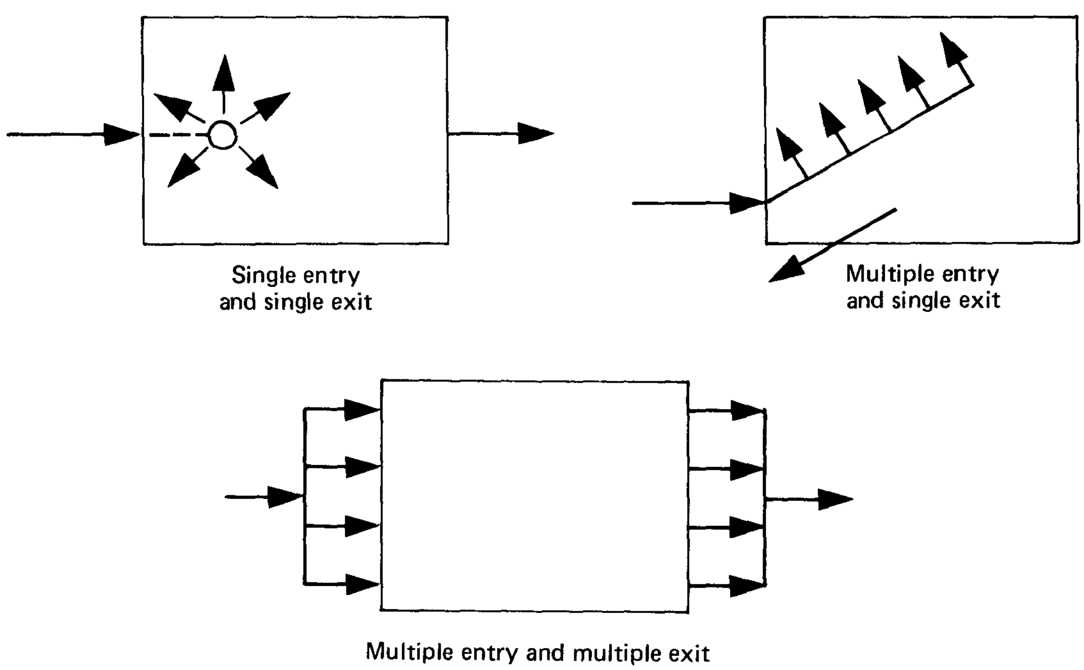


Figure II-4. Methods for feed and withdrawal from ponds.

POND TRANSFER INLETS AND OUTLETS

Pond transfer inlets and outlets should be constructed to minimize head loss at peak recirculation rates, assure uniform distribution to all pond areas at all recirculation rates, and maintain water-surface continuity between the supply channel, the ponds, and the return channel.

Transfer pipes should be numerous and large enough to limit peak head loss to about 3-4 inches with the pipes flowing about two-thirds to three-quarters full. Supply- and return-channel sizing should assure that the total channel loss is no more than one-tenth of the transfer-pipe losses. When such a ratio is maintained, uniform distribution is assured.

By operating with the transfer pipes less than full, unobstructed water surface is maintained between the channels and ponds, which controls scum buildup in any one area.

Transfer inlets and outlets usually are made of bitumastic-coated, corrugated-metal pipe, with seepage collars located near the midpoint. This type of pipe is inexpensive, strong enough to withstand rough handling and rapid backfilling, and flexible enough to allow for the differential settlement often encountered in pond-dike construction.

Specially made fiber-glass plugs can be provided to close the pipes. The plugs may be installed from a boat. Pond recirculation must be shut down to remove the plugs. Such plugs permit any pipe to be closed without expensive construction of sluice gates and access platforms at each transfer point. Concrete launching ramps into each pond and channel assure easy boat access for sampling, aquatic plant control, and pond maintenance.

POND-DIKE CONSTRUCTION

Pond and channel dikes usually can be constructed with side slopes between 6 horizontal to 1 vertical and 2 horizontal to 1 vertical. The final slope selected will depend on the dike material and the water-erosion protection to be provided. All soils, regardless of slope, will require some type of protection in zones subject to wave action, hydraulic turbulence, or aerator agitation. Examples of turbulent zones are areas around the discharge areas at the recirculation pumping station and areas around the influent and effluent connections.

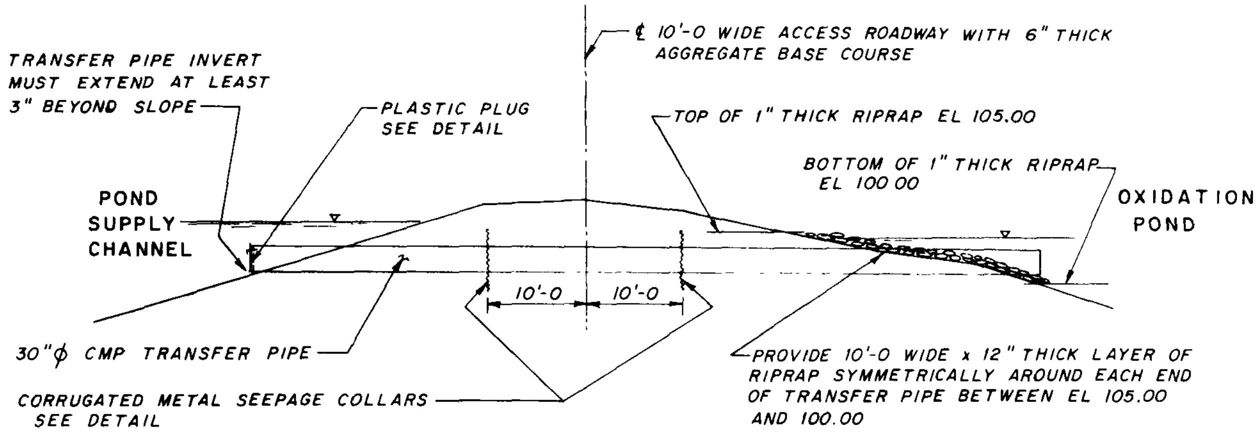
If the wind is always in one direction, wave-action-erosion protection usually can be limited to those areas that receive the full force of the wind-driven waves. Protection should always extend from at least 1 foot below the minimum water surface to at least 1 foot above the maximum water surface.

Protection against hydraulic turbulence should extend several feet beyond the area subject to such turbulence. Protection material should not impede the control of aquatic plant growth.

Pond and channel dikes must be kept completely free from grass and aquatic plants if the ponds are to achieve peak efficiency and operate without odor and insect nuisance. Weeds and aquatic growths usually are controlled by periodic spraying, although cutting and actual physical removal are sometimes necessary. Ponds with luxuriant shoreline growths of cattails and other aquatic plants may seem healthy and beautiful. Closer inspection, however, reveals that such growths harbor heavy accumulation of septic scum, which causes odors and loss of treatment capability.

The tops of the dikes should be at least wide enough for a 10-foot-wide, all-weather gravel road. Such a road is essential for pond inspection and for the control of insects, erosion, and plant growth on the dike surfaces.

Figure II-5 shows some details of dike design.



TYPICAL DIKE CROSS SECTION AT TRANSFER PIPE

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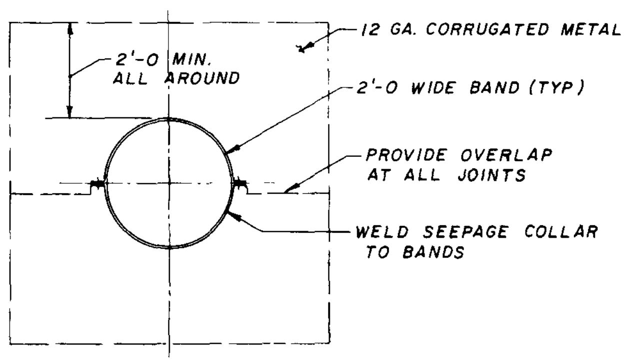
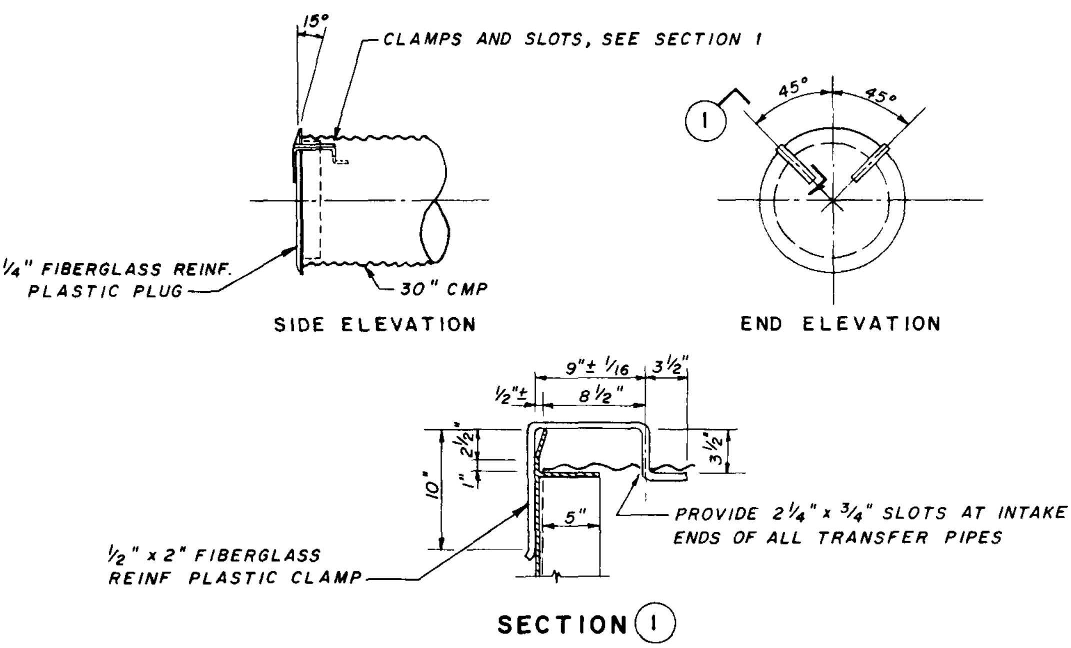


TYP. SEEPAGE COLLAR DETAIL

NO SCALE



TYPICAL PLUG DETAIL

NO SCALE

Figure II-5. Details of dike design.

SUPPLEMENTAL AERATION AND MIXING

While intermittent mixing has been applied to shallow, high-rate aerobic ponds,[4] greater attention has been given to mechanical mixing and to aeration within the cells of facultative ponds. Sometimes, when ponds must treat high, seasonal BOD loading or operate under winter conditions, or when there is no more room for expansion, supplementation of the ponds' photosynthetic oxidation capacity is required. (When no oxygen is supplied by photosynthesis, the system is called an aerated lagoon.[b])

The supplementation usually is achieved by installing compressed air diffusers or mechanical aerators. When the ponds' extra needs are relatively minor and uniform throughout the year, compressed-air aeration may be best. Indeed, if the ponds are located in a cold climate, year-round aeration may be necessary to maintain whatever photosynthetic activity is possible during freezing weather. Preventing surface freezing also allows direct oxygen transfer. When supplemental oxygen requirements are high or when the requirements are either seasonal or intermittent, mechanical aerators are used.

In addition to transferring oxygen to the liquid, aeration breaks up the thermal stratification that normally develops in oxidation ponds. Marais[3] reports that the persistent stratification in ponds diminishes the nonmotile algae population, because the algae settle below the photic zone and die from lack of light. Mixing tends to increase algae numbers and to maintain aerobic conditions deeper in the pond. By increasing algae numbers, the pond can produce more oxygen, thus increasing its capacity for organic loading.

Surface agitation also breaks up the thin surface layer of slick or scum that forms on calm days. If not destroyed, the scum layer can diminish performance both by decreasing the photosynthetic rates and by decreasing surface aeration.

Mechanical aerators generally are divided into two types: cage aerators (fig. II-6)[9] and the more common turbine and vertical-shaft aerators (fig. II-7). Cage aerators are relatively new in the United States (see ch. IV, case 1) and work particularly well in shallow ponds (less than 5 feet deep). Propeller aerators require a minimum depth depending on the horsepower of the unit. For shallow ponds a large number of low-horsepower units are required, and the cost per horsepower rises.

The cage aerator appears to have an area of influence of as much as 1,200 feet (as determined by photographs). While no precise comparison has been made, this device appears to have a much greater pumping capacity than the propeller aerator. The latter device tends to recycle much of the volume pumped, especially in shallow ponds.

Floating propeller aerators are always mounted out in the pond, far enough apart to minimize interference with one another or with other pond features. When used for shallow ponds, they require minimum-depth pits lined with erosion-resistant surfaces. These surfaces are usually some form of paving, often concrete. Power access is usually via underwater cable, while maintenance access is almost always by boat.

Floating cage aerators may be mounted either in the pond or directly off the dike slopes (as at Sunnyvale, case 1, ch. IV). When mounted off the dike slopes, they can be close to the pond transfer inlets. The entire dike slope in the immediate area is provided with erosion protection. Units

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[b]For information on design of aerated lagoons, see references 5-8.

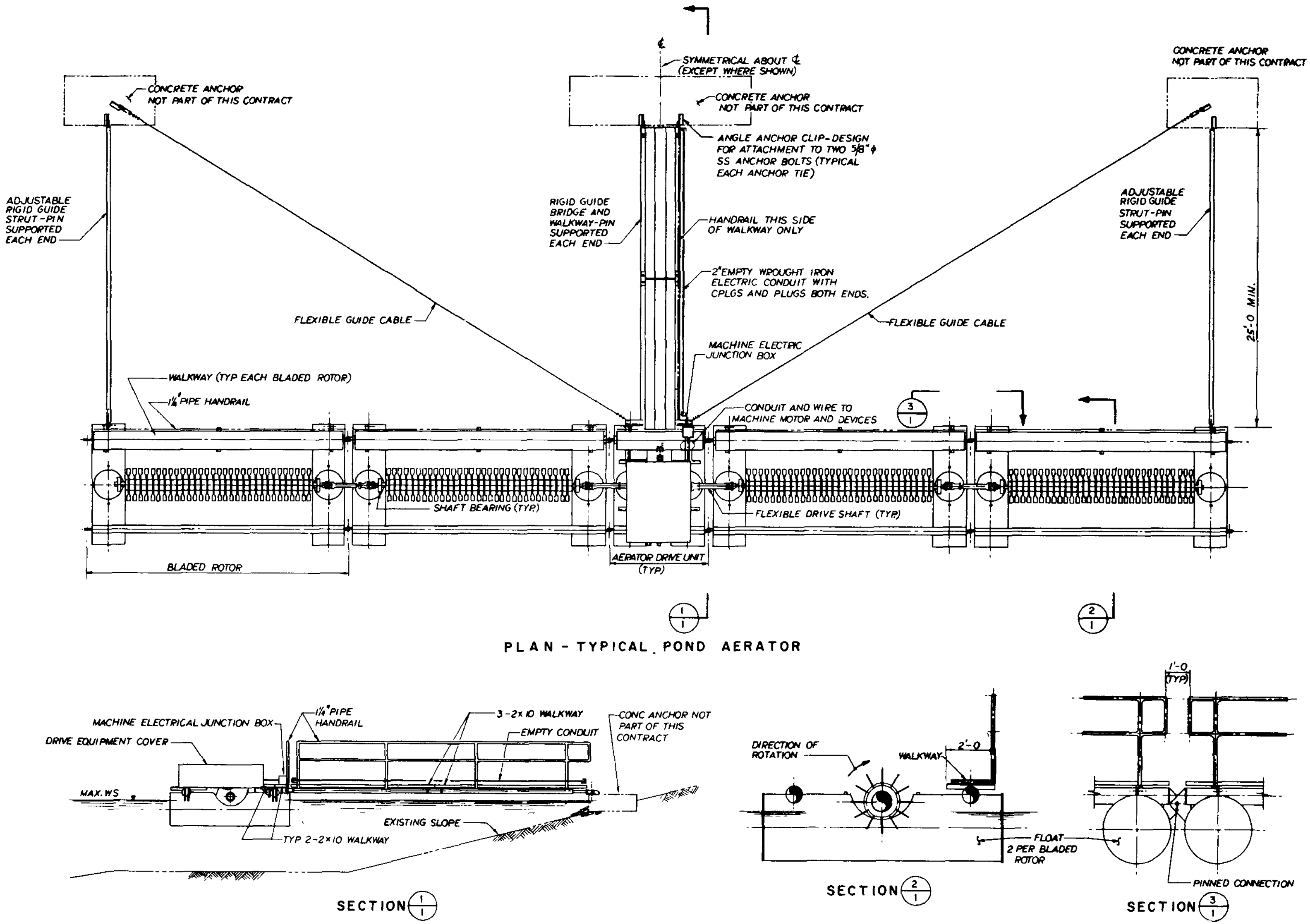

Figure II-6. Floating cage aerator.

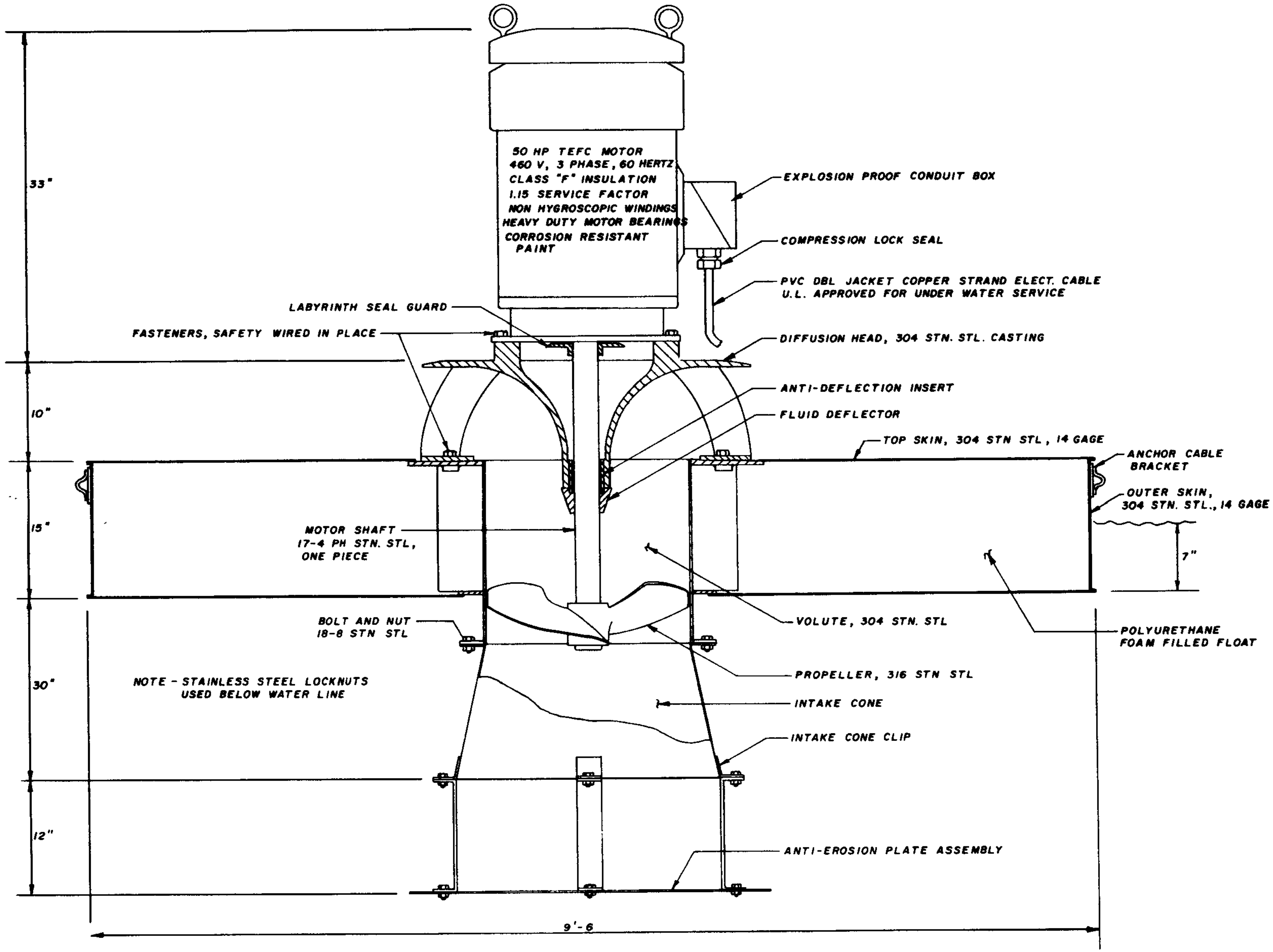

Figure II-7. Floating propeller aerator.

mounted on the slope offer easy access for maintenance and repair and the extra reliability of above-water power supply.

Most previous pond-aeration systems seem to have used diffused aeration. For best efficiency, these systems require that the ponds be deepened to 10 feet.[10]

Pond aeration and mixing systems serve mainly to increase the oxidation capacity of the pond. They are useful in overloaded ponds that generate odors.

REFERENCES

[1] D. S. Parker, J. R. Monser, and R. G. Spicher, "Unit Process Performance Modeling and Economics for Cannery Waste Treatment," presented at the 23d Purdue Industrial Waste Conference, May 7-9, 1968.

[2] M. G. McGarry and M. B. Pescod, "Stabilization Pond Criteria for Tropical Asia," *Second International Symposium for Waste Treatment Lagoons*, pp. 114-132, Missouri Basin Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[3] G. V. R. Marais, "Dynamic Behavior of Oxidation Ponds," *Second International Symposium for Waste Treatment Lagoons*, pp. 15-46, Missouri Basin Health Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[4] W. J. Oswald, ch. 17, *Water Quality Management*, edited by P. H. McGavhey, University of California, Sanitary Engineering Research Laboratory, Berkeley, Calif., 1966.

[5] *Second International Symposium for Waste Treatment Lagoons*, Missouri Basin Engineering Council and Federal Water Quality Administration, Kansas City, Mo., June 23-25, 1970.

[6] G. A. Boulier and T. J. Atchison, *Practical Design and Application of the Aerated-Facultative Lagoon Process*, 2nd ed., Hinde Engineering Co., Highland Park, Ill., 1975.

[7] W. W. Eckenfelder, Jr., *Industrial Water Pollution Control*, McGraw-Hill, New York, 1966, ch. 15.

[8] Biological Waste Treatment, course given at Manhattan College, New York.

[9] Brown and Caldwell, Contract for pond aerators, prepared for city of Sunnyvale, Calif., Dec. 1969.

[10] R. E. McKinney, J. N. Dornbush, and J. W. Vennes, "Waste Treatment Lagoons—State of the Art," Missouri Basin Engineering Health Council, EPA WPCRS, 17090EHX, Washington, D.C., July 1971.

Chapter III

UPGRADING LAGOONS THROUGH ALGAE REMOVAL

The presence of algae in oxidation-pond effluent is undoubtedly the most common problem in upgrading lagoons to meet discharge permit requirements. Algae are manifested principally by high suspended solids and long-term biochemical oxygen demand (BOD) measurements. Figure III-1 shows effluent BOD from the Stockton, Calif., ponds during the 1970 summer canning season (ch. IV, case 2). Physical separation of the algae removed virtually all the long-term BOD. With proper design and operation of the pond treatment system, insertion of an algae-removal step can produce an effluent low in both oxygen-demanding substances and nutrients.

In contrast, figure III-2 shows the effluent BOD for an activated sludge plant also receiving a heavy canning load. That effluent also has a high 30-day BOD, but much less can be removed by solids separation because more of the long-term BOD was in the ammonia form and thus not removable by physical separation.

Techniques to remove algae from pond effluent have included coagulation-clarification processes, filtration, centrifugation, microstraining, chlorination, and land application. In-pond removal systems include aquaculture, series arrangements, intermittent discharge, chlorine addition, or coagulant addition to promote sedimentation within the ponds. Because past mistakes can teach as much as past successes, techniques that have not yet produced adequate results are discussed here with those that have.

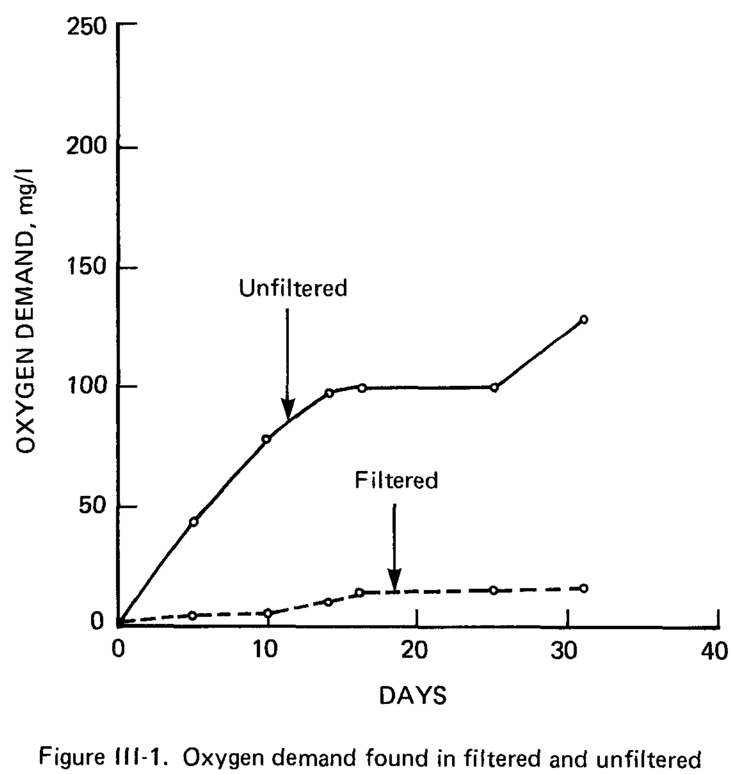

Figure III-1. Oxygen demand found in filtered and unfiltered samples of oxidation-pond effluent, September 1970.

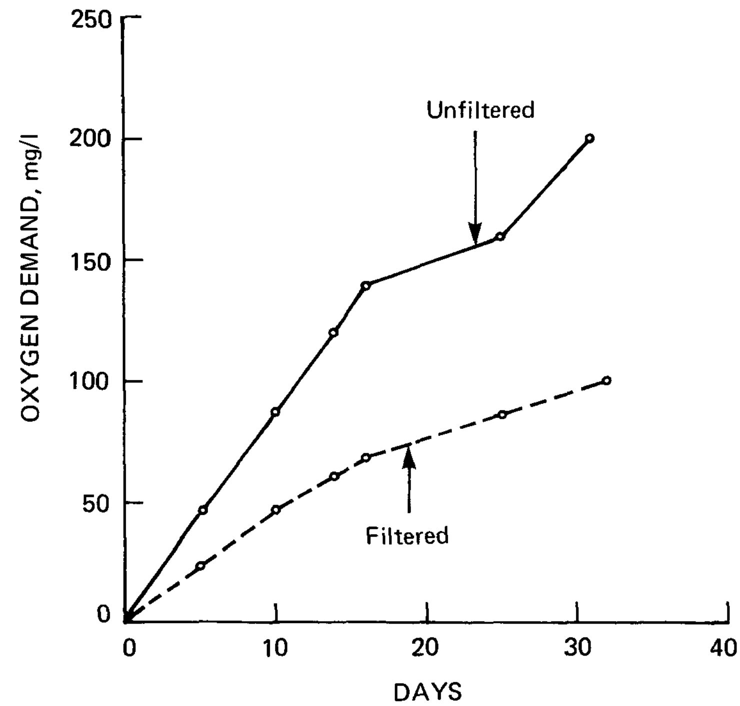

Figure III-2. Oxygen demand found in filtered and unfiltered samples of activated-sludge effluent, September 1970.

TYPES OF ALGAE IN OXIDATION PONDS

Because the types of algae present in an oxidation pond can influence the effectiveness of the separation techniques applied to them, it is useful to review those that can occur. Briefly, algae are divided into four classes: green algae, blue-green algae, diatoms, and pigmented flagellates.

The most frequently observed green algae in oxidation ponds are *Chlorella* and *Scendesumus*, which are small (less than 20 μm) and nonmotile. Because of their small size and low density, they will remain in suspension with minimal fluid motion. Green algae also have a negative charge (or zeta potential) that prevents their natural flocculation in a normal oxidation-pond environment. The negative charge and the small size of these planktonic green algae make sand filtration or microstraining ineffective, and their small size and low density make sedimentation ineffective.

Blue-green algae are nuisance organisms in ponds. The most common species is the filamentous algae, *Oscillatoria*. These algae frequently form mats that emit foul odors. These mats also block light penetration and reduce surface aeration and mixing. *Oscillatoria* cells are typically 200 to 300 μm in length with a diameter of about 5 μm, and they generally have a slimy coating. Blue-green algae have a characteristic gliding movement said to result from the propagation of rhythmic contraction waves within the cell. This movement may allow *Oscillatoria* to pass through restrictions. Blue-green algae tend to clog filters because of their large size, but when aggregated together, they are removable by microstraining.

Diatoms often occur in oxidation ponds, at least in small numbers. They have a silica shell, are nonmotile, and are large enough to clog sand filters.

The most frequently observed pigmented flagellates are *Euglena* and *Chlamydomonas*. These algae are motile and can compensate for variations in lighting conditions by swimming. The flexible cell wall of *Euglena* allows it to pass through constrictions. These algae have typical maximum dimensions of 15 to 30 μm (excluding flagella). They resist removal by sedimentation and flotation because they can swim away in the process effluent.

COAGULATION-CLARIFICATION PROCESSES

Algae cells do not naturally flocculate to a high degree because they are mutually repulsed by negative charges. Consequently, they tend to settle at such a low rate that normal pond turbulence keeps them in suspension. This effect can be overcome by adding chemicals to modify the algae surface charge and provide a matrix for bridging across algae cells to allow aggregation into flocs, which can then be removed easily by sedimentation or flotation.

Organic polymers have been studied extensively for coagulation, but the consensus is that good coagulation cannot be achieved in economic doses.[1-8] Most successful applications have involved the use of inorganic coagulants, occasionally in combination with organic polymers acting as coagulant aids.[2,5-7,9-16] Alum has been used most widely, and occasionally lime has been selected. Ferric or ferrous compounds have seldom been used because of the color they impart to the effluent. Each of these chemicals, alone or in combination with others, may be the most appropriate in particular circumstances. The coagulant chosen will depend on pond effluent quality, the type and concentration of predominant algae, process considerations, and total cost (including sludge disposal). Procedures leading to coagulant selection include jar tests, pilot tests, and engineering feasibility studies.

Coagulation-Flocculation-Sedimentation

Although sedimentation has been used to clarify many waste streams, it cannot by itself be used for algae removal. Chemical coagulants must first be added to destabilize the algae. Then the algae-coagulant particles must be aggregated to form flocs large enough to settle and be removed in a sedimentation tank. Thus sedimentation involves three stages: chemical coagulation, flocculation, and sedimentation.

A number of investigators have obtained high algae removal using the coagulation-flocculation-sedimentation sequence.[2,9,17,18] Representative performance data are shown in table III-1. Over-

Table III-1.—*Summary of coagulation-flocculation-sedimentation performance*

Investigator and location	Coagulant	Dose, mg/l	Overflow rate, gal/min/ft²	Detention time, min	BOD			SS		
					Influent, mg/l	Effluent, mg/l	Percent removed	Influent, mg/l	Effluent, mg/l	Percent removed
van Vuuren et al.,[9] Windhoek, South Africa	Alum[a]	216-300	0.27	200	27.3	9.5	95	85	17	80
	Lime[b]	300-[c]400	.27	200	27.3	3.5	87	85	8	92
Goleuke et al.,[2] Richmond, Calif.	Alum	100	.78	150	23.0	1.0	96	199	13	93
Goodwin,[17] Napa, Calif.	Lime Alum	[d]200 45	([e])	([e])	30.0	3.6	88	102	23	79

[a]As $Al_2(SO_4)_3 \cdot 14.3\ H_2O$ (molecular weight = 600).
[b]As CaO.
[c]pH 10.7.
[d]pH 10.8.
[e]Not available.

flow rates for conventional sedimentation have been in the range of 0.2 to 0.8 gpm/ft^2 with hydraulic detention times of 3 to 4 hours. The flocculation-tank-design criteria that were found to be adequate in one study were detention times of 25 minutes with a G value of 36 to 51 s^{-1}.[19] Underflow solids have generally been quite thin (in the range of 1 to 1.5 percent) when alum or iron is used.

Reductions in sedimentation tank size now appear possible because of the recent application of tube settlers (fig. III-3) to the sedimentation process for algae removal. A pilot study conducted at Regina, Saskatchewan, Canada, indicated that overflow rates with tube settlers could be as high as 5.0 gal/min/ft^2 without deterioration in effluent quality.[20] In other cases, however, the allowable solids-loading rate (thickening capacity of the sludge) might limit sedimentation tank rates to values below 5.0 gal/min/ft^2. The full-scale Regina facility features two flocculator-clarifiers of 60-foot diameter, fitted with tube settlers having a total capacity of 24 mgd. Design overflow rate in the 18-foot-deep upflow portion of the clarifier is 3.7 gal/min/ft^2, and calculated detention times in the flocculator and clarifier are 11 minutes and 36 minutes, respectively. The facility is equipped to employ either alum or lime.[21]

Neptune Micro-Floc recommends somewhat lower overflow rates for tube settlers than those used in the Regina facility. Typically, only two-thirds of the sedimentation tank area is covered with settling tubes, with this area receiving a loading rate of 3.0 to 3.5 gal/min/ft^2, while overall surface loading would be 2.0 to 2.5 gal/min/ft^2. These general recommendations are based on Neptune's full-scale applications similar to algae removal; individual designs might vary from these numbers for a variety of reasons.[22]

The Los Angeles County Sanitation Districts have the longest record of experience with a coagulation-flocculation-sedimentation system, at the Antelope Valley Tertiary Treatment Plant in Lancaster, Calif., constructed in 1970. The Lancaster facility is discussed as case 3 in chapter IV.

In designing a coagulation-flocculation-sedimentation facility, care should be taken to insure that conditions promoting autoflotation (described under flotation in the following section) are not encouraged. Floating sludge in the sedimentation tank defeats the purpose of the process. To prevent this effect, supersaturation should be relieved by preaeration before sedimentation, and photosynthesis in the sedimentation tank should be prevented by covering the tank surface.

Coagulation-Flotation

The flotation process involves the formation of fine gas bubbles that are physically attached to the algae solids, causing the solids to float to the tank surface. Chemical coagulation results in the

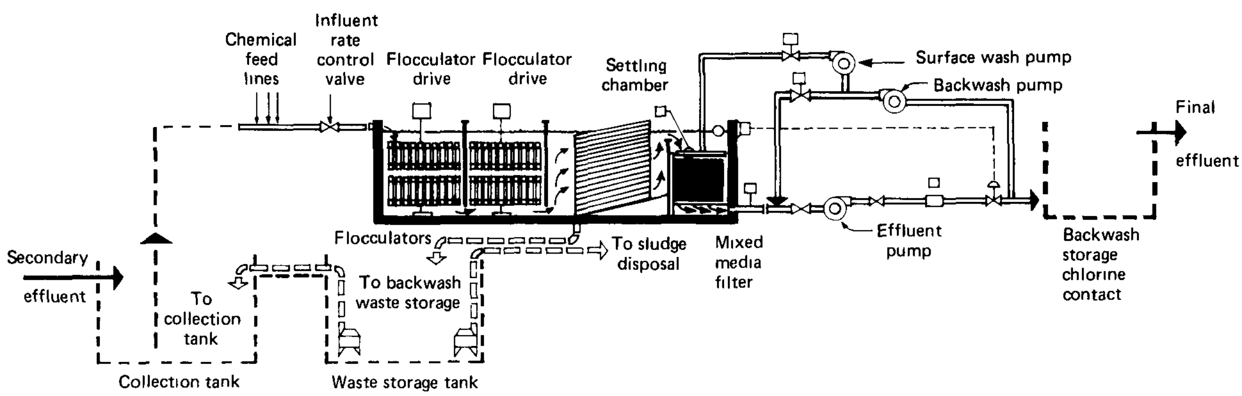

Figure III-3. Tube settlers in a package tertiary plant (courtesy, Neptune Micro-Floc).

formation of a floc-bubble matrix that allows more efficient separation to take place in the aeration tank.

Two means are available for forming the fine bubbles used in the flotation process: autoflotation and dissolved-air flotation. Autoflotation results from the provision of a region of turbulence near the inlet of the flotation tank (which causes bubbles to be formed from the dissolved gases) and from oxygen supersaturation in the ponds. In dissolved-air flotation, a portion of the influent (or recycled effluent) is pumped to a pressure tank where the liquid is agitated in contact with high-pressure air to supersaturate the liquid. The pressurized stream is then mixed with influent, the pressure is released, and fine bubbles are formed. These become attached to the coagulated algae cells. Table III-2 presents a summary of operating and performance data on coagulation-flotation studies. Snider's data[27] are from a full-scale 0.8-mgd, poultry-processing plant; the other data involve pilot studies.

Autoflotation

Information on autoflotation has been developed at Windhoek, South Africa, and Stockton, Calif.[14,15,24,28] For autoflotation to be effective, the dissolved oxygen (DO) content of the pond must exceed about 13-15 mg/l. Furthermore, it is advantageous to use carbon dioxide (CO_2), rather than acid, as the pH adjustment chemical with alum. This approach increases the partial pressure of CO_2 and increases the probability of bubble formation, which will improve performance (table III-2).[14,24]

Autoflotation can perform well under the proper circumstances. Its major disadvantage is that it depends on the development of gas supersaturation within the oxidation pond. At Windhoek, the tertiary ponds could be supersaturated around the clock because of their light organic loading and the presence of favorable climatic conditions. At Stockton, the required degree of supersaturation was present only intermittently, and then for less than half the day. The Stockton pond organic loadings (90 pounds BOD per acre per day during summer) are closer to normal facultative pond loadings than those at Windhoek.

Generally, autoflotation is usable for only a part of the day. The only way to compensate is to increase the number of flotation tanks accordingly and use the process whenever it is operable. The extra cost will favor the selection of dissolved-air flotation in nearly all instances.

Dissolved-Air Flotation

The principal advantage of coagulation/dissolved-air flotation over coagulation-flocculation-sedimentation is the smaller tanks required. Flotation can be undertaken in shallow tanks with hydraulic residence times of 7 to 20 minutes, rather than the 3 to 4 hours required for deep sedimentation tanks. Overflow rates for flotation are higher, about 2.0 gal/min/ft^2 (excluding recycle) compared to 0.8 gal/min/ft^2 or less for conventional sedimentation tanks.

Sedimentation, however, does not require the air-dissolution equipment of flotation, making it a simpler system to operate and maintain. This factor is especially important for small plants, and it was crucial in the selection of sedimentation over flotation for the Antelope Valley Tertiary Treatment Plant in Lancaster (ch. IV, case 3).[10]

Another advantage of flotation over sedimentation is that a separate flocculation step is not required. In fact, a flocculation step after chemical addition has been found to be detrimental when placed ahead of the introduction of the pressurized flow into the influent.[24,29] The normal purpose of a flocculator is to provide, by gentle agitation, the opportunity for large flocs to form. The downstream introduction of the pressurized stream and the resultant turbulent shearing causes floc

Table III-2.—*Summary of typical coagulation-flotation performance*

Investigator and location	Coagulant	Dose	Overflow rate, gal/min/ft²	Detention time, min	BOD			SS		
					Influent, mg/l	Effluent, mg/l	Percent removed	Influent, mg/l	Effluent, mg/l	Percent removed
Autoflotation:										
van Vuuren et al.;[23]	Alum	220 mg/l	3.5	8	12.1	2.8	77	([a])	([a])	([a])
Windhoek, South Africa	CO_2	to pH 6.5	1.8	8	12.1	4.4	64	([a])	([a])	([a])
Parker et al.;[14] Stockton, Calif.	Alum	200 mg/l	2.0	22	([a])	([a])	([a])	70	11	85
	CO_2	to pH 6.3								
	Alum	200 mg/l	2.0	22	([a])	([a])	([a])	156	75	44
	Acid	to pH 6.5								
Dissolved-air flotation:										
Parker et al.;[14] Stockton, Calif.	Alum	225 mg/l	[b]2.7	[b]17	46	5	89	104	20	81
	Acid	to pH 6.4								
Ort;[16] Lubbock, Tex.	Lime	150 mg/l	([a])	[d]12	280-450	0.3	>99	240-360	0-50	>79
Komline-Sanderson;[24] El Dorado, Ark.	Alum	200 mg/l	[c]4.0	[c]8	93	<3	>97	450	36	92
Bare et al.;[25] Logan, Utah	Alum	300 mg/l	[e]1.3-2.4	([a])	([a])	([a])	([a])	100	4	96
Stone et al.;[26] Sunnyvale, Calif.	Alum	175 mg/l	[f]2.0	[f]11	([a])	([a])	([a])	150	30	80
	Acid	to pH 6.0-6.3								
Snider[27]	Alum	125 mg/l	3.3	([a])	65	7.7	88	90	10	89

[a]Not available.
[b]Including 33-percent pressurized (35-60 psig) recycle.
[c]Including 100-percent pressurized recycle.
[d]Including 30-percent pressurized (50 psig) recycle.
[e]Including 25-percent pressurized (45 psig) recycle.
[f]Including 27-percent pressurized (55-70 psig) influent.

breakup to occur, defeating the purpose of the upstream flocculation step. Further, the coagulating power of the chemicals has been lost by this time, and it becomes necessary to add new coagulants to form good float.

Optimization of Dissolved-Air-Flotation Operation

Operating parameters used in dissolved-air flotation include surface-loading rates, air/solids ratio, pressurization level, coagulant dose, and pH adjustment. Physical design parameters for the flotation tank include the coagulant-addition point, the choice of influent versus recycle pressurization, and the design details for the flotation tank. The last item is important because most proprietary tank designs were developed for sludge-thickening applications, and some manufacturers have not reevaluated designs for optimal algae removal.

Surface-Loading Rates. Studies at Stockton and Sunnyvale, Calif.,[14,15,26] and at Logan City, Utah,[25] indicate that maximum surface-loading rates generally vary from 2.0 to 2.7 gal/min/ft^2 (including effluent recycle, where used), depending on tank design. Stone et al.[26] found, in pilot studies at Sunnyvale, that loadings greater than 2.0 gal/min/ft^2 caused deteriorating performance. The flotation tank used in the study was of poor hydraulic design, however, and it was concluded that higher loading rates might be used in prototype facilities. Stone et al. also concluded that influent pressurization produced better results than recycle pressurization and allowed use of smaller tanks as well. Bare[25] found that 2.35 gal/min/ft^2 was optimum, and Parker et al.[14] used 2.7 gal/min/ft^2 at Stockton. Alum was the coagulant used in all cases.

Pressurization and Air/Solids Ratio. The air-solids ratio is defined as the weight of air bubbles added to the process divided by the weight of suspended solids (SS) entering the tank. Values used generally range from 0.05 to 0.10.[14,25] The air/solids ratio is determined by influent solids concentration, pressure level used, and percentage of influent or recycled effluent pressurized. Pressurization levels used in dissolved-air flotation generally range from 25 to 80 psi. Pressure may be applied to all or a portion of the influent or to a portion of the flotation-tank effluent, which is then recycled to the tank influent. The latter mode has traditionally been used for sludge-thickening applications when the influent solids have been flocculated and pressurising the influent might cause floc breakup.

pH Sensitivity of Metal Ion Flocculation. pH is extremely important in alum and iron coagulation. It is possible to adjust the wastewater pH by adding acid (H_2SO_4, for example), and thus take full advantage of the pH sensitivity of the coagulation reactions. The acid dose required to reach a desired wastewater pH level depends on the coagulant dose and wastewater alkalinity.

Figure III-4 shows the effect of pH suppression on effluent SS levels during pilot studies at Sunnyvale,[26] using alum as the coagulant. It was concluded that not much could be gained by suppressing pH below 6.0, and that the range of 6.0 to 6.3 could be used for optimum performance. Subsequent neutralization can be accomplished by adding caustic soda.

Alum Dose. Pilot studies at Stockton[14] and Sunnyvale[26] (fig. III-5) show the effect of influent total suspended solids (TSS) and alum dose on effluent TSS concentrations. Figure III-5 shows that influent TSS has a relatively minor effect on effluent quality. The benefit of increasing alum doses is most pronounced up to 150 to 175 mg/l. Beyond that range, increased alum addition results in only marginal improvement in effluent TSS.

Physical Design. It was noted above that proprietary flotation-tank designs do not possess certain features found to be important in pilot and full-scale studies of algae removal. Features incorporated in the flotation tank designs for Sunnyvale and Stockton (ch. IV, cases 1 and 2) are shown in figure III-6 and illustrate important design concepts.

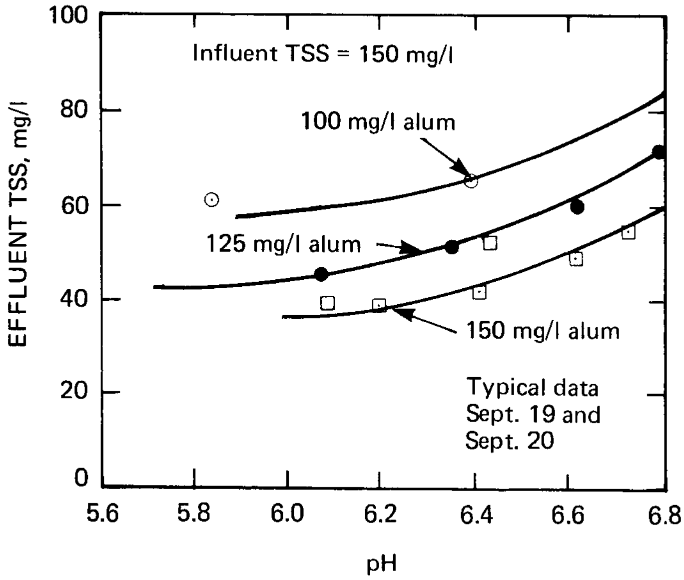

Figure III-4. Effect of alum dose and pH on flotation performance (Sunnyvale Pilot Studies[26]).

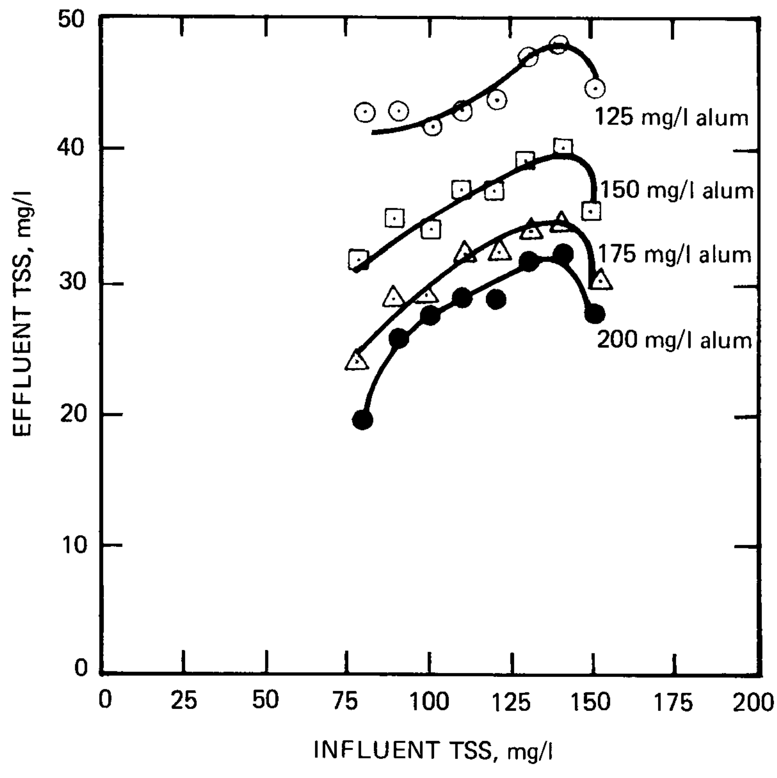

Figure III-5. Effect of alum dose on flotation effluent suspended solids.

- A portion of the flotation-tank influent rather than recycled effluent is pressurized. Better results were obtained in the Sunnyvale studies using partial influent pressurization and the same overall hydraulic loading rate. Thus, smaller tanks can be used. Usually pressurization of 25 percent of the flow will provide good results.

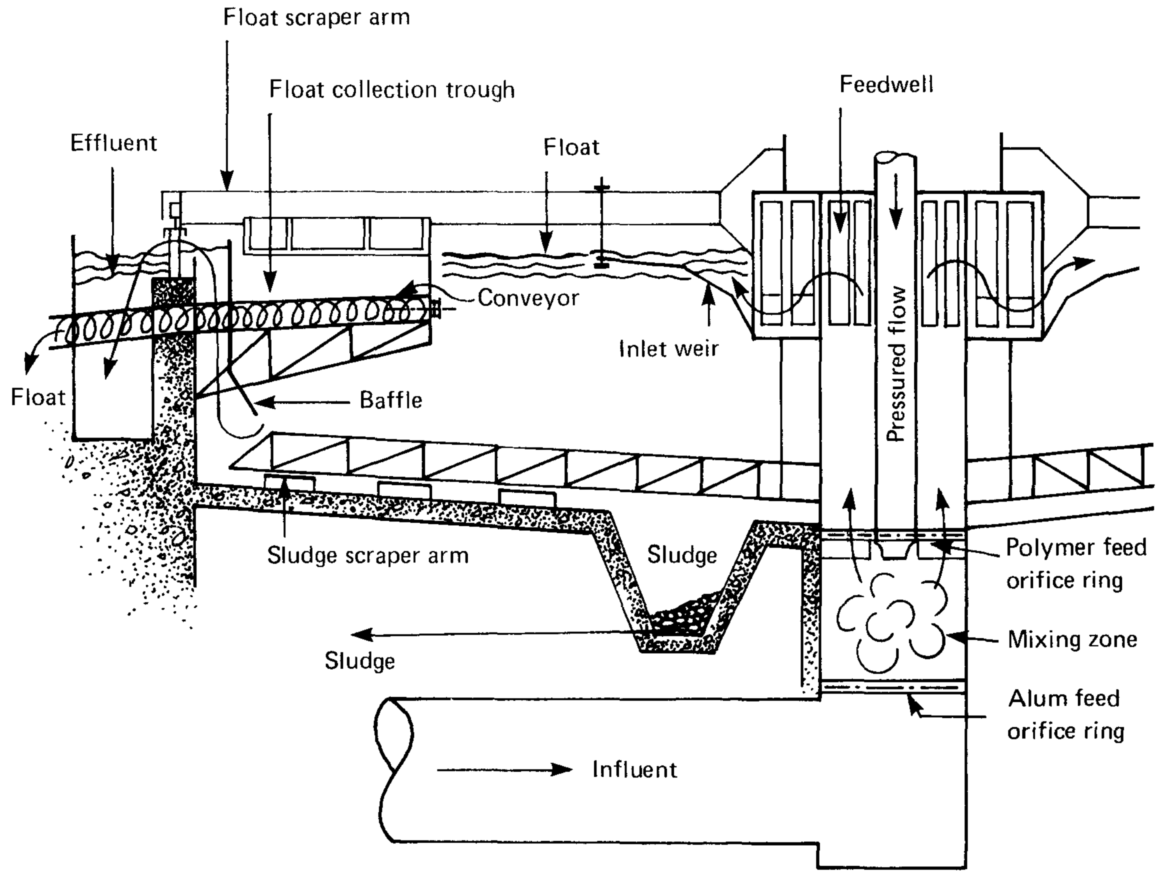

Figure III-6. Conceptual design of dissolved-air flotation tanks.

- The location for alum addition is via orifice rings at the point of pressure release where intense turbulence is available for excellent initial mixing of chemicals. This also permits the simultaneous coprecipitation of algae, bubbles, and chemical floc and results in excellent flotation performance. Altering this order of chemical addition invariably leads to performance deterioration.

- The point of pressure release is in the feedwell. An orifice, rather than a valve, can be used on the pressurized line because the dissolved-air-flotation tanks can operate at constant flow, using the oxidation ponds for flow equalization. In most proprietary designs, a valve is provided on the pressurized line at the outside tank wall, and this permits bubbles to coalesce in the line leading to the feedwell.

- Care is taken to distribute the wastewater flow evenly into the tank. An inlet weir distributes the flow around the full circumference of the inlet zone and a double ring of gates is used to dissipate turbulence. One full-scale circular tank introduced the influent unevenly, causing nearly all the influent to flow through one-quarter of the tank.

- Influent is introduced at the surface rather than below the surface as in most proprietary tank designs. The buoyancy of the rising influent introduced below the surface causes density currents that result in short-circuiting of solids into the effluent.

- Provision of sludge and float scrapers and positive removal of sludge and float will aid performance.

- Effluent baffles extending down into the tank inhibit short-circuiting of solids.

In addition, the tank surface should be protected from wind currents to prevent movement of the relatively light float across the tank. In rainy climates, the flotation tank should be covered because the float is susceptible to breakdown by rain. Alternatively, the flotation tank could be shut down during rainy periods, which would necessitate larger tanks to accommodate higher flow rates in dry weather.

Float Concentration

It is necessary to remove and dispose of the chemical-algae float that rises to the water surface. Flotation generally can result in a higher sludge concentration than does sedimentation for two reasons. First, float removal from the flotation unit takes place on the liquid surface where the operator has good visual control over the thickening process. Second, the float is thickened by draining the liquid from the float, a procedure with a greater driving force promoting thickening than the mechanism in sedimentation, which involves settling and compacting the loose algae-alum floc.

During experimental work at Stockton,[14] it was found that variations in skimmer operation yielded changes in float concentration. For example, improvement in float concentration from 0.13 percent to 2.45 percent resulted from increasing the period between skimming from 2-3 minutes to 15-30 minutes. A further improvement to 3.6 percent occurred when the skimmer was positioned slightly above the water surface level to minimize the inclusion of water in the float. This increase occurred even with a simultaneous decrease in the skimming period to 7-8 minutes.

Bare[25] reported float concentrations of 1.0 to 1.3 percent with alum-coagulation/dissolved-air flotation. Concentrations increased to about 2 percent when a second flotation was allowed to occur in the skimmings receiving tank. Stone et al.[26] reported float concentrations of 1.3 to 2.1 percent in the Sunnyvale studies with specific gravities of 0.45 to 0.55.

Solids Handling and Treatment

Satisfactory disposition must be made of the algae-chemical sludge generated by coagulation-clarification processes. Application of conventional solids-handling and -treatment processes required increased capital and operating expenses, and this consideration was among those that led Middlebrooks et al. to recommend against using coagulation-clarification processes for small plants.[28]

Most of the relevant work to date has involved alum-algae sludges, with very little work done with lime-algae sludge. Disposal and dewatering of alum-algae sludge are notoriously difficult, which is not surprising since algal sludge and alum sludge are difficult to process individually.

Both centrifugation and vacuum filtration of unconditioned algae-alum sludge have produced marginal results because of dewatering difficulties and the need for using low-process loading rates.[2,15] Heat treatment using the Porteous process at temperatures ranging from 193° to 213° C has been shown to improve subsequent vacuum filter yield and cake concentration to a limited extent. Filter yield was low and ranged from 0.9 to 2.5 lb/ft^2/h. Cake concentrations during the study ranged from 8.3 to 21.6 percent total solids, using raw sludge with a solids concentration of about 4 percent.[15]

Use of Zimpro low oxidation, at temperatures ranging from 180° to 220° C, has resulted in vacuum filter cake concentrations ranging from 15 to 19 percent total solids, at a filter-yield rate ranging from 0.67 to 3.05 lb/ft^2/h.[15]

Zimpro high oxidation, at temperatures ranging from 220° to 275° C, was also investigated because it would lead directly to ultimate disposal of the sludge. Evaluation showed that cake concentration and filter yield were marginal, indicating that ultimate disposal should incorporate lagoons. The high-oxidation process removes about 97 percent of the volatile suspended solids (VSS) from the sludge, which is important in producing a stable end product. Although some of the volatile solids are made soluble in the liquid, the final solids are stable and suitable for lagoon storage.[15]

Only limited investigations have been made into the use of centrifugation for concentrating algae-chemical sludges. At Firebaugh, Calif., a Bird solid-bowl centrifuge and a DeLaval yeast-type separator were used to dewater sludge. Both devices were considered failures, although the use of sludge-conditioning aids, such as organic polymers, might be expected to improve their performance.[7] A DeLaval self-cleaning basket machine, also tested, was able to concentrate a 2-3-percent feed to 10 percent total solids with a recovery of 98 percent.

Centrifugation has been used for lime (calcium carbonate) classification of raw sewage sludges,[30,31] but the only report on its use for algae-lime sludge did not present specific details.[16]

Another process that has been investigated is a chemical-oxidation scheme, called Purifax, that employs chlorine as the oxidant. This process was capable of stabilizing the sludge, and yielded a product that could be dewatered on sand drying beds or in a lagoon; however, chlorine costs are relatively high.[15]

Initial work on anaerobic digestion of algae-alum sludge, at the University of California, indicated that the process held little promise for future use.[32] Volatile matter reduction was less than 44 percent, and the digested sludge was unstable and slow to dewater. Subsequent work has shown that algae can be anaerobically degraded successfully if they are killed before their introduction into the digester.[33]

While these relatively complex processes have generally proved unsatisfactory, there is a comparatively simple, and surprisingly effective, solution to the solids-handling problem—return of the algae-alum sludge to the oxidation pond. The return of algae-alum sludge to a pond loaded at 50 lb/acre/day has been studied for a period of 90 days, with a control pond monitored for comparative purposes.[19] No significant differences were observed between the control and test pond in terms of predominant species, or in such effluent characteristics as BOD, alkalinity, dissolved aluminum, nitrogen, phosphorus, or DO. Effluent solids averaged 341 mg/l in the control pond and were only slightly higher, 379 mg/l, in the test pond. The depth of algal sludge accumulated because of return of the sludge was estimated to be only 1 or 2 inches per year. It was concluded that this technique would be a simple solution to the problem of disposing of large volumes of chemically precipitated sludge.[19]

When algae-alum sludge is returned to the oxidation pond, it must be distributed in such a way that sludge does not build up at a single point. Furthermore, when air is contained in the float, procedures must be found to remove it before introducing the sludge into the pond, or floating-sludge problems will result. Several methods have been investigated for breaking down collected float, including the use of high-shear pumps, pumps using a vacuum, high-shear mixers, and water sprays.

Coagulant Recovery

Because chemicals are used in large quantities for coagulation, their generation and reuse may be a way to reduce overall operating costs. Use of acid to reduce pH to about 2.5 can result in a 70 percent alum recovery.[2,34] Because phosphorus is also released at low pH, acid recovery will be limited to those situations where phosphorus removal is not required.

Algae-lime sludge can be centrifuged, with the calcium carbonate being separated into the cake and the algae going into the centrate. Other studies[30,31] have shown that the classified calcium carbonate can be reclaimed for lime reuse. It is expected, however, that the centrate will be difficult to dewater because of its algal content.

Although efforts in coagulant recovery from algae sludges have only been exploratory thus far, there is evidence that further investigations could yield useful results.

FILTRATION

Many efforts have been made to remove algae from pond effluent through some type of filter. Conventional rapid sand or multimedia filters have been used both for *direct* filtration of algal laden waters and for *polishing* filtration, which follows coagulation-clarification. Two other types of filters, which depend in part on biological action for their effectiveness, are submerged rock filters and intermittent sand filters.

Direct Filtration

Experiments with direct sand filtration generally have resulted in poor SS removals, as indicated in table III-3. Without coagulation, algae have a low affinity for sand; furthermore, green algae are too small to be efficiently removed by straining. The larger diatoms can be removed effectively, but special precautions must be taken in media design to insure that the filter does not become rapidly clogged.

In general, work to date indicates that direct filtration of oxidation pond effluent is impractical unless algae concentrations are low.

Polishing Filtration

Use of a rapid sand or multimedia filter system to reduce SS concentrations in coagulation-clarification effluent is very effective, with effluent SS levels less than 10 mg/l and turbidities less

Table III-3.—*Performance summary, direct filtration with rapid sand filters*

Investigator	Coagulant aid and dose, mg/l	Filter loading, gal/min/ft²	Filter depth, feet	Sand size, mm	Finding
Borchardt et al.[35,a]	None	0.2-2.0	2.0	d_{50} = 0.32	Removal declines to 21 to 45 percent after 15 hours.
	Fe: 7	2.1	2.0	d_{50} = 0.40	50-percent algae removal.
Davis et al.[36,a]	None	.49	([b])	d_{50} = 0.75	22-percent algae removal.
	None	.49	([b])	d_{50} = 0.29	34-percent algae removal.
	None	1.9	([b])	d_{50} = 0.75	10-percent algae removal.
	None	1.9	([b])	d_{50} = 0.29	2-percent algae removal.
Foess et al.[37,a]	None	2.0	2.0	d_{50} = 0.71	pH 2.5, 90-percent algae removal. pH 8.9, 14-percent removal.
Lynam et al.[38,c]	None	1.1	0.92	d_{10} = 0.55	62-percent SS removal.
McGhee[39,d]	None	.5-3.0	2.0	d_{10} = 0.22 and 0.5	22- to 66-percent SS removal.

[a]Lab culture of algae.
[b]Not available.
[c]Oxidation pond effluent.
[d]Upflow sand filter.

than 4 Jackson turbidity units (Jtu).[18,26] Diatomaceous earth filters also work efficiently,[10] but filter cycles may be short because of binding of the filter by algae and other particulate matter, which will result in excessive diatomaceous earth use and high operating costs.

Baumann and Cleasby[40] have shown that, while there are many similarities between water filtration (for which the most information is available) and wastewater filtration, there are also differences that must be properly accounted for in design. In particular, the quantity of solids in wastewater streams is generally higher and the characteristics much more variable than for water treatment. Furthermore, filter effluent turbidities and SS concentrations will generally be much lower for water-treatment applications. Therefore, direct application of designs developed for water-treatment plants may result in less than optimum operation and performance.

It is essential for filter runs of reasonable length that the filter remove solids throughout the entire depth of media (deep-bed filtration) and not mainly at the filter surface. Deep-bed filters can be designed by using high filtering velocities (up to 6 gal/min/ft^2), which permit deeper penetration of the solids into the filter, and by allowing the water to pass through a coarse-to-fine media gradation. It is advantageous in wastewater filtration to use a greater depth of filter media (60 to 70 inches) than in water filtration (30 to 50 inches), to allow for greater floc storage in the filter.

Backwashing operations for wastewater filtration will also differ from those techniques used in water filtration. Auxiliary agitation of the media is essential to proper backwashing. Either air scour should be used or surface (and possibly subsurface) washers should be installed to insure that the original cleanliness and grain classification will be restored.

Figures III-7 and III-8 and table III-4 summarize the performance of a dual-media filter used for polishing flotation-tank effluent in the Sunnyvale pilot studies.[26] The filter material was 48 inches of anthracite (2.4-4.8-mm particle size), above 18 inches of sand (0.8-1.0 mm). The loading rate was 5.6 gal/min/ft^2. Figure III-7 shows effluent turbidity as a function of filter-run duration. Solids breakthrough occurred after 10 hours. Figure III-8 shows development of the headloss profile with time. The uniform headloss increase at all depths indicates that the filter has removed solids uniformly throughout the depth of the filter. This factor is important in optimizing filter runs.

Design procedures for effluent filtration are described further in the EPA Technology Transfer Seminar publication, *Wastewater Filtration, Design Considerations.*[40]

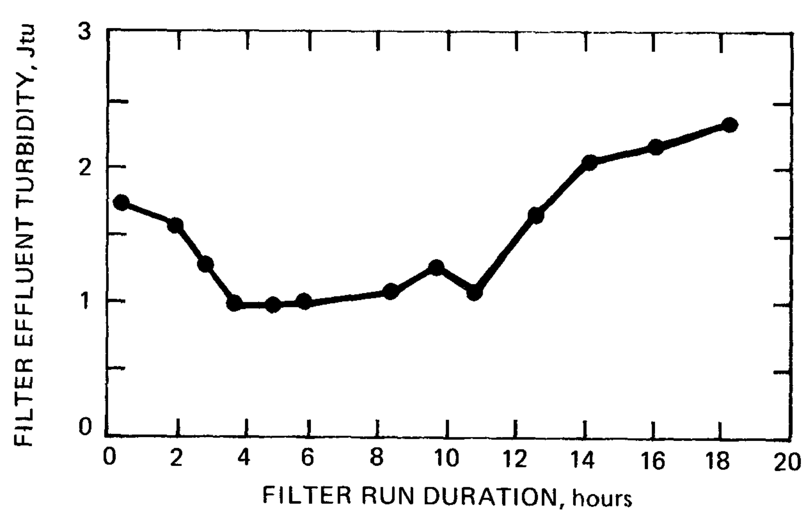

Figure III-7. Dual-Media filter effluent turbidity profile (Sunnyvale Pilot Studies[26])

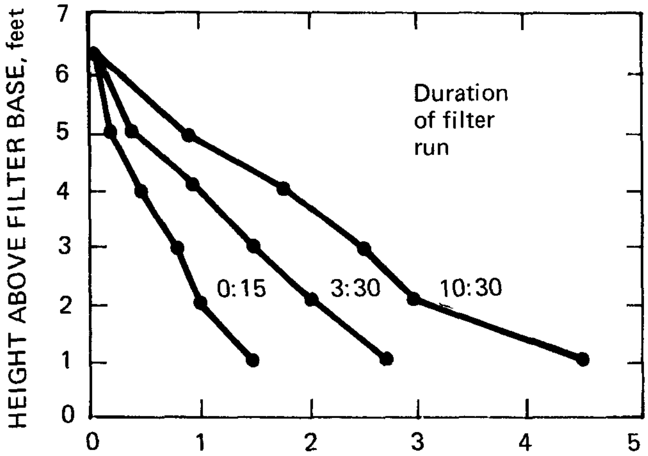

Figure III-8. Filter headloss profile (Sunnyvale Pilot Studies[26]).

Table III-4.—*Filter-performance summary, Sunnyvale pilot studies*[26]

Run date	Filter run duration, hours	Turbidity, Jtu		TSS, mg/l		VSS, mg/l	
		Influent	Effluent	Influent	Effluent	Influent	Effluent
July 11	3:15	6.9	4.2	67	34	42	27
July 11	6:45	6.9	4.2	67	32	42	22
July 25	4:00	6.4	.8	19	10	15	9
July 26	6:15	7.0	2.0	39	22	22	11
July 31	8:10	5.8	1.0	28	6	10	1
Aug. 1	4:15	5.8	1.2	28	2	10	1
Aug. 30	8:00	—	—	32	18	18	16
Sept. 4	6:10	7.0	.7	22	10	18	8
Sept. 7	7:15	12.0	.9	46	12	29	6
Sept. 11	7:15	7.0	2.1	39	11	21	6
Sept. 12	4:45	12.0	3.0	32	[a]4	22	[a]3
Sept. 13	3:15	9.9	3.2	42	6	26	4
Sept. 14	8:30	7.4	3.8	39	3	24	1
Sept. 17	8:00	4.2	2.0	33	5	22	2
Sept. 18	7:15	9.0	3.0	41	3	23	2
Sept. 19	6:15	8.0	2.5	46	6	22	3
Sept. 20	5:10	8.0	2.6	38	2	22	1
Sept. 21	8:30	10.0	3.2	62	6	39	4

[a]After Sept. 11, 1973, the preservative added to filtered samples was changed. Values for TSS and VSS before that date are too high because of postprecipitation in the sample bottles before laboratory analysis.

Submerged Rock Filters

A relatively new experimental approach to algae removal is the submerged rock filter that has been studied extensively at the University of Kansas.[6,41-45] Basically, the system operates by allowing pond effluent to travel through a porous rock wall, causing the algae to settle out on the rock surface. The accumulated algae are then biologically degraded.

Available data indicate that the rock filter is relatively inactive biologically at cold temperatures (less than 10° C); therefore, larger rock filter volumes must be used in cold climates. The effect of influent solids on removal efficiency is uncertain. Most experimental work has been done with influent TSS concentrations below 70 mg/l. Because many facultative lagoons operate with TSS levels of 75 to 150 mg/l, it is necessary to know the performance for such conditions.

An unresolved design consideration for the rock filter is its useful life. Because of the high ratio of capital to operating expenses, the total cost of the system is very sensitive to the length of time it is assumed to operate until plugging failure occurs. A life of 20 to 30 years has been calculated on the assumption that the filter will fail when the voids are full, but experience with other filtration systems indicates that failure will occur much sooner because of flow channelization within the media.

Intermittent Sand Filtration

Investigators at Utah State University have reported on the successful experimental application of the intermittent sand filter for algae removal.[46-48] A drawback, however, is the low influent SS levels encountered. A maximum influent SS concentration of 72 mg/l resulted in effluent concentrations less than 4.0 mg/l for loading rates of 4.6 and 9.2 gal/day/ft^2. The impact of higher SS levels (75 to 150 mg/l) has not yet been determined.

Another factor that has not been accurately evaluated is the most significant component of operating costs—sand cleaning. Observations of similar, slow-sand-filter-cleaning operations in water-treatment applications show that filter cleaning is labor intensive, and this factor may make the intermittent sand filter uneconomical in many instances.

MICROSTRAINING

Although microstraining has often been used in attempts to upgrade pond effluent, the results have been consistently disappointing (table III-5). There are many reports describing successful

Table III-5.—*Summary of microstrainer performance*

Investigator and location	Finding
Golueke et al.,[2] Richmond, Calif.	"At the most, only an extremely small amount of algae was removed by the machine even with the addition of filter aid, decrease in flow rate, and the slowing of the rotational speed of the filter."
Dryden et al.,[10] Lancaster, Calif.	A 23-µm microstrainer was tested. "Removals with the microstrainer were totally inadequate and blinding by the bodies of crustacea and other foreign material occurred quite rapidly."
Lynam et al.,[38] Chicago, Ill.	56-percent BOD removal. 61-percent SS removal. Less than 43 percent of the algae were removed.
California Department of Water Resources,[7] Firebaugh, Calif.	25- and 35-µm screens were tested. "Operation of the unit soon showed that algae were passing through the finer screen. Removals up to 30 percent were obtained, but most of this was due to algae settling in the influent and effluent chambers."

applications of microstraining to algae removal for water supplies. However, they usually deal with removal of the relatively large filter-clogging algae, such as diatoms and clumped blue-green algae, which are larger than the effective opening of the microstrainers. The desirable species of pond algae, such as *Chlorella* and *Scendesumus*, tend to be smaller than the smallest size microstrainer opening available, 23 μm.

Preliminary studies with ultrafiltration, using membranes with smaller openings, have yielded SS removals of 98.8 percent on pond effluent when the predominant algae were *Chlorella*, *Scendesumus*, and *Euglena.*[49] Work with ultrafiltration has not reached the point, however, where there is practical field application. Uncertainties remain in the areas of membrane cleaning and economics.

CENTRIFUGATION

Investigators at the University of California have shown that centrifugation, unaided by coagulants, can successfully remove algae from oxidation pond waters.[2,7,29,50] It is usually more expensive than coagulation-clarification, however. Equipment costs are high and power costs are excessive. For example, an 80-90-percent removal of 200 mg/l of pond effluent solids has an energy requirement of 8,000 kWh per million gallons. At 1.5 cents/kWh, the energy cost alone could be $120 per million gallons.

LAND TREATMENT

In the past, pond effluent has been applied to land more for disposal than for treatment purposes, but land treatment, particularly overland flow, may become more common in the future for polishing pond effluents before stream discharge. Plans are being developed for upgrading the Newman, Calif., treatment facilities through overland flow treatment of pond effluent before discharge to the San Joaquin River.[51] Effluent SS concentrations for the existing facilities average about 80 mg/l. Other improvements to be undertaken at the same time include adding a trickling filter and a chlorine-disinfection system and upgrading the existing septic tanks and oxidation pond. The overland flow system will provide algae and nutrient removal and dechlorination for the design flow of 1.1 mgd.

A pilot study at Davis, Calif., involved the use of overland flow for upgrading oxidation-pond effluent.[52] At a loading rate of 0.68 in/day (two 3-hour applications a day), an influent BOD concentration of 73 mg/l was reduced to 18 mg/l and an SS concentration of 82 mg/l was reduced to 19 mg/l. But difficulties caused by high winds during a portion of the tests resulted in an increase of effluent SS concentrations.

IN-POND REMOVAL SYSTEMS

All the foregoing approaches involve the use of tertiary processes to remove algae from oxidation-pond effluent. Another approach is to modify oxidation-pond construction or operation to produce a pond effluent that will meet the SS levels set in the discharge requirements. Four basic, identifiable methods are aquaculture, series pond operation, intermittent discharge, and chemical addition, perhaps in combination with modified pond operation.

Aquaculture

A relatively new approach to algae removal, aquaculture, involves the use of an ecological food chain to produce a useful product—fish—in contrast to a product requiring further disposal. The many uses of fish range from reduction to animal food to sale of live fish for bait.

The Oklahoma State Department of Health has been studying aquaculture as an algae-removal process since 1970.[53] In preliminary studies, seven species of fish in a six-cell, series-operated pond system were studied to determine the ability of such a system to produce an effluent that meets secondary treatment requirements. Over a 4-month summertime period, the mean BOD, SS, and coliform organism concentrations were 6 mg/l, 12 mg/l, and 20 MPN per 100 ml, respectively. One drawback to the studies was that no data were presented giving effluent concentrations *before* the fish were put into the ponds. Nevertheless, the encouraging results should stimulate further investigations into this unique algae-removal method.

Series Arrangement of Oxidation Ponds

Series ponds are recommended by some State regulatory agencies for encouraging algae sedimentation within the pond cells. A parallel-series arrangement can also encourage sedimentation. The efficiency of sedimentation in ponds, however, is limited by factors such as wind mixing and algae species. Smaller ponds usually result in less mixing. Pigmented flagellates and crustaceans are not removed efficiently in sedimentation ponds.

Favorable reports indicating consistent SS reductions with the series arrangement cannot be found in the literature,[54] nor could an EPA task force find examples of successful application of series-arranged ponds.[55]

Series Ponds With Intermediate Chlorination

Normally, chlorination is used to disinfect effluent, but it has been observed that chlorine added to pond water will also kill algae and cause settling. In 1946, a series of four oxidation ponds near Dublin, Calif., was followed by a chlorine-contact pond. At a 3.75-mgd flow, the chlorine-contact pond had a retention time of 13.5 hours. All the algae were reportedly killed with a chlorine dose of 12 mg/l. Between pond inlet and outlet, the BOD was reduced from 45 to 25 mg/l, SS from 110 to 40 mg/l, and turbidity from 170 to 40 mg/l.[56] Similar reductions were reported in a later study, which found that VSS could be reduced 52 percent and turbidity 32 percent through chlorination.[57] The flocculating effect of chlorine is thought to result from rupture of the algae cell wall and the release of cellular metabolites that may serve as algal flocculants.[58]

In 1972, Chem Pure, Inc., announced a proprietary system using the chlorine-kill concept.[59] In this system, lagoon effluent is dosed with chlorine, then fed into an underground "algae-destruction chamber" where algae are killed and settled out. Once a month, "a standard septic tank truck" pumps the algae and other matter out of the bottom of the destruction chamber. Data suggested a residence time of less than an hour, or about the time normally provided in a conventional chlorine-contact tank.

An adverse side-effect of chlorine use is that algae death and cell lysis cause release of a substantial amount of soluble BOD into the effluent. It has been found that the effluent BOD_5 from an oxidation pond at Concord, Calif., will increase from 20 mg/l to as high as 65 mg/l when 8 mg/l of chlorine are applied.[60] Similar increases in the soluble organic content of chlorinated pond waters have been observed elsewhere.[58]

Intermittent Discharge Lagoons

The operations of 49 intermittent discharge lagoons in Michigan have been well documented.[61] The lagoons in Michigan usually have very low applied BOD loadings, about 20 lb/acre/day. All the systems were designed for discharge twice yearly, with wastewater retention between late November and April 15 and from about May 15 to October 15. Discharge times coincide with periods of low algae-solids levels. Mean effluent concentrations were about 15 mg/l BOD and 30 mg/l SS, low enough to comply with EPA's definition of secondary treatment. This form of treatment is uncertain for warmer climates because suitable periods must be found for pond discharge in the absence of the severe climatic cycles that exist in States such as Michigan.

Intermittent Discharge Lagoons with Chemical Addition

Several investigations of in-pond precipitation of suspended matter conducted in the Province of Ontario, Canada, have proved both the simplicity and effectiveness of the procedure when used under the proper circumstances.[62] Alum was applied from a motor boat to small, seasonal retention lagoons and resulted in low effluent SS concentrations in the subsequent discharge from the lagoons. Treated SS values were generally less than 10 mg/l compared with untreated concentrations of 65 mg/l or less. Only 2 to 3 man-hours per acre were required for the treatment with liquid alum, and sludge buildup was less than 0.1 inch per application (two applications a year). At these sludge-deposition rates, pond dewatering and sludge removal would only be required after many years of operation. It has been suggested that chemical application be investigated for oxidation ponds in southern climates where there are high algae levels year round.[63]

SUMMARY

To put the foregoing information into perspective, a performance ranking has been developed and is presented in table III-6. It is based on an assumed pond SS level of 150 mg/l. This level would

Table III-6.—*Estimated performance of alternative algae-removal systems*

System	Mean effluent SS,[a] mg/l
Microstraining	>60
Direct filtration without coagulants	>60
In-pond removal—series arrangement, continuous discharge	>30
In-pond removal with chlorination[b]	>30
Submerged rock filter[c]	<30
Centrifuge	<30
Intermittent discharge lagoons[d]	<30
Aquaculture	<30
Overland flow	<30
Coagulation-flocculation-sedimentation	10-30
Coagulation-flotation	10-30
Intermittent sand filtration	20
In-pond chemical addition to intermittent discharge lagoons[d]	<10
Coagulation-clarification followed by filtration	<10

[a]Assumes pond effluent suspended solids at 150 mg/l, except as noted.
[b]Accompanied with the release of BOD.
[c]Tentative ranking—full-scale testing to date is based on pond effluent suspended solids averaging less than 73 mg/l.
[d]May be limited to northern U.S. climatic conditions.

be typical of the average monthly SS level of a facultative oxidation pond in the summer. Lower SS levels may prevail at other times of the year, but the summertime level was selected because it represents the level of highest stress on the algae-removal system.

These performance estimates are rough generalizations of the experience gained to date. This type of projection is somewhat hazardous because performance data from the various investigations may not be comparable as a result of differences in test methods, algal properties, concentrations, coagulant type and dose, and quality. Nonetheless, this kind of ranking must be done by the decisionmaker to narrow his range of alternatives in specific situations.

The cost of the alternative systems is the big gap in knowledge. The best established cost/performance data base is for coagulation-clarification systems, because full-scale examples of these processes do exist. Most of the other attractive systems have seen only limited pilot testing, and it is difficult to project their long-range performance or full-scale costs. Based on experience to date, conclusions must be mostly intuitive concerning which types of systems are simpler, require less operation and maintenance, or are more economical.

REFERENCES

[1] J. M. Cohen, G. A. Rourke, and R. L. Woodward, "Natural and Synthetic Polyelectrolytes as Coagulant Aids," *J. Am. Water Works Assoc.*, *50*, 463, Apr. 1958.

[2] C. G. Golueke and W. J. Oswald, "Harvesting and Processing Sewage-Grown Planktonic Algae," *J. Water Pollut. Cont. Fed.*, *37*, 471, Apr. 1965.

[3] North American Aviation, Inc., *A Study of the Use of Biomass Systems in Water Renovation - Final Report*, SID 67-401, PH 86-65-31, May 1967.

[4] M. W. Tenney et al., "Algae Flocculation with Synthetic Organic Polyelectrolytes," *Appl. Microbiol.*, *18*, 965, 1969.

[5] M. G. McGarry, "Algae Flocculation with Aluminum Sulfate and Polyelectrolytes," *J. Water Pollut. Cont. Fed.*, *42*, R191-201, May 1970.

[6] D. M. Martin, Several Methods of Algae Removal in Municipal Oxidation Ponds, masters thesis, Department of Civil Engineering, University of Kansas, Dec. 1970.

[7] California Department of Water Resources, *Removal of Nitrate by an Algal System*, EPA WPCRS, 1303ELY4/71-7, Washington, D.C., Apr. 1971.

[8] R. C. Tilton, J. Murphy, and J. K. Dixon, "The Flocculation of Algae with Synthetic Polymeric Flocculants," *Water Res.*, *6*, 155, 1972.

[9] L. R. J. Van Vuuren and F. A. Van Duuren, "Removal of Algae from Wastewater Maturation Pond Effluent," *J. Water Pollut. Cont. Fed.*, *37*, 1256, Sept. 1972.

[10] F. D. Dryden and G. Stern, "Renovated Wastewater Creates Recreational Lake," *Environ. Sci. Technol.*, *2*, 268-278, Apr. 1968.

[11] M. G. McGarry and C. Tongkasame, "Water Reclamation and Algae Harvesting," *J. Water Pollut. Cont. Fed.*, *43*, 824, May 1971.

[12] A. Shindala and J. W. Stewart, "Chemical Coagulation of Effluents from Municipal Waste Stabilization Ponds," *Water Sewage Works*, Apr. 1971.

[13] S. D. Lin, R. L. Evans, and D. B. Beuscher, *Algae Removal by Alum Coagulation*, Report of Investigation 68, Illinois State Water Survey, Urbana, 1971.

[14] D. S. Parker, J. B. Tyler, and T. J. Dosh, "Algae Removal Improves Pond Effluent," *Water Wastes Eng.*, *10*, 1, Jan. 1973.

[15] Brown and Caldwell, "Report on Pilot Flotation Studies at the Main Water Quality Control Plant," prepared for the city of Stockton, Calif., Feb. 1972.

[16] J. E. Ort, "Lubbock WRAPS It Up," *Water Wastes Eng.*, *9*, 9, Sept. 1972.

[17] Personal communication, E. Goodwin to D. S. Parker, Napa County Sanitation District, July 1972.

[18] Personal communication, F. D. Dryden to D. S. Parker, County Sanitation Districts of Los Angeles County, May 1975.

[19]C. E. Parker, "Algae Sludge Disposal in Wastewater Reclamation," doctoral dissertation, University of Arizona, 1966.

[20]M. J. Stewart, "Chemical Treatment of Stabilization Pond Effluent Followed by High Rate Clarification," presented at 40th Annual Meeting of the Pacific Northwest Pollution Control Assoc., Vancouver, British Columbia, Oct. 1973.

[21]Personal communication, M. J. Stewart to D. S. Parker, Associated Engineering Services, Ltd., Dec. 1973.

[22]Personal communication, S. P. Hansen to D. S. Parker, Neptune Micro-Floc, Inc., Dec. 1973.

[23]L. R. J. Van Vuuren, P. G. J. Meiring, M. R. Henzen, and F. F. Kolbe, "The Flotation of Algae in Water Reclamation," *Int. J. Air Water Pollut.*, *9*, 823, 1965.

[24]Komline-Sanderson Engineering Corp., "Algae Removal Application of Dissolved Air Flotation," unpublished paper, Aug. 1972.

[25]W. F. R. Bare, N. B. Jones, and E. J. Middlebrooks, "Algae Removal Using Dissolved Air Flotation," *J. Water Pollut. Cont. Fed.*, *47*, 1, Jan. 1975.

[26]R. W. Stone, D. S. Parker, and J. A. Cotteral, "Upgrading Lagoon Effluent to Meet Best Practicable Treatment," *J. Water Pollut. Cont. Fed.*, *46*, 8, Aug. 1975.

[27]I. F. Snider, Jr., "Algae Removal by Air Flotation," presented at Symposium No. 9, Ponds as a Wastewater Treatment Alternative, University of Texas at Austin, July 22-24, 1975.

[28]E. J. Middlebrooks et al., "Techniques for Algae Removal From Wastewater Stabilization Ponds," *J. Water Pollut. Cont. Fed.*, *46*, 12, Dec. 1974.

[29]A. R. Ramani, "Factors Influencing Separation of Algal Cells from Pond Effluents by Chemical Flocculation and Dissolved Air Flotation," doctoral dissertation, University of California at Berkeley, 1974.

[30]D. S. Parker, D. G. Niles, and F. J. Zadick, "Processing of Combined Physical-Chemical-Biological Sludge," *J. Water Pollut. Cont. Fed.*, *46*, 2281, Oct. 1974.

[31]D. S. Parker, G. A. Carthew, and G. A. Horstkotte, "Lime Recovery and Reuse in Primary Treatment," *Proc. Am. Soc. Civ. Eng.*, *J. Environ, Eng. Div.*, *101*, 985, Dec. 1975.

[32]C. G. Golueke, W. J. Oswald, and H. B. Gotaas, "Anaerobic Digestion of Algae," *Appl. Microbiol.*, *5*, 1, Jan. 1957.

[33]Personal communication, W. J. Oswald to D. S. Parker, July 1975.

[34]G. L. Culp and A. F. Slechta, "Water Reclamation Studies at the South Tahoe Public Utility District," *J. Water Pollut. Cont. Fed.*, *39*, 737, May 1967.

[35]J. A. Borchardt and C. R. O'Melia, "Sand Filtration of Algae Suspensions," *J. Am. Water Works Assoc.*, *53*, 12, Dec. 1961.

[36]E. Davis and J. A. Borchardt, "Sand Filtration of Particulate Matter," *Proc. Am. Soc. Civ. Eng.*, *J. Sanit. Eng. Div.*, *92*, SA5, Oct. 1966.

[37]G. W. Foess and J. A. Borchardt, "Electrokinetic Phenomenon in the Filtration of Algae Suspensions," *J. Am. Water Works Assoc.*, *61*, 7, July 1969.

[38]G. Lynam, G. Ettelt, and T. McAllon, "Tertiary Treatment of Metro Chicago by Means of Rapid Sand Filtration and Microstrainers," *J. Water Pollut. Cont. Fed.*, *41*, 2, Feb. 1969.

[39]T. J. McGhee, "Upflow Filtration of Oxidation Pond Effluent," *University of Nebraska, Water Resources Research Institute*, Technical Completion Report A-034-NEB, June 1975.

[40]E. R. Baumann and J. L. Cleasby, *Wastewater Filtration, Design Considerations*, U.S. Environmental Protection Agency, Technology Transfer Seminar Publication, EPA-625/4-74007, Cincinnati, Ohio, July 1974.

[41]W. J. O'Brien, "Polishing Lagoon Effluents with Submerged Rock Filter," presented at the Symposium on Upgrading Stabilization Ponds to Meet New Discharge Standards, Logan, Utah, Nov. 1974.

[42]W. J. O'Brien, R. E. McKinney, M. D. Turvey, and D. M. Martin, "Two Methods of Algae Removal from Oxidation Pond Effluents," *Water Sewage Works*, *120*, 66, 1973.

[43]J. L. Martin and R. Weller, "Removal of Algae from Oxidation Pond Effluent," masters thesis, University of Kansas at Lawrence, 1973.

[44]R. A. Hirsekorn, "A Field Study of Rock Filtration for Algae Removal," masters thesis, University of Kansas at Lawrence, 1974.

[45]W. J. O'Brien, "Algae Removal by Rock Filtration," presented at Symposium No. 9, Ponds as a Wastewater Treatment Alternative, University of Texas at Austin, July 22-24, 1975.
[46]J. H. Reynolds, S. E. Harris, D. S. Filip, and E. J. Middlebrooks, "Intermittent Sand Filtration to Upgrade Lagoon Effluents, Preliminary Report," presented at the Symposium on Upgrading Waste Stabilization Ponds to Meet New Discharge Standards, Logan, Utah, Aug. 1974.
[47]E. J. Middlebrooks and G. R. Marshall, "Stabilization Pond Upgrading with Intermittent Sand Filters," presented at the Symposium on Upgrading Waste Stabilization Ponds to Meet New Discharge Standards, Logan, Utah, Aug. 1974.
[48]J. H. Reynolds, S. E. Harris, D. W. Hill, D. S. Filip, and E. J. Middlebrooks, "Intermittent Sand Filtration for Upgrading Waste Stabilization Ponds," presented at Symposium No. 9, Ponds as a Wastewater Treatment Alternative, University of Texas at Austin, July 22-24, 1975.
[49]G. Shelef, R. Matz, and M. Schwartz, "Ultrafiltration and Microfiltration Processes for Treatment and Reclamation of Pond Effluents in Israel," presented at the IAWPR Specialty Conference, Applications of New Concepts of Physical-Chemical Wastewater Treatment, Nashville, Tenn., Sept. 1972.
[50]W. J. Oswald and C. G. Golueke, "Harvesting and Processing of Waste-Grown Microalgae," *Algae, Man and the Environment,* Edited by D. F. Jackson, Syracuse University Press, New York, 1968.
[51]Brown and Caldwell, *Facilities Plan for Wastewater Treatment,* city of Newman, Feb. 1976.
[52]Personal communication, E. D. Schroeder to D. S. Parker, Dec. 1975.
[53]M. S. Coleman, J. P. Henderson, H. G. Chichester, and R. L. Carpenter, "Aquaculture as a Means to Achieve Effluent Standards," *Wastewater Use in the Production of Food and Fiber—Proceedings,* proceedings of conference at Oklahoma City, Okla., Mar. 5-7, 1974, U.S. EPA, Environmental Protection Technology Series, EPA 660/2-74-04-041, Washington, D.C., June 1974.
[54]S. R. Goswami and W. L. Busch, "3-Stage Ponds Earn Plaudits," *Water Wastes Eng., 9,* 4, Apr. 1972.
[55]Personal communication, D. Ehreth to D. S. Parker, U.S. Environmental Protection Agency, 1974.
[56]D. H. Caldwell, "Sewage Oxidation Ponds," *Sewage Works J., 18,* 3, May 1946.
[57]R. Dingus and A. Rust, "Experimental Chlorination of Stabilization Pond Effluent," *Public Works, 100,* 3, Mar. 1969.
[58]W. F. Echelberger, J. L. Pavoni, and P. C. Singer, "Disinfection of Algal Laden Water," *Proc. Am. Soc. Civ. Eng., J. Sanit. Eng. Div., 97,* Oct. 1971.
[59]"Algae Destruction Chamber Upgrades Sewage Lagoon Effluents," *Civ. Eng., 42,* 11, Oct. 1972.
[60]L. W. Hom, "Kinetics of Chlorine Disinfection in an Ecosystem," *Proc. Am. Soc. Civ. Eng., J. Sanit. Eng. Div., 98,* Feb. 1972.
[61]D. M. Pierce, "Performance of Raw Waste Stabilization Lagoons in Michigan with Long Period Storage Before Discharge," presented at the Symposium on Upgrading Waste Stabilization Ponds to Meet New Discharge Standards, Logan, Utah, Aug. 1974.
[62]H. J. Graham and R. B. Hunsinger, "Phosphorus Removal in Seasonal Retention Lagoons by Batch Chemical Precipitation," unpublished paper, Ontario Ministry of the Environment, Research Branch, 135 St. Clair West, Toronto, Ontario, Canada, undated.
[63]R. F. Lewis and J. M. Smith, *Upgrading Existing Lagoons,* U.S. Environmental Protection Agency, OR&D, MERL, Cincinnati, Ohio, Oct. 1973.

Chapter IV

EXAMPLES OF UPGRADING PONDS

CASE 1. SUNNYVALE WATER-POLLUTION-CONTROL PLANT

Process Modifications

Sewage-treatment facilities for the city of Sunnyvale, Calif., were first placed in operation in September 1956. They included a primary treatment plant with an average daily capacity of 7.5 million gallons of domestic sewage and nonseasonal industrial wastes, and a holding pond with a capacity of 200 million gallons for seasonal wastes from two large canneries that processed fruit and vegetables. Effluents from the primary plant and the holding pond were discharged directly to Guadalupe Slough, a tributary to south San Francisco Bay.

By 1960, the domestic sewage flow had reached the capacity of the primary plant, and conditions in Guadalupe Slough, because more effluents were discharged from the treatment facilities, had deteriorated so much that at times they failed to comply with the minimum requirements established by the Regional Water Quality Control Board. In a study authorized by the city, Brown and Caldwell recommended doubling the capacity of the primary plant and adding an oxidation pond. The facilities were not completed until 1967.

Growth of both domestic and industrial wastes since 1960, and the more stringent requirements of the Regional Water Quality Control Board, required further improvement of the plant. This improvement was completed by the canning season of 1971; three more primary settling basins were added (for a total of nine) and aerators were added to the two ponds. The addition of aerators is the primary concern of this discussion. (See figs. IV-1 and IV-2.)

Originally, the large pond (325 acres) had been used as an oxidation pond for secondary treatment of the domestic wastewaters. The wastewater from the canneries was put directly in the smaller holding pond (100 acres). This pond was designed to operate anaerobically, with odors controlled by calcium or sodium nitrate additives. A considerable quantity of nitrate was required, resulting in high operating costs during the food-processing season. Attempted close control of nitrate addition resulted in insufficient amounts being added at times, so that hydrogen sulfide odors did occur.

Design provided for the effluent from the holding pond to be discharged to the oxidation pond at a rate that would maintain aerobic conditions in the oxidation pond. Seasonal wastes increased in quantity and strength beyond expectations, and the holding pond did not have sufficient capacity to contain the waste for the entire canning season. From 1960 on it was necessary to discharge some of the holding-pond contents to Guadalupe Slough during the canning season.

During the past few years, attempts were made to improve the situation by putting the cannery waste through the primary plant and operating the two lagoons in parallel. The small pond received heavier loadings, however, and continued to produce odors. Also, hydrogen sulfide odors continued to develop in Guadalupe Slough.

In an upgrading step, floating cage aerators were placed near the inlets to the ponds to increase their oxidation capacity. The aerators are used only during the canning season, when supplemental

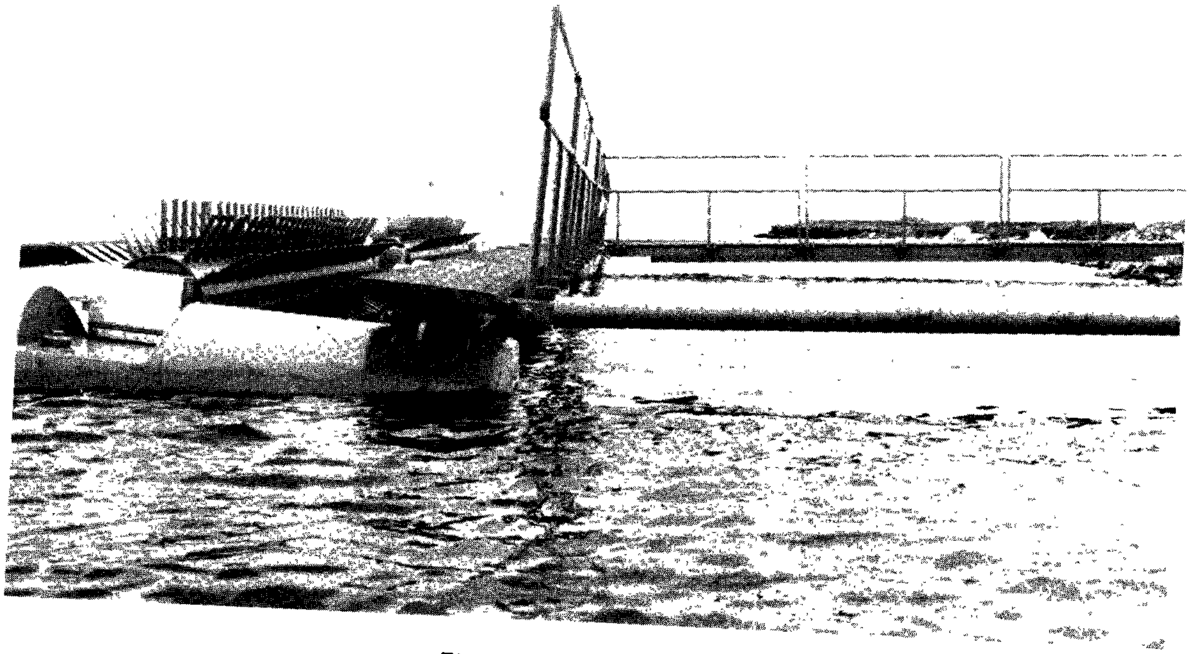

Figure IV-1. Cage aerator.

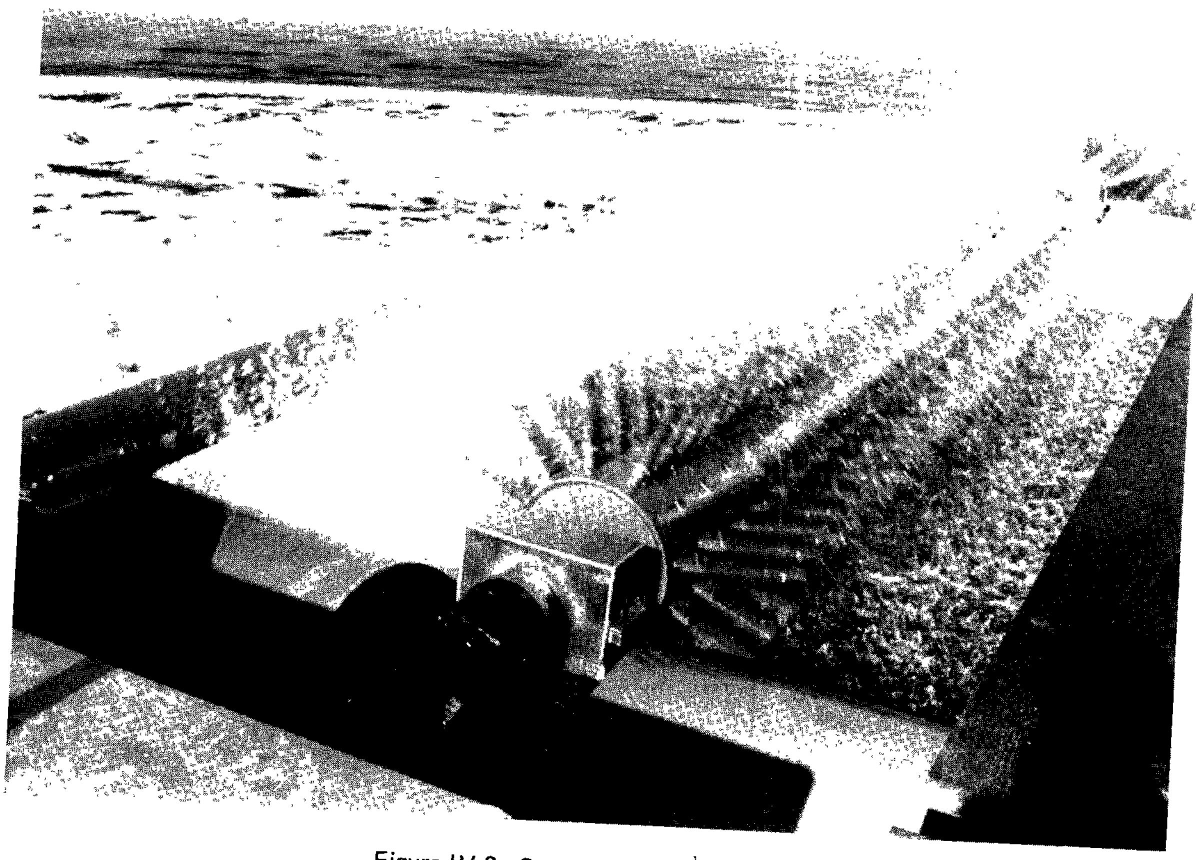

Figure IV-2. Cage aerator in operation.

oxygenation capacity is required. Figure II-6 shows a drawing of the aerator and figure IV-3 shows a diagram of the ponds. (An aerial view of the ponds, 1969 enlargement, is given in fig. IV-4.) The influent and the recirculation flows are mixed in the channel. The flow is then discharged to the ponds through a series of pipes along their edges. The aerators are generally near the transfer pipes; however, no pipes are located near the last two aerators of the large pond. Near the last two aerators the discharge line leads from the pond to the chlorine contact chamber, and then to Guadalupe Slough. These two aerators prevent short circuiting of wastewater.

Operating the ponds in this manner substantially has improved effluent quality. The ponds and Guadalupe Slough contain dissolved oxygen (DO) at all times and are odor free. Fish have returned to the slough. Tables IV-1 and IV-2 give design data for before and after upgrading (1967 and 1971). Table IV-3 shows operating data for 1970 and 1971. Capital costs for pond upgrading are given in table IV-4. Table IV-5 shows the operating cost changes caused by plant expansion.

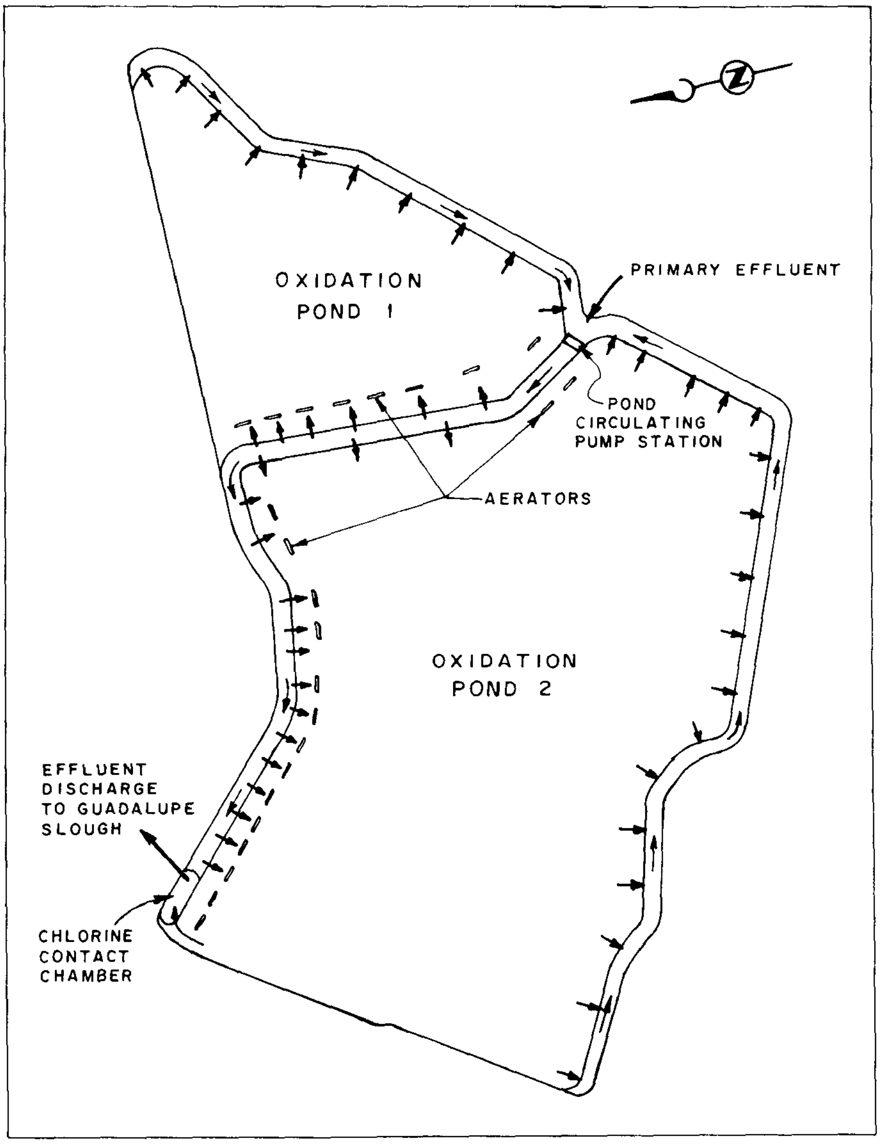

Figure IV-3. Diagram of Sunnyvale ponds.

Figure IV-4. Sunnyvale sewage-treatment works, 1969 enlargement.

Table IV-1.—*Sunnyvale water-pollution-control plant design data, 1967*

Component	Quantity
Design loadings:	
Domestic:	
Daily average flow, mgd	15
BOD, mg/l	270
BOD, lb/day	33,600
SS, mg/l	300
SS, lb/day	37,400
Industrial waste (seasonal):	
Daily average flow, mgd	8.0
BOD, mg/l	1,800
BOD, lb/day	120,000
SS, mg/l	500
SS, lb/day	33,000
Preaeration tanks, domestic sewage only:	
Number	6
Width, feet	19
Length, feet	35
Average water depth, feet	10.5
Detention time, hours	0.5
Air supplied per tank, ft^3/min	300
Air supplied per tank, ft^3/gal/min	0.17
Maximum hydraulic capacity per tank, mgd	6.75
Maximum hydraulic capacity bypass channel, mgd	50
Sedimentation tanks, domestic sewage only:	
Number	6
Width, feet	19
Length, feet	110
Average water depth, feet	10
Effluent weir per tank, feet	164
Detention time, hours	1.5
Mean velocity, ft/min	1.2
Overflow rate, gal/day/ft^2 at daily average flow	1,200
Maximum hydraulic capacity, mgd	6.75
Maximum hydraulic bypass channel, mgd	50

Table IV-1.—*Sunnyvale water-pollution-control plant design data, 1967—Concluded*

Component	Quantity
Primary treatment, domestic sewage only:	
Assumed BOD reduction, percent	35
BOD reduction, mg/l	95
BOD reduction, lb/day	11,800
Assumed SS reduction, percent	60
SS reduction, mg/l	180
SS reduction, lb/day	22,400
Primary effluent, domestic sewage only:	
BOD, mg/l	175
BOD, lb/day	21,800
SS, mg/l	120
SS, lb/day	15,000
Oxidation pond, domestic sewage only:	
Number	1
Area, acres	325
Loading, 5-day BOD, lb/acre/day	67
Detention, days	36
Circulation pumps:	
Number	4
Capacity each, gal/min	44,000
Head, feet	3.5
Engine-generators:	
Number	3
Rated output, kW (high-low)	223-167
Speed, r/min (high-low)	1,000-750
Frequency, c/s (high-low)	66-50
Industrial wastes holding pond:	
Net water area, acres	100
Maximum water depth, feet	6
Maximum capacity, millions of gallons	200

Table IV-2.—*Sunnyvale water-pollution-control-plant design data, 1971*

Component	Quantity
Design loadings:	
Domestic:	
Average daily flow, mgd	22.5
BOD, mg/l	270
BOD, lb/day	50,000
SS, mg/l	300
SS, lb/day	56,000
Industrial waste, seasonal:	
Average daily flow, mgd	8.0
BOD, mg/l	1,800
BOD, lb/day	120,000
SS, mg/l	500
SS, lb/day	33,000
Preaeration tanks:	
Number	7
Width, feet:	
Six at	19.0
One at	20.7
Length, feet:	
Six at	20.5
One at	58.7
Average water depth, feet:	
Six at	10.5
One at	11.0
Average daily flow, mgd:	
Six at	2.7
One at	7.5
Detention time, hours:	
Six at	0.29
One at	0.32
Air supplied per tank, ft^3/min:	
Six at	130
One at	250
Air supplied per tank, ft^3/gal:	
Six at	.074
One at	.048
Maximum hydraulic capacity per tank, mgd:	
Six at	6.75
One at	20
Maximum hydraulic capacity bypass channel, mgd	50

Table IV-2.—*Sunnyvale water-pollution-control-plant design data, 1971—Continued*

Component	Quantity
Sedimentation tanks:	
Number	9
Width, feet	19
Length, feet	110
Average water depth, feet	10
Effluent weir per tank, feet	164
Detention time, hours	1.5
Mean velocity, ft/min	1.2
Overflow rate, gal/ft^2/day	1,200
Maximum hydraulic capacity per tank, mgd	6.75
Maximum hydraulic capacity bypass channel, mgd	50
Primary treatment efficiency, domestic only:	
Assumed BOD reduction, percent	35
BOD reduction, mg/l	95
BOD reduction, lb/day	17,000
Assumed SS reduction, percent	60
SS reduction, mg/l	180
SS reduction, lb/day	34,000
Primary effluent, domestic only:	
BOD, mg/l	175
BOD, lb/day	33,000
SS, mg/l	120
SS, lb/day	22,000
Oxidation ponds:	
Number	2
Area, acres	425
Average depth, feet	4.25
Mechanical aerators:	
Number	24
Maximum power, input to rotors, hp	1,800
Efficiency, lbs O_2 input per hph	1.86
Oxygen input, lb/day	76,500
Loading, 5-day BOD, total lb/day:	
Noncanning season	33,000
Canning season	141,000
5-BOD reduction capacity:	
Noncanning season (winter months), photosynthetic:	
Unit, lb/acre/day	80
Total, lb/day	35,000

Table IV-2.—*Sunnyvale water-pollution-control-plant design data, 1971—Concluded*

Component	Quantity
Oxidation ponds—Continued	
Canning season (summer months):	
Photosynthetic:	
Unit, lb/acre/day	175
Total, lb/day	77,000
Mechanical aeration, lb/day	59,000
Photosynthetic plus mechanical aeration, lb/day	136,000
Detention, days:	
Noncanning season	27
Canning season	20
Circulation pumps:	
Number	4
Capacity each, mgd	63.5
Head, feet	3.5

Table IV-3.—*BOD_5 removals during canning season by ponds before and after installation of aerators*

Season	Pond influent BOD		Pond effluent BOD		Percent removal[a]
	mg/l	10^3 lb/day	mg/l	10^3 lb/day	
July 8-Oct. 1, 1970 (before aerators)	347.2	[b]67	64	[c]7	89
June 30-Oct. 2, 1971 (after aerators)	405.5	[d]64	29	4	94

[a]Based on mass emission, lb/day.
[b]Maximum value, 102,000 lb/day; effluent value is fairly consistent.
[c]Does not include BOD in effluent from industrial holding pond.
[d]Maximum value, 121,000 lb/day.

Table IV-4.—*Summary of capital costs for Sunnyvale aerators*

Item	Cost in dollars
Aerators (24)	587,000
Levee riprap	40,000
Aerator anchor blocks (4.5 cubic yards per aerator)	28,950
Pond transfer pipes (6 installed; more may be needed at other installations)	32,000
Pond power-load centers (5)	138,710
Direct burial cable	140,000
Main switch gear	24,000
Unload, position, and hook up aerators	31,900
Total	1,022,560

Table IV-5.—*Operating costs associated with pond upgrading*

Item	Cost in dollars	
	1970 (before aerators)	1971 (after aerators)
Gas and electricity[a]	15,000	58,000
Chemicals[b]	54,000	0
Labor[c]	0	10,000
Total	69,000	68,000

[a]Includes power for remainder of plant, which was also expanded in 1971.
[b]Calcium and sodium nitrate, phosphoric acid, and anhydrous ammonia.
[c]One employee added in 1971.

Ammonia and Algae Removal

Although the 1971 modifications improved the effluent quality of the Sunnyvale plant, the Federal secondary treatment requirement of 30 mg/l biochemical oxygen demand (BOD) and suspended solids (SS) could not be met. Moreover, the California Regional Water Quality Control Board, San Francisco Bay Region, has determined from studies carried out over the past decade that, to protect the water quality of south San Francisco Bay, existing facilities must produce an effluent of a quality higher than that defined as secondary-treatment quality. Effluent-quality requirements for Sunnyvale include those presented in table IV-6. The requirement for nondissociated ammonia in the receiving water necessitates removal (or conversion to nitrate) of ammonia in the wastewater because of limited dilution available.

Table IV-6.—*Summary of Sunnyvale wastewater discharge requirements*

Constituent	Criteria		
	30-day average	Maximum daily	Instantaneous maximum
Effluent limitations:			
BOD:			
mg/l	10	20	
lb/day	3,650	7,300	
SS:			
mg/l	10	20	
lb/day	3,650	7,300	
Oil and grease:			
mg/l	5	10	
lb/day	1,825	3,650	
Chlorine residual (as Cl_2), mg/l			0
Settleable matter, ml/l/h	.1		0.2
Turbidity, Jtu			10
Receiving water limitations:			
Nondissociated ammonium hydroxide (as N), mg/l			0.025
DO, mg/l	5.0 minimum; annual median of 80 percent saturation.		
pH	Effluent pH must not vary from ambient pH by more than 0.2 pH units.		

To select the treatment scheme that most fully satisfies the discharge requirements as well as engineering and economic constraints, an analysis was made of all potentially feasible alternatives. Two basic alternatives and five subalternatives were identified.[1] The two basic alternatives were:

- To retain and upgrade existing treatment facilities and processes and provide additional treatment facilities to meet the requirements.
- To retain existing primary facilities, abandon oxidation ponds (use them as holding basins), provide secondary and tertiary treatment processes, and retain existing sludge-handling and -disposal facilities.

The five subalternatives and their cost estimates are given in table IV-7. The group 1 alternatives, involving retention of the ponds and use of tertiary facilities, were less expensive than the group 2 alternatives.

The five feasible plans were evaluated as to their compliance with water-quality goals, flexibility and reliability, cost-effectiveness, reclamation potential, and environmental and social impacts. The apparent best alternative project was subalternative 1(c), which consisted of adding to the existing facilities dissolved-air flotation and filtration for algae removal, a fixed-growth reactor (trickling filter) for ammonia removal, breakpoint chlorination for supplemental ammonia removal, and dechlorination for toxicity control.

Table IV-7.—*Sunnyvale treatment alternatives*
[Thousands of dollars]

Item	Group 1. Existing primary treatment and oxidation ponds, plus flotation, filtration, and dechlorination			Group 2. Existing primary treatment, plus activated sludge, filtration, and dechlorination	
	Plus breakpoint chlorination, alternative 1(a)	Plus ammonia adsorption on clinoptilolite, alternative 1(b)	Plus nitrification in fixed-growth reactor, alternative 1(c)	Plus nitrification in fixed-growth reactor, alternative 2(a)	Plus breakpoint chlorination, alternative 2(b)
Capital costs[a]	6,078	9,662	9,010	17,620	14,685
Annual operation and maintenance costs:					
Existing treatment	603	603	603	480	480
New treatment	1,158	740	497	473	1,130
Total	1,761	1,343	1,100	953	1,610
Annual cost of capital investment[b]	573	911	850	1,662	1,386
Total annual cost of treatment	2,334	2,254	1,950	2,615	2,996

[a]*Engineering News Record,* Construction Cost Index 2800, Jan. 1976.
[b]Interest at 7 percent over a 20-year planning period.

To optimize the design and operational efficiency of the tertiary treatment unit processes, extensive pilot-plant studies were carried out in 1973 and 1974.[1] The results of these pilot studies, discussed in chapter III, formed the basis for plant design. Design data for the tertiary facilities are presented in table IV-8. Design criteria for nitrification in fixed-growth reactors (FGR) are presented in chapter 4 of the Technology Transfer publication, *Process Design Manual for Nitrogen Control.*[2]

Figure IV-5 shows the flow diagram for the tertiary facilities under two operational modes. Under mode 1, pond effluent undergoes dissolved-air flotation ahead of nitrification in the FGR. Mode 2 reverses the order of these two unit processes.

The principal advantage of mode 2 operation is reduced chemical costs, as shown in table IV-9.[3] Nitrification produces acidity, and therefore can be used to offset, or perhaps eliminate, the required acid addition for pH adjustment to optimize dissolved-air flotation when nitrification is first in the flow diagram. Furthermore, under mode 2, less caustic would need to be added to raise the pH before discharge. However, because an FGR of a given size will produce higher effluent ammonia levels under mode 2 than under mode 1, chlorine costs for supplemental breakpoint chlorination are higher for mode 2 when breakpoint chlorination is required, which partly offsets the advantage of mode 2. Receiving-water requirements indicate that breakpoint chlorination will be required for interim shallow-water discharge requirements, which will apply until an outfall into the bay is constructed. For deepwater discharge through the outfall, an effluent ammonia-nitrogen requirement of 4.0 mg/l is anticipated; for the interim shallow-water discharge, 0.5 mg/l. The pilot studies demonstrated that the practical limit for the FGR effluent ammonia-nitrogen concentration is about 2 to 3 mg/l.

The low construction bid for the project was $10,460,000 (September 1975), about $2 million below the engineer's estimate. (It was believed that a lack of available construction projects in the

Table IV-8.—*Design data, Sunnyvale tertiary treatment facilities*

Parameter	Value
Basic design loadings:	
Design population, thousands	124
Design flow, mgd:[a]	
Canning season	24
Noncanning season	16
Maximum TSS, mg/l	175
Pond effluent pH, maximum	8
Peak ammonia loading as N, mg/l:	
High-temperature operation[b]	25
Low-temperature operation[b]	22
Plant influent structures:	
Pond pumping station:	
Number of pumps	3
Capacity each, mgd	8
Total dynamic head, feet	20
Biological nitrification:	
FGR pumps:	
Number	3
Capacity each, mgd	16
Total dynamic head, feet	36
FGR's:	
Number	3
Diameter, feet	92
Media depth, feet	19
Top surface area per unit, ft^2	6,650
Total media volume, 1,000 ft^3	379
FGR media unit surface area, ft^2	42
Hydraulic loading, gpm/ft^2	1.7
Recirculation ratio, percent	100
Surface loading rate, ft^2/lb NH_3 oxidized per day	5,000
Ammonia conversion as N, mg/l:	
Mode 1:[c]	
High-temperature operation[b]	21
Low-temperature operation[b]	15
Mode 2:[c]	
High-temperature operation[b]	16
Low-temperature operation[b]	11
Solids removal:	
Dissolved-air flotation system:	
Number of units	3
Diameter, feet	60
Sidewater depth, feet	7
Area per unit, ft^2	2,820
Flow rate per unit, mgd	8
Surfacing loading rate, $gal/min/ft^2$	2.0

Table IV-8.—*Design data, Sunnyvale tertiary treatment facilities—Continued*

Parameter	Value
Solids removal—*Continued*	
Dissolved-air flotation system—*Continued*	
Solids loading rate, $lb/ft^2/day$	4.2
Influent pressurization flow, percent of total	25
Air to solids ratio, lb air per lb influent solids	0.10
Pressurization level, psig	80
Influent pH	6.0-6.3
Assumed TSS removal, percent	75
Assumed TSS removal per unit, lb/day	9,000
Float-removal system:	
Assumed float-production rate, gal/min/unit	114
Assumed solids concentration, percent	2
Assumed float density, lb/ft^3	31
Float ejectors:	
Number	6
Capacity each, gal/min	75
Design TDH, feet	8
Float mixers	
Number	2
Horsepower each	15
Float pumps:	
Number	3
Capacity each, gal/min	125
Discharge pressure, psi	60
Effluent filtration:	
Dual-media filters:	
Number	3
Dimensions per half filter:	
Length, feet	32
Width, feet	15
Area per filter, ft^2	960
Filtration rate, $gal/min/ft^2$	5.8
Maximum backwash rate, $gal/min/ft^2$	35
Air backwash:	
Rate, $ft^3/min/ft^2$	4
Pressure, psig	5
Assumed filter bed expansion, percent	10
Filter media depth, inches	66
Anthracite:	
Depth, inches	48
Effective size, mm	1.18
Sand:	
Depth, inches	18
Effective size, mm	0.94
Pea gravel depth, inches	7.5
Filtered-water pumping station:	
Number of pumps	4
Capacity each, mgd	8
Total dynamic head, feet	16

Table IV-8.—*Design data, Sunnyvale tertiary treatment facilities—Concluded*

Parameter	Value
Effluent filtration—*Continued*	
Backwash-water-pumping station:	
Number of pumps	3
Capacity each, gal/min	8,400
Total dynamic head, feet	20
Breakpoint chlorination:	
Level, NH_3-N, mg/l	8
Cl_2/NH_3 dosage ratio	10:1
pH control point	7.0
Maximum Cl_2 dose, mg/l	80
Disinfection:	
Chlorine contact tank:	
Maximum Cl_2 dose, mg/l	15
Cl_2 mixer horsepower	10
Number of cells	3
Dimensions per cell:	
Width, feet	10
Depth, feet	12
Length per pass, feet	124
Total volume, 1,000 ft^3	134
Detention time, minutes	60
Dechlorination:	
Dechlorination mixing basin:	
Maximum SO_2 dose, mg/l	12
SO_2 mixer horsepower	10
Chemical treatment:	
Chlorine feed capacity, 1,000 lb/day	24
Sulfer dioxide feed capacity, 1,000 lb/day	6
Sulfuric acid:	
Feed capacity, 1,000 lb/day	20
Maximum dosage rate, mg/l	98
Alum, $Al_2(SO_4)_3 \cdot 14.3\ H_2O$:	
Feed capacity, lb/day	30
Maximum dosage rate, mg/l	150
Polyelectrolyte:	
Feed capacity, lb/day	1,000
Maximum dosage rate, mg/l	5
Caustic soda:	
Feed capacity, 1,000 lb/day	34
Maximum dosage rate, mg/l:	
With breakpoint chlorination	170
Without breakpoint chlorination	80

[a] Includes allowance for 6.7 percent backwash recycle flow for dual-media filters.

[b] High-temperature range = 13° C to 19° C. Low-temperature range = 7° C to 11° C.

[c] Mode 1 operation = nitrification of dissolved-air flotation tank effluent. Mode 2 operation = nitrification of pond effluent, before dissolved-air flotation.

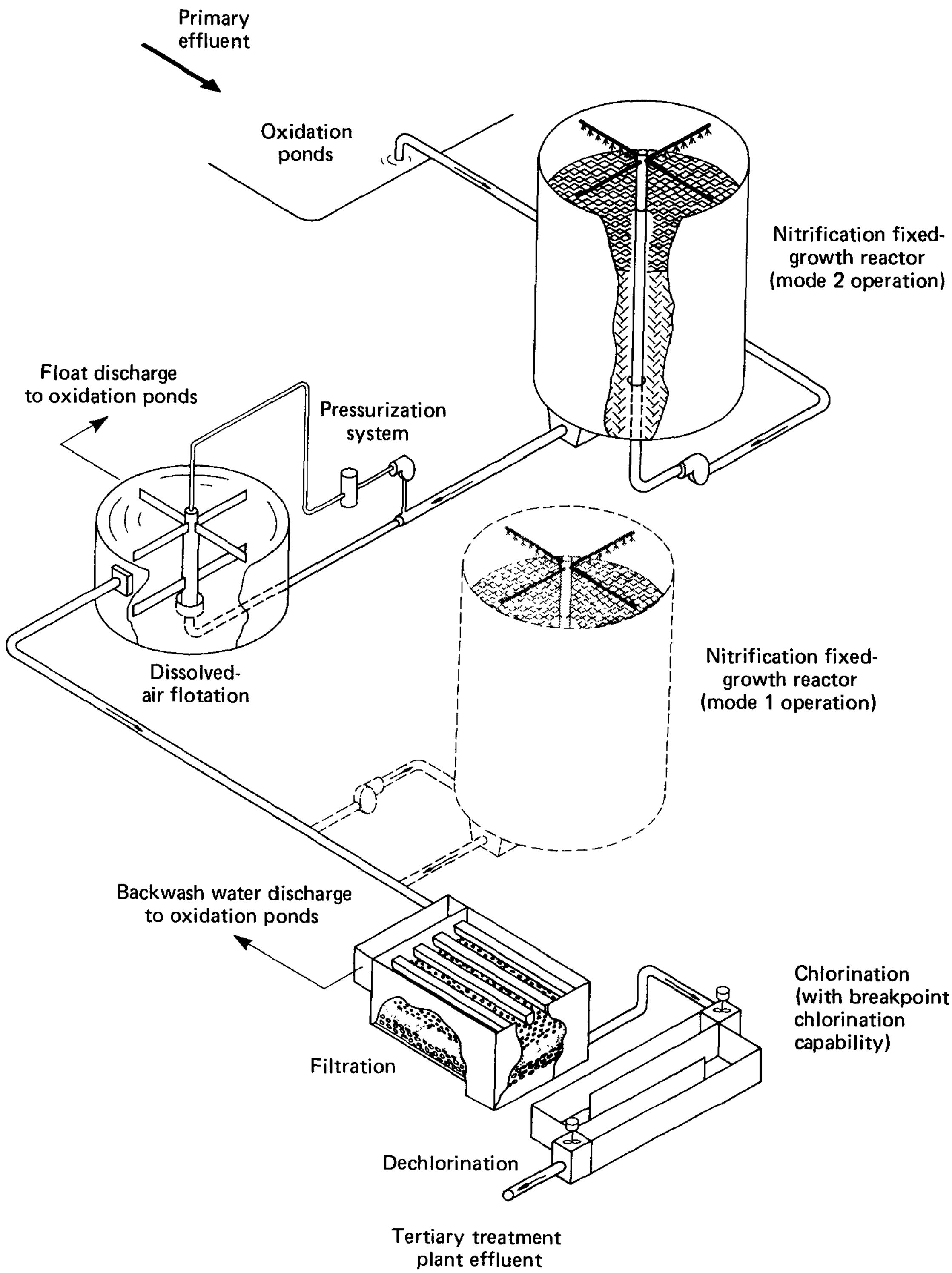

Figure IV-5. Sunnyvale tertiary facilities flow diagram.

area at the time resulted in a bid lower than would normally have been expected.) Construction began in January 1976 and is expected to be completed in November 1977. A breakdown for the construction cost is shown in table IV-10. Estimated 1978 annual operation and maintenance costs

Table IV-9.—*Comparative chemical costs, Sunnyvale tertiary facilities*

Chemical	Unit cost, dollars per ton	Annual costs, thousand dollars			
		With breakpoint chlorination		Without breakpoint chlorination	
		Mode 1	Mode 2	Mode 1	Mode 2
Chlorine	200	170.7	231.5	85.3	85.3
Acid	44	125.2	8.4	125.2	8.4
Caustic	140	498.1	401.1	318.7	159.4
Total		794.0	641.0	529.2	253.1

Table IV-10.—*Sunnyvale tertiary-treatment facilities construction costs, September 1975*[3]

Item	Capital cost, dollars
Mobilization	35,000
Site work	220,000
Pond pump station	220,000
Pond access bridge	190,000
Tower pump station	260,000
FGR's	2,400,000
Flotation distribution structure	300,000
Flotation tanks	1,375,000
Control building	775,000
Blower and chemical-feeder building	640,000
Dual-media filters	1,375,000
Filtered-water pump station	300,000
Chlorine mixer and distribution structure	160,000
Chlorine contact tanks	160,000
Dechlorination tank	100,000
Backwash pump station	120,000
Outside piping	675,000
Chemical storage tanks	200,000
Chlorination building	170,000
Dechlorination system	12,000
Temporary outfall facilities	23,000
Primary/secondary plant improvements	750,000
Total construction cost	10,460,000

are $1,500,000 per year with breakpoint chlorination and $900,000 per year without it. This estimate corresponds to unit costs of $230 and $140 per million gallons, respectively, for flows of 16 mgd during 8 months of the noncanning season and 22 mgd during 4 months of the canning season.

CASE 2. STOCKTON REGIONAL WASTE WATER CONTROL FACILITY

The city of Stockton, Calif., located near the confluence of the San Joaquin and Sacramento rivers, has an unusual water-quality problem that requires a unique solution. Historically, the cities of the San Joaquin Valley, particularly Stockton, have been agriculturally oriented. This orientation has resulted in industries that produce unusually heavy loading at the city's Regional Waste Water Control Facility during peak canning periods.

Stockton serves six canners and six other major wet industries, including food processors, in its municipal system. In the summer of 1975, these industries caused a peak monthly flow of 40 mgd to the city's treatment plant. BOD loading during that period reached a high of 5,300,000 lb/mo. Flows during the remainder of the year are 16 mgd, with 1,300,000 lb/mo. of BOD. Unfortunately, the peak occurs at the period of critical water quality and low flow in the San Joaquin River, a tidal estuary of San Francisco Bay, into which the plant's effluent is discharged.

The Central Valley Regional Water Quality Control Board has established discharge requirements that include the following provisions:

- The waste discharge shall "not cause the dissolved oxygen of the receiving waters to fall below 5.0 mg/l at any time."
- The waste discharge shall "not cause the total nitrogen control of receiving waters to fall below 3.0 mg/l at any time."

A study of the DO dynamics of the Stockton ship channel, which provides a deepwater link to San Francisco Bay, established the assimilative capacity of the channel for oxygen-demanding materials discharged from the Stockton Regional Waste Water Control Facility.[4] The long-term oxygen demand was found to be associated principally with algae; therefore, physical removal of the algae from the pond effluent eliminated most of the long-term BOD. A projection of long-term BOD loads compared with the assimilative capacity of the river indicated that algal removal would permit the DO criterion to be met. At the same time, algal removal would also accomplish nitrogen removal, because most of the nitrogen is in organic form and associated with algae.

To meet the new requirements, Stockton is currently enlarging and modifying its treatment plant. A phased design and construction program has been prepared that will enable the city to be in compliance with waste-discharge requirements by 1978. This program involves improvements to the entire plant, including the following elements (fig. IV-6):

- Preliminary treatment
- Primary sedimentation
- Secondary treatment (trickling filtration)
- Tertiary treatment (oxidation ponds and algal-removal facilities)
- Disinfection
- Solids treatment

Pilot Algae-Removal Studies

Pilot studies were conducted at the Stockton plant during the summer of 1971 to develop design criteria for the tertiary algae-removal facilities. At that time pilot-scale and plant-scale tests

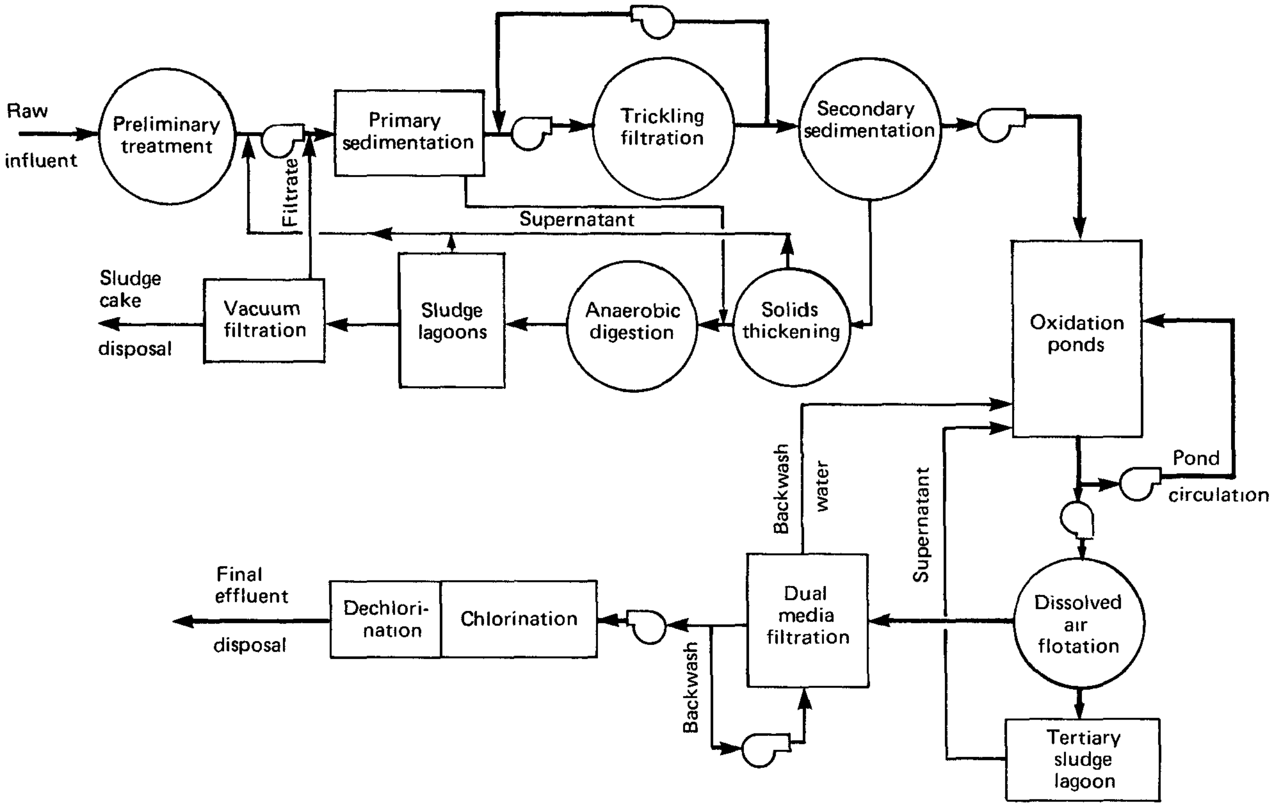

Figure IV-6. Stockton regional wastewater control facility flow diagram.

had established both coagulation-flocculation-sedimentation and coagulation-flotation as workable, dependable procedures for removal of algae from pond effluents. An economic analysis indicated that flotation would be superior to sedimentation because of higher allowable overflow rates and shorter residence times. It was anticipated that greater sludge concentration could be obtained at approximately the same chemical dose, and smaller tanks could be used.

Because of these anticipated advantages of flotation over sedimentation, it was decided to operate a pilot flotation process to determine if flotation was applicable to Stockton's wastes and to develop design concepts and criteria for a full-scale unit.[5] Of particular interest was the comparison of pressurized dissolved-air flotation with autoflotation. Results of the studies indicated that while autoflotation exhibited a potential for algae removal, its overall performance was erratic because there was DO supersaturation in the ponds for only a part of the day. Some of the pilot-study results are summarized in chapter III.

Full-Scale Tertiary Facilities

Studies of dissolved-air flotation showed the process to be feasible, and it was chosen subsequently for use in the full-scale facility. In addition to dissolved air-flotation, effluent polishing will be provided by dual-media filtration. Breakpoint chlorination will also be available for ammonia removal if it is required at those times when the dissolved-air-flotation unit is not being operated. Effluent disinfection and dechlorination facilities will also be provided. Construction of the tertiary facilities started in March 1976 and will be completed in September 1978. A flow diagram for the tertiary-treatment facilities is shown in figure IV-7 and design data are given in table IV-11.

Bids for construction of the tertiary facility were opened on January 27, 1976, and ranged from $16,600,000 to $18,800,000. Annual operation and maintenance costs are estimated at

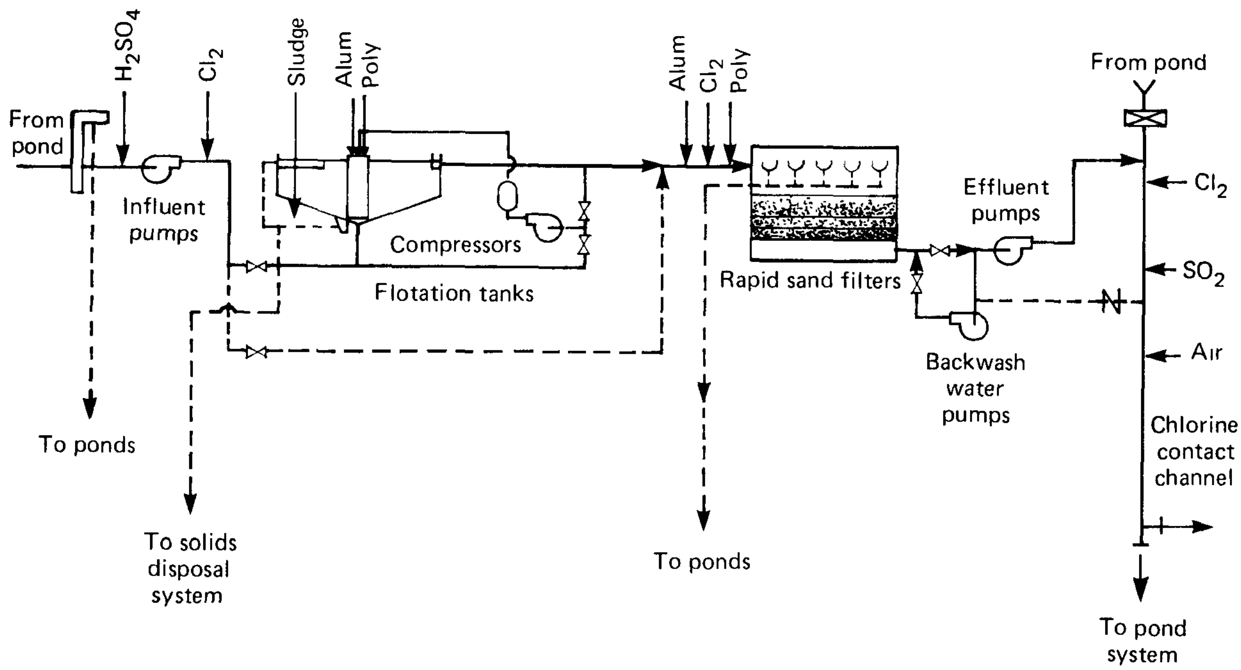

Figure IV-7. Stockton tertiary facilities flow diagram.

Table IV-11.—*Design data, Stockton tertiary facilities*

Parameter	Value
Tertiary ponds, existing:	
Number	4(4)
Area, net water surface, acres	630
Volume, million gallons	1,320
Loading during noncanning season:	
BOD total, 1,000 lb/day	3.2
BOD, pounds per surface acre per day	5
Loading during canning season:	
BOD total, 1,000 lb/day	57
BOD, pounds per surface acre per day	90
Detention, days:	
During noncanning season	57
During canning season	23
Circulation pumping units:	
Number	4
Capacity each, mgd	60
Circulation ratio, at peak	4.4
Dissolved-air-flotation loadings:	
Flow, mgd	55
SS concentration, mg/l	170
pH, peak	9.5
Ammonia, peak concentration, mg/l	6.5

Table IV-11.—*Design data, Stockton tertiary facilities—Continued*

Parameter	Value
Chemical treatment:	
Alum, peak rates:	
Dry dose, mg/l (17 percent Al_2O_3)	250
Volume, 1,000 gal/day (8.3 percent Al_2O_3)	21.2
Sulphuric acid, peak rate (93 percent H_2SO_4):	
Dose, meq/l	3.0
Volume, gal/day	4,700
Polyelectrolyte, peak rate (0.5 percent solution):	
Dose, mg/l	2.0
Volume, gal/min	15.0
Chlorine, peak capacities:	
Prechlorination:	
mg/l	17.5
1,000 lb/day	8
Filter influent:	
mg/l	17.5
1,000 lb/day	8
Disinfection:	
mg/l	5
1,000 lb/day	2.3
Ammonia removal:	
mg/l	105
1,000 lb/day	48
Dechlorination:	
Sulphur dioxide, peak rate, mg/l	8.3
1,000 lb/day	3.8
Raw-water pumping station:	
Traveling water screens:	
Number	3
Capacity each, mgd	43
Basket width, each, feet	5
Depth through screen, feet:	
Low-pond elevation	4
High-pond elevation	7
Velocity through screen, peak, ft/s:	
Low-pond elevation	2.5
High-pond elevation	1.5
Raw water pumps:	
Number	4
Capacity each, mgd	13.75
Total head each, feet	11.0
Flotation tanks:	
Number	4
Diameter each, feet	85
Side water depth, feet	7
Solids loading rate, lb/ft^2/day	5.1

Table IV-11.—*Design data, Stockton tertiary facilities—Continued*

Parameter	Value
Flotation tanks—*Continued*	
Assumed float concentration, percent	3
Assumed float weight, lb/ft^3	41
Peak float-discharge rate, gal/min	600
Surface loading rate, including pressurized flow, $gal/min/ft^2$	2.4
Pressurized flow, gal/min	4,500
Pressure, maximum, psig	80
Air flow, maximum, scfm	80
Air/solids ratio, minimum, lb air per lb solids:	
Mode 1	0.179
Mode 2	0.179
Dual-media filters:	
Number (bifurcated)	4
Width, feet	34
Length, feet	50
Filtration rate, $gal/min/ft^2$:	
All filters in service	5.7
One in backwash	7.5
Media:	
Anthracite coal:	
Depth, feet	4
Effective size, mm	1.0-1.1
Sand:	
Depth, feet	1.5
Effective size, mm	0.65-0.75
Gravel depth, feet	0.67
Backwash:	
Air:	
Rate, $ft^3/min/ft^2$	4
Volume, ft^3/min	3,400
Water:	
Rate, $gal/min/ft^2$:	
Minimum	13
Maximum	26
Volume, mgd:	
Minimum	16.0
Maximum	32.0
Filtered-water pumping station:	
Number of pumps	3
Capacity each, mgd	21.5
Total head, feet	15.7
Chlorine contact canal:	
Length, feet	1,030
Average width, feet	19.26

Table IV-11.—*Design data, Stockton tertiary treatment facilities—Concluded*

Parameter	Value
Chlorine contact canal—*Continued*	
Depth, feet	7.63
Detention time, minutes	30
Reaeration blowers:	
Number	2
Capacity each, ft^3/min	1,500

$1,100,000 per year, based on 3 months of operation. This corresponds to a unit cost of $250 per million gallons.

CASE 3. ANTELOPE VALLEY TERTIARY TREATMENT PLANT--LANCASTER

The Los Angeles County Sanitation Districts (LACSD), at their Antelope Valley Tertiary Treatment Plant in Lancaster, Calif., have the longest operating experience record with a coagulation-flocculation-sedimentation algae-removal system. The plant was designed and constructed in 1970 by the LACSD after several years of process research conducted in cooperation with the U.S. Public Health Service.[6] The purpose of the facility is water reclamation. Algae and phosphorus are removed from the oxidation pond effluent by alum precipitation, settling, and filtration (fig. IV-8). Sludge is disposed of by pumping it back to the treatment plant's headworks. Treatment ahead of the tertiary plant consists of primary sedimentation followed by oxidation ponds. Primary sludge is processed through digestion.

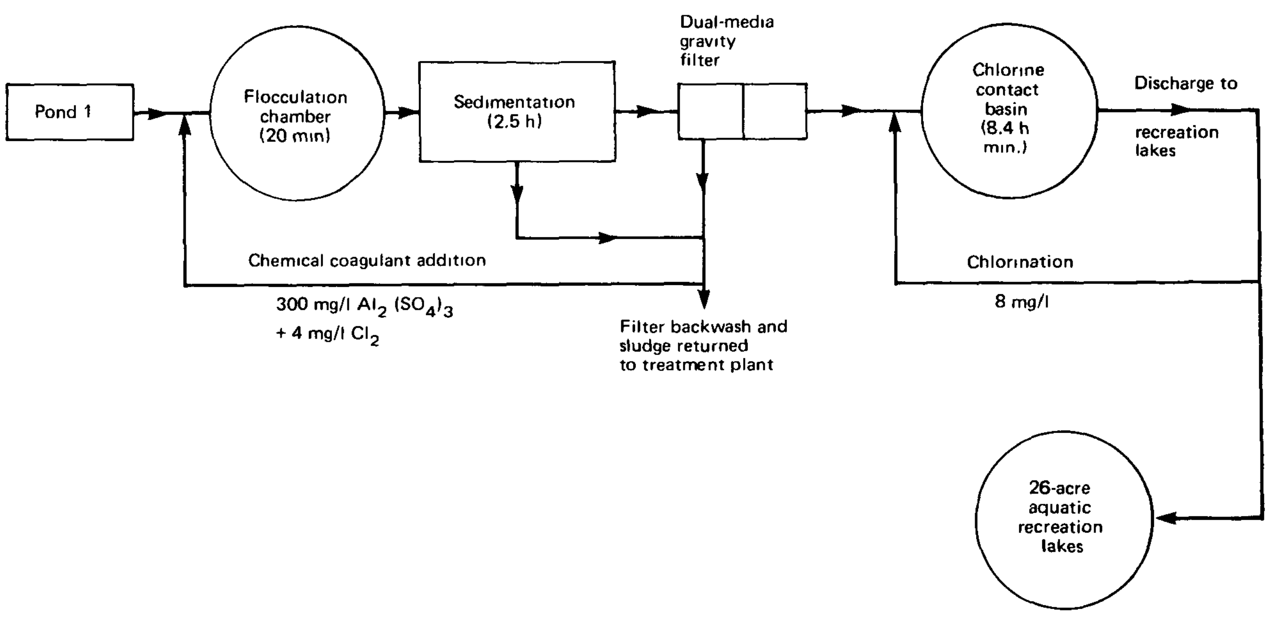

Figure IV-8. Antelope Valley tertiary treatment plant flow diagram.

Table IV-12.—*Antelope Valley tertiary-treatment-plant operation*

Parameter	February 1973	September 1973
Influent[a] flow-rate, mgd	0.089	0.563
Percent of flow returned[b]	16	17
Aluminum sulfate dose, mg/l	[c]243 ± [d]44	337 ± 31
Flocculation pH range	6.2 to 6.6	6.3 to 6.8
Mean time between filter backwash, hours	112	14
Influent quality:		
SS, mg/l	156 ± 19	129 ± 10
Alkalinity, mg/l[e]	234 ± 4	251 ± 3
Total phosphate, mg/l[f]	30.2 ± 5.9	25.6 ± 5.0
pH range	9.3 to 9.8	9.3 to 9.5
Temperature, °C	9 ± 2	18 ± 2
Effluent quality:		
Turbidity, Jtu	1.0 ± 0.3	0.8 ± 0.3
Total phosphate, mg/l[f]	0.19 ± 0.05	0.15 ± 0.05

[a]Influent to tertiary plant.
[b]Backwash water and sludge returned to headworks.
[c]Mean value (typical).
[d]Standard deviation (typical).
[e]As $CaCO_3$.
[f]As PO_4.

Effluent from the Lancaster plant is delivered to the county of Los Angeles for use in recreational lakes. The 0.5-mgd facility was completed at a construction cost of $243,000. Cost of operation and maintenance for fiscal year 1973-74 was $304 per million gallons, exclusive of amortization. Seasonal flows vary. During summer months at design capacity, operation and maintenance costs are $200 to $240 per million gallons; in winter months at low flows, unit operation and maintenance costs are in the range of $600 to $800 per million gallons.[7] Operating costs are borne by the County Parks and Recreation Department.

Operating data for 2 representative months of operation are shown in table IV-12.[7] Summertime flows are at design capacity, while wintertime flows decline because of seasonal evaporation-precipitation patterns and because no releases are made from the lakes in the winter. Operating data confirm that the plant removes algae as well as phosphates very efficiently and consistently. Alum doses are higher than would normally be required for algae removal alone. The preliminary pilot work demonstrated that effluent phosphate levels on the order of 0.05 mg/l would aid in preventing algae regrowth in the lakes.[6] It was projected that aluminum sulfate doses of about 300 mg/l (525 mg/l as alum $[Al_2(SO_4)_3 \cdot 14.3\,H_2O]$) would be required for obtaining low phosphate residuals, whereas only about 70 to 120 mg/l of aluminum sulfate (120 to 210 mg/l as alum) is normally required for removing algae when phosphorus is not a critical problem. For those facilities operated exclusively for algae removal, therefore, operation and maintenance costs should be lower than at Lancaster.

CASE 4. RICHFIELD SPRINGS SEWAGE-TREATMENT PLANT

Richfield Springs is a town of approximately 1,600 persons, located in central New York State about 60 miles west of Albany. One of the first communities of its size to install sanitary sewers, Richfield Springs installed the initial lines in 1895 and added sewers and a primary plant in 1927. As a result of increasingly stringent discharge requirements developed to protect nearby Lake Canadarago from eutrophication, a new treatment facility was designed and constructed in 1972.

Upgraded Plant

The flow diagram for the upgraded facility that provides phosphorus removal is shown in figure IV-9, and design data are given in table IV-13. Treatment consists of two series-operated aerated lagoons followed by a package tertiary-treatment unit (two Neptune Micro-Floc SWB 150's) providing flocculation, tube settling clarification, and mixed media filtration. Chlorine disinfection precedes discharge to Ocquionis Creek.

Measured daily flows have ranged from less than 0.05 mgd to 2.5 mgd under drought and storm conditions, respectively. The aerated lagoons can treat a peak wet-weather flow of 2.5 mgd, caused by the combination of very old sewers and a periodically high ground water table. The tertiary units have a design capacity of 0.3 mgd. When the flow into the lagoon exceeds the tertiary-treatment capacity, the lagoons act as an equalizing reservoir. Lagoon effluent bypasses the tertiary units to the chlorine contact tank when the lagoon's storage capacity is reached.

After pretreatment, wastewater is pumped to lagoon 1. From there it flows by gravity to lagoon 2 and is pumped to the tertiary unit. After tertiary treatment, the wastewater flows by gravity to the chlorine contact tanks before entering Ocquionis Creek. Flows bypassing the tertiary

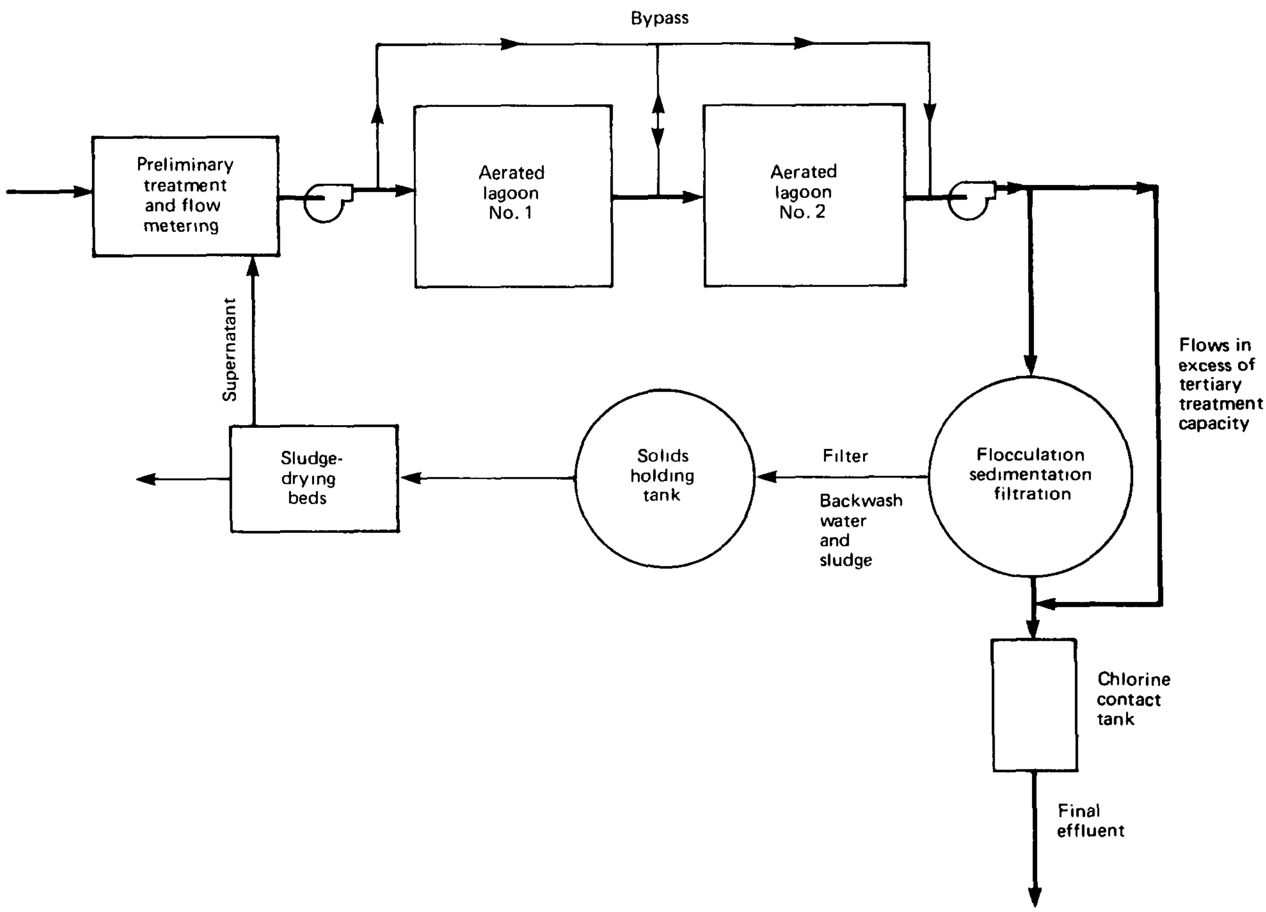

Figure IV-9. Richfield Springs sewage treatment plant flow diagram.[1]

Table IV-13.—*Design data, Richfield Springs Sewage Treatment Plant*

Parameter	Value
Flow, mgd:	
Average dry weather	0.30
Peak wet weather	2.5
Aerated lagoons:	
Number	2
Surface area, acres	2.25
Depth, feet	6-12
Detention time at average dry weather flow, days	18
Lagoon air supply:	
Number of compressors	2
Capacity each, ft^3/min	210
Aeration-tubing length, feet	12,000
Tertiary treatment units:	
Number	2
Design flow each, mgd	0.15
Alum dose, mg/l	70-120
Flocculator detention time, minutes	20
Tube settler overflow rate, $gal/min/ft^2$	2.6
Multimedia filter:	
Depth, total, inches	30.0
Garnet	4.5
Sand	9.0
Anthracite	16.5
Specific gravity:	
Garnet	4.2
Sand	0.6
Anthracite	1.5
Effective size, mm:	
Garnet	0.30
Sand	0.45
Anthracite	1.00
Loading rate, $gal/min/ft^2$	4.0
Backwash rate, $gal/min/ft^2$	16
Backwash flow each, mgd	0.03

unit flow by gravity to the chlorine contact tank. The tertiary unit backwash water and solids enter a large holding tank where solids settling takes place. The supernatant is pumped back to the plant headworks, and the sludge is put on drying beds or spread on land.

The lagoons use 12,000 feet of Air-Aqua aeration tubing with the closest tube spacing near the inlet of the first lagoon. The design aeration air supply, using one of two blowers, is 210 ft^3/min. The lagoon surface area is about 2.25 acres with a volume of 26 acre-ft at a depth of 11.5 feet. Normal operating depths range from 6 to 12 feet.

Each Neptune Micro-Floc SWB 150 unit is designed for a flow of 150,000 gal/day. The tube settlers (see fig. III-3) contain 39-inch-long tubes with a cross-sectional area of 2.0 in^2 each, placed on a slope of 7.5 degrees from the horizontal. The 30-inch-deep filter bed is composed of garnet, sand, and anthracite. The filter-loading rate is 4.0 gal/min/ft^2. The filters are backwashed for 8-10 minutes every 4 to 5 hours.

Plant Performance

In 1973 and 1974, the New York State Department of Environmental Conservation, Environmental Quality Research Unit, conducted an extensive monitoring program at the Richfield Springs plant[8] as a part of a larger study on eutrophication of Canadarago Lake. Raw sewage, lagoon effluent, and final effluent were sampled and analyzed for BOD, chemical oxygen demand (COD), SS, nitrogen compounds, phosphorus, sulfate, turbidity, coliform organisms, and DO.

Tables IV-14 and IV-15 show plant performance for a full 2-year period and a low-flow, summertime period, respectively. The data in table IV-15 represent a period when algal activity in the lagoons would be greatest and the raw wastewater strongest. For the long-term period (table IV-14), BOD and SS removals each averaged 94 percent. Total phosphorus removal averaged 87 percent. Final effluent concentrations for BOD, SS, and total phosphorus averaged 4.0, 7.0, and 0.34 mg/l, respectively.

Table IV-14.—*Richfield Springs plant performance, February 1973 through February 1975*[8]

Constituent[a]	Raw influent	Lagoon effluent	Removal, percent	Final effluent	Lagoon-final removal, percent	Raw-final removal, percent
Flow, mgd	0.58	–	–	0.37	–	–
COD,[b] mg/l	145	51.3	64.6	23.2	54.6	84.0
BOD, mg/l	64.2	16.9	73.8	4.0	72.3	93.8
TSS, mg/l	103	30.7	70.0	7.0	77.2	93.2
TKN, mg/l	13.6	[c]6.4	52.9	5.3	17.0	61.0
NH_3 as N, mg/l	4.9	2.7	–	2.9	–	–
Total N, mg/l	15.2	8.2	46.1	6.9	15.8	54.7
NO_3 as N, mg/l	0.60	1.8	–	1.6	–	–
Total P, mg/l	2.63	1.18	55.1	0.340	71.2	87.1
Alk,[d] mg/l	212	191	–	134	–	–
SO_4,[b] mg/l	127	115	–	167	–	–
Turbidity, Jtu	–	34.2	–	0.97	97.2	–
pH	7.47	7.84	–	7.39	–	–
Coliforms, *MPN*/100 ml	–	[e]29,081	–	[e]302	99.0	–
DO, mg/l	–	11.0	–	10.9	–	–

[a]Based on a 2-week sampling frequency.
[b]Frozen sample data.
[c]May be low because of inclusion of some soluble organic N values in average.
[d]Field data; as $CaCO_3$.
[e]Logarithmic mean.

Table IV-15.—*Richfield Springs plant performance—September 25 through November 7, 1973 (low-flow period)*[8]

Constituent[a]	Raw influent	Lagoon effluent	Removal, percent	Final effluent	Lagoon-final removal, percent	Raw-final removal, percent
Flow, mgd	0.36	–	–	0.36	–	–
COD,[b] mg/l	294	93.6	68.3	34.1	63.5	88.5
BOD, mg/l	98.5	28.5	71.0	4.55	84.0	95.4
TSS, mg/l	131.2	46	64.9	14	69.6	89.3
TKN, mg/l	15.0	[c]7.0	53.3	5.4	23.0	64.0
NH_3 as N, mg/l	5.3	5.4	–	3.2	–	–
NO_3 as N, mg/l	0.25	2.47	–	1.90	–	–
Total N, mg/l	15.3	9.5	37.9	7.3	76.8	52.3
Total P, mg/l	6.2	2.34	62.3	0.543	76.8	91.2
Alk,[d] mg/l	78.3	166.7	–	126	–	–
SO_4,[b] mg/l	170	199	–	206	–	–
Turbidity, Jtu	–	61	–	3.5	94.3	–
Coliforms, *MPN*/100 ml	–	[e]33,884	–	[e]1,053	96.9	–
DO, mg/l	–	10.1	–	8.4	–	–

[a]Based on a two-week sampling frequency.
[b]Frozen sample data.
[c]May be low because of inclusion of some soluble organic N values in average.
[d]Field data; as $CaCO_3$.
[e]Logarithmic mean.

Cost

The Richfield Springs plant was placed in operation in January 1973, at a total construction cost of $577,000. The cost attributable to the tertiary portion of the plant was about $170,000. A construction cost breakdown is given in table IV-16.

Table IV-16.—*Construction cost, Richfield Springs Sewage Treatment Plant, 1972*[8]

Item	Cost, dollars
Site work	60,000
Concrete work	52,000
Miscellaneous equipment	185,000
Buildings	38,000
Barminutor	7,000
Tertiary equipment	110,000
Standby power	15,000
Electrical	28,000
Fencing	10,000
Miscellaneous	47,000
Subsequent change orders	25,000
Total plant construction cost	[a]577,000

[a]Cost for tertiary system estimated at $170,000.

Table IV-17 gives operating costs for the Richfield Springs plant and estimated operating costs for the tertiary-treatment units. Table IV-18 presents treatment costs in dollars per year and dollars per million gallons.

Table IV-17.—*Operating costs, Richfield Springs Sewage Treatment Plant*[8]
[Dollars]

Item	Estimate for tertiary plant	Total
Labor	1,500	8,000
Power	500	4,000
Chemicals	3,500	4,500
Miscellaneous	500	2,500
Total	6,000	19,000

Table IV-18.—*Total treatment costs, Richfield Springs Sewage Treatment Plant*[8]
[Dollars]

Item	Plant		Total
	Secondary	Tertiary	
Amortized capital cost:[a]			
Per year	38,500	16,000	54,500
Per million gallons at 0.58 mgd	180	–	–
Per million gallons at 0.37 mgd	–	120	–
Operating cost:			
Per year	13,000	6,000	19,000
Per million gallons at 0.58 mgd	61	–	–
Per million gallons at 0.37 mgd	–	44	–
Total cost:			
Per year	51,500	22,000	73,500
Per million gallons at 0.58 mgd	241	–	–
Per million gallons at 0.37 mgd	–	164	–

[a]Amortized at 7 percent over 20 years.

REFERENCES

[1]R. W. Stone, D. S. Parker, and J. A. Cotteral, "Upgrading Lagoon Effluent for Best Practicable Treatment," *J. Water Pollut. Cont. Fed.*, *47*, 2019, 1975.

[2]U.S. Environmental Protection Agency, Office of Technology Transfer, *Process Design Manual for Nitrogen Control*, EPA-625/1-75-007, Cincinnati, Ohio, Oct. 1975.

[3]Brown and Caldwell, "Report on Design Criteria, Tertiary Facilities," prepared for the city of Sunnyvale, Feb. 1975.

[4]Brown and Caldwell, "Benefits of Proposed Tertiary Treatment to San Joaquin River Water Quality," prepared for the city of Stockton, Nov. 1970.

[5]D. S. Parker, J. B. Tyler, and T. J. Dosh, "Algae Removal Improves Pond Effluent," *Water Wastes Eng.*, *10*, 1, Jan. 1973.

[6]F. D. Dryden and G. Stern, "Renovated Wastewater Creates Recreational Lake," *Environ. Sci. Technol.*, *2*, 4, Apr. 1968.

[7]Personal communication, F. D. Dryden to D. S. Parker, County Sanitation Districts of Los Angeles County, May 1975.

[8]T. J. Tofflemire, I. G. Carcich, F. T. Martin, and R. Bloomfield, "Tertiary Treatment for Phosphorus Removal by Alum Addition," New York State Department of Environmental Conservation, Environmental Quality Research Unit, Technical Paper No. 59, Apr. 1975.

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METRIC CONVERSION TABLES

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Length	meter	m	*Basic SI unit*	39.37 in = 3.281 ft = 1.094 yd
	kilometer	km		0.6214 mi
	millimeter	mm		0.03937 in
	micrometer or micron	μm or μ		3.937 X 10^{-5} in = 1 X 10^{4} Å
Area	square meter	m^2		10.76 sq ft = 1.196 sq yd
	square kilometer	km^2		0.3861 sq mi = 247.1 acres
	square millimeter	mm^2		0.001550 sq in
	hectare	ha	The hectare (10,000 m^2) is a recognized *multiple unit and will* remain in international use	2.471 acres
Volume	cubic meter	m^3		35.31 cu ft = 1.308 cu yd
	litre	l		1.057 qt = 0.2642 gal = 0.8107 X 10^{-4} acre ft
Mass	kilogram	kg	*Basic SI unit*	2.205 lb
	gram	g		0.03527 oz = 15.43 gr
	milligram	mg		0.01543 gr
	tonne	t	1 tonne = 1,000 kg	0.9842 ton (long) = 1.102 ton (short)
Force	newton	N	The newton is that force that produces an acceleration of 1 m/s^2 in a mass of 1 kg	0.2248 lb = 7.233 poundals
Moment or *torque*	newton meter	N-m	The meter is measured perpendicular to the line of action of the force N. *Not a joule*	0.7375 lb-ft 23.73 poundal-ft
Flow (volumetric)	cubic meter per second	m^3/s		15,850 gpm = 2,119 cfm
	liter per second	l/s		15.85 gpm

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Velocity linear	meter per second	m/s		3.281 fps
	millimeter *per second*	mm/s		0.003281 fps
	kilometers per second	km/s		2,237 mph
angular	radians per second	rad/s		9.549 rpm
Viscosity	pascal second	Pa-s		0.6722 poundal(s)/sq ft
	centipoise	Z		1.450 X 10^{-7} Reyn (μ)
Pressure or stress	*newton per* square meter or pascal	N/m^2 or Pa		0.0001450 lb/sq in
	kilonewton per square meter or kilopascal	kN/m^2 or kPa		0.14507 lb/sq in
	bar	bar		14.50 lb/sq in
Temperature	Celsius (centigrade)	°C		(°F−32)/1.8
	Kelvin (abs.)	°K		°C + 273.2
Work, energy, quantity of heat	joule	J	1 joule = 1 N-m where meters are measured along the line of action of force N.	2.778 X 10^{-7} kw-hr = 3.725 X 10^{-7} hp-hr = 0.7376 ft-lb = 9.478 X 10^{-4} Btu
	kilojoule	kJ		2.778 X 10^{-4} kw-hr
Power	watt	W	1 watt = 1 J/s	44.25 ft-lbs/min
	kilowatt	kW		1.341 hp
	joule per second	J/s		3.412 *Btu/hr*

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Precipitation, run-off, evaporation	millimeter	mm	For meteorological purposes, it may be convenient to measure precipitation in terms of mass/unit area (kg/m^2) 1 mm of rain = 1 kg/m^2	
Flow	cubic meter per second	m^3/s		35.31 cfs
	liter per second	l/s		15.85 gpm
Discharges or abstractions, yields	cubic meter per day	m^3/d	1 l/s = 86.4 m^3/d	0.1835 gpm
	cubic meter per year	m^3/year		264.2 gal/year
Usage of water	*liter per person* per day	*l/person/* day		0.2642 gcpd

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents*
Density	kilogram per cubic meter	kg/m^3	The density of water under standard *conditions is* 1,000 kg/m^3 or 1,000 g/l or 1 g/ml	0.06242 lb/cu ft
Concentration	milligram per liter (water)	mg/l		1 ppm
BOD loading	kilogram per cubic meter per day	$kg/m^3/d$		0.06242 lb/cu ft/day
Hydraulic load per unit area, e.g., filtration *rates*	cubic meter per square meter per day	$m^3/m^2/d$	If this is converted to a velocity, it should be expressed in mm/s (1mm/s = 86.4 m^3/m^2/day)	3.281 cu ft/sq ft/day
Air supply	cubic meter or liter of free air per second	m^3/s l/s		
Optical units	lumen per square meter	lumen/m^2		0.09294 ft candle/sq ft

*Miles are U.S. statute, qt and gal are U.S. liquid, and oz and lb are avoirdupois

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