

A  
Final Report

to

Texas Electric Service

WASTEWATER EFFLUENT FOR POWER PLANT COOLING

by

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## SUMMARY

The objective of this investigation was to determine from literature sources if integration of wastewater treatment with power-plant cooling was technically and economically feasible. Moreover, it was necessary to point out the mutual beneficial aspects of the two functions and to delineate what further research was required. The investigation was limited to 400 and 1000 megawatt powerplants located in the north-central Texas region. The only sources of water considered in the study were raw and secondary treated effluents. Water quality requirements for any type of recreational use of the pond would not serve as constraints.

Based on the topography of the region, a 400 megawatt fossil fuel powerplant would require a 560 acre pond with an average wastewater flow of 5 MGD to account for evaporation and a similar 1000 megawatt system would need 1400 acres with a 20 MGD flow. Because of the long retention time, present BOD<sub>5</sub> effluent standards could be obtained from the proposed ponds using raw wastewater. The heat added by the condenser system in winter would be beneficial to the biological purification processes. To avoid probable summer time odor and nuisance conditions associated with high areal organic loading, maximum flows would be limited to 20 and 50 MGD raw wastewater, respectively, following plain sedimentation. Grease and other floatables in raw wastewater would need to be removed by pre-treatment processes. The growth of microorganisms in the pond would cause effluent suspended solids to be higher than present standards allow, and post-treatment would be required. Also, the coliform dieoff rate in the pond probably would not be sufficient to meet effluent standards. Salt buildup in the pond due to evaporation would limit the minimum wastewater flows at 10 and 40 MGD to 560 and 1400 acre ponds, respectively, for the assumptions made in this study. Pre-treatment processes for nutrient removal might be required to prevent condenser system difficulties or to improve aesthetic qualities of the pond even though eutrophic conditions could not be alleviated. To meet probable future effluent standards, further carbon, nitrogen and phosphorus removal would be required.

The costs of the potential systems are delineated and compared to present municipal systems as regards present and probable future effluent requirements. A significant economic advantage is apparent for the integrated system.

A detailed summary is presented of further research studies required to answer questions raised in this study. A suggested research plan and estimated budget also are outlined.



## INTRODUCTION

The expanding needs for electrical energy are imposing requirements for increased generating capacity on the electrical power industry. As new large-capacity powerplants are added, additional cooling waters will be needed; however, the sources of cooling waters will not be increasing and the water requirements for municipal, industrial, agricultural and other uses will be increasing.

Within the past decade the reuse of municipal wastewaters has been receiving greater consideration as a significant source of water for all uses (1,2,3). Such broad reuse of municipal wastewaters will require treatment beyond that presently employed, thereby increasing treatment costs.

The increased requirements for cooling waters confronting the electrical power industry and the required reuse of municipal wastewaters to meet water demands within the near future could initiate a new concept: an integrated wastewater treatment and powerplant cooling water system. Such an integrated system could insure a cooling water source for the utility. Municipal wastewaters represent a relatively constant source of water and, though not exempt, would be the least sensitive to drought conditions in areas with marginal water resources. However, the volume of wastewater flow would not be sufficient for once-through cooling methods and would require a recirculation system. The presence of waste energy in the form of heat would be available at no cost and could be an asset to biological processes assimilating the wastewater within the cooling pond, particularly during the winter. Also, other energy sources associated with powerplants would be available at minimum costs, particularly low transmission line expenses, for use in additional wastewater treatment processes necessary to meet required standards.

## SCOPE OF STUDY

To provide a range of operating conditions for evaluation of the concept of integrated systems, powerplant capacities of 400 and 1000 megawatts are considered. The powerplant will be located in the north-central Texas area.

Municipal wastewaters are assumed to be the only significant source of water to an integrated system. A flow of 50 million gallons per day (MGD) is taken as the most likely maximum flow available.

The water is assumed to be either a raw wastewater or a secondary effluent. Raw wastewater is defined as that which has received no treatment, and secondary effluent is defined as that which has been subjected to a form of treatment which removes approximately 80-90 percent of the organic concentration expressed as Biochemical Oxygen Demand (BOD) and 85-95 percent of the Suspended Solids (SS). Typical characteristics of these two types of wastewaters are shown in Table 1, and the indicated values were used for purposes of this report.

The requirements imposed on an integrated wastewater treatment and cooling water system must be defined within the constraints and imposed by the requirements for the separate systems. With respect to a cooling water system, the major areas of consideration are the required surface area for heat dissipation, the minimum required inflow to sustain the system losses (i.e., evaporation, seepage, additional uses), and adequate water quality for cooling purposes (i.e., to avoid scale deposition and condenser fouling). With respect to a wastewater treatment system incorporating the cooling pond as an integral process, major areas of consideration will be the range of hydraulic retention times provided by the cooling pond, whether the wastewater input has received any treatment prior to entering the system, the wastewater assimilative capacity of the cooling pond, and additional treatment processes required beyond the cooling pond to meet both required present and probable future effluent standards.

TABLE 1  
WASTEWATER CHARACTERISTICS

	Raw	Secondary
Biochemical Oxygen Demand (BOD <sub>ultimate</sub> )	300	25-50
Suspended Solids (SS)	200	25
Organic Nitrogen (as N)	10	5
Ammonia Nitrogen (as N)	25	20
NO <sub>3</sub> (as N)	0	0
Total Phosphorus (as P)	15	10

In developing potential systems, available data from studies reported in the literature and current data from operating systems will be evaluated. These systems will use, but are not restricted to, conventional wastewater treatment processes and systems. The concept of an integrated system with energy sources not normally available to municipal wastewater treatment facilities does provide latitude in the evaluation of new and innovative wastewater treatment processes. Additionally, the advantages and disadvantages of fossil fuel (both gaseous and solid) and nuclear powerplants are assessed in terms of the treatment processes selected as regards useful by-products or reducing particular problems associated with specific processes. Water quality requirements for all types of recreational use of the pond are not considered feasible and thus do not serve as constraints.

A summary will be prepared delineating further research required.

## POND

The common point in an integrated wastewater treatment-cooling water system is the pond. The pond's dual function places constraints on each purpose. In this section the physical characteristics of the pond are estimated based on the cooling system requirements. Subsequently, the wastewater assimilative capacity of the pond is estimated in terms of BOD removal, growth of microorganisms, nutrient removal, coliform removal, sludge buildup, and increase in salt concentration.

### Requirements for the Cooling Water System

The engineering design of a cooling pond is not part of the scope of this investigation, but approximate surface area requirements are necessary to provide estimates of the wastewater assimilative capacity of the pond. Approximate evaporative losses also were determined to define the minimum required wastewater flow. Based on the topography of north-central Texas, an average depth of 15 feet was assumed. The physical characteristics are shown in Table 2. (The associated computations are presented in Appendix A.)

The lowest average monthly water temperature in the winter is expected to be 15°C, approximately 5°C above normal. In summer, the highest pond temperature is expected to be 35°C as compared to a normal average monthly temperature of 29.5°C. Hence, the heat

TABLE 2

PHYSICAL CHARACTERISTICS OF COOLING PONDS

Powerplant Capacity (MW)	Required Surface Area (acre-feet)	Average Depth (feet)	Volume (acre-feet)	Daily Evaporative Losses (acre-feet)	Minimum Required Daily flow (MGD)
400	560	15	8,400	12	5
1000	1400	15	21,000	55	20

load is expected to be beneficial in terms of the pond's wastewater assimilative capacity during the winter but perhaps detrimental in summer. Growth of algae reaches a maximum at 25°C to 30°C and as the temperature exceeds 30°C the algal population may decrease (4).

Little quantitative data were found in the literature regarding use of wastewater for cooling purposes. Lake Brauning is a 1350 acre closed system cooling pond which uses the San Antonio River as make-up water. The majority of the San Antonio River water is composed of wastewater effluent from an activated sludge treatment system. The ratio of effluent to river water is frequently as high as 10:1 at this point. The make-up water is removed from the River approximately 12 miles downstream from the treatment plant. No significant operating problems have occurred after eight years operation (5). Lake Arlington is a 2275 acre system cooling pond which receives drainage from a number of streams flowing through urban areas. Unknown quantities of treated municipal and industrial wastewater effluents entered these streams until recent development of a regional wastewater treatment facility. One spring a massive algal "bloom" of predominantly blue-green algae occurred for approximately a three month period. No change in the intermittent chlorination of condenser waters was made throughout this period. Subsequent cleaning of the condensers showed no greater fouling than normal (6).

Secondary effluents have been utilized directly for cooling purposes, but usually in conjunction with cooling towers. In the majority of cases the secondary effluent is subjected to additional treatment. The presence of ammonia, nitrogen and phosphorus have caused problems with corrosion and scale deposition (7,8). Treatment processes for ammonia and phosphorus removal as well as the addition of bacteriacides and pH control have been used to control

these problems. However, some installations in Europe and South Africa use secondary effluent as a source of cooling water without additional treatment for make-up water in cooling tower systems (7).

The report of the Committee on Water Quality Criteria has made available some recommendations for cooling waters (20). It appears that a wide range in water quality is acceptable for heat exchange cooling. Table 3 is abstracted from the report and indicates that most water quality parameters should be met by the pond contents. However, important parameters such as ammonia, phosphate, and pH are either not specified or may not be met. Also, suspended solids limitations are probably applicable to inorganic particulates rather than algal cells. It is anticipated that all water quality problems can be overcome by proper condenser materials selection and cleaning methods.

Based on the data in Table 3, it is assumed that wastewater with water quality characteristics similar to effluents from activated sludge, trickling filtration or other secondary treatment effluents would be acceptable for condenser systems.

#### Wastewater Assimilative Capacity

The purpose of this section is to evaluate the wastewater assimilative capacity of the ponds described in Table 2. Little information is available on the effect of wastewater on cooling ponds or even Texas reservoirs. Hence, to accomplish this objective, a detailed literature review was conducted on waste stabilization ponds. (See Appendix B.) Subsequently, the wastewater assimilative capacity was estimated on the basis of this evaluation of the waste stabilization pond literature tempered by experience and literature available on Texas cooling ponds and reservoirs and other lakes and reservoirs receiving substantial wastewater effluent flows. The latter is required because of the significant differences between a typical waste stabilization pond and the proposed cooling ponds in terms of surface area, depth, recirculation ratio and mixing characteristics.

#### BOD Removal

The poorest conditions for BOD removal by the pond microorganisms would be during the cold temperature period. At this time cooling ponds and reservoirs in Texas have no temperature or density gradient with

TABLE 3

QUALITY REQUIREMENTS OF WATER AT POINT OF USE  
FOR COOLING IN HEAT EXCHANGES

Characteristic	Once through		Make-up for Recirculation	
	Fresh	Brackish(1)	Fresh	Brackish(1)
Silica (SiO <sub>2</sub> )	50	25	50	25
Aluminum (Al)	(2)	(2)	0.1	0.1
Iron (Fe)	(2)	(2)	0.5	0.5
Manganese (Mn)	(2)	(2)	0.5	0.02
Calcium (Ca)	200	420	50	420
Magnesium (Mg)	(2)	(2)	(2)	(2)
Ammonia (NH <sub>4</sub> )	(2)	(2)	(2)	(2)
Bicarbonate (HCO <sub>3</sub> )	600	140	24	140
Sulfate (SO <sub>4</sub> )	680	2,700	200	2,700
Chloride (Cl)	600	19,000	500	19,000
Dissolved solids	1,000	35,000	500	35,000
Copper (Cu)	(2)	(2)	(2)	(2)
Zinc (Zn)	(2)	(2)	(2)	(2)
Hardness (CaCO <sub>3</sub> )	850	6,250	130	6,250
Free mineral acidity (CaCO <sub>3</sub> )	(3)	(3)	(3)	(3)
Alkalinity (CaCO <sub>3</sub> )	500	115	20	115
pH, units	5.0-8.3	6.0-8.3	(2)	(2)
Color, units	(2)	(2)	(2)	(2)
Organics:				
Methylene blue active substances	(2)	(2)	1	1
Carbon tetrachloride extract	(4)	(4)	1	2
Chemical oxygen demand (O <sub>2</sub> )	75	75	75	75
Dissolved oxygen (O <sub>2</sub> )	(2)	(2)	(2)	(2)
Temperature, F	(2)	(2)	(2)	(2)
Suspended Solids	5,000	2,500	100	100

- (1) Brackish water-dissolved solids more than 1,000 mg/l by definition 1963 census of manufacturers.
- (2) Accepted as received (if meeting total solids or other limiting values); has never been a problem at concentrations encountered.
- (3) Zero, not detectable by test.
- (4) No floating oil.

depth and hence water quality characteristics are similar at all depths. Thus, to estimate the BOD removal, the cooling pond is assumed to be a single completely mixed reactor. The BOD removal parameters reported in the literature (9,10, 11) for waste stabilization ponds were utilized. Passage of the microorganisms through the condenser system subject to some intermittent chlorination is assumed to have a negligible effect on the wastewater degradation process. The results are presented in Table 4 to show the effect of detention time and the expected temperature increase on the efficiency of the biological processes to stabilize the wastewater. Detention times are based on the influent flow less estimated evaporative losses. For the 560 acre pond at the expected temperature of 15°C the wastewater flow would be approximately 30 MGD before the present BOD<sub>5</sub> effluent standard of 20 mg/l was exceeded. For the 1400 acre pond for the same conditions, the flow would be somewhat less than 100 MGD before exceeding the standard.

It is quite likely that the cooling pond would not behave as a single, ideal, completely mixed reactor. Short-circuiting could occur if the raw wastewater is mixed with the condenser return flow and floats across the top of the cooling pond. Conversely, the long narrow shape of usual cooling pond sites can impart some plug-flow character to the pond. Nevertheless, because the raw wastewater will be mixed with condenser cooling water and diluted at least ten-fold before entering the pond, the single completely mixed reactor model is probably most applicable.

The BOD removal parameters utilized in the calculations were obtained from literature on facultative waste stabilization ponds, which seldom have depths beyond six feet. Calculations are presented in Table 5 based on the assumption that only the top six feet of the pond are effective. Note by comparing Tables 4 and 5 that the detention time is drastically reduced. Under such circumstances, between a 15 to 20 MGD wastewater flow through the 560 acre pond and less than 50 MGD flow through the 1400 acre pond would produce an unacceptable soluble BOD<sub>5</sub> in the effluent. These estimates are probably conservative because the aerobic zone of the cooling pond will most likely be deeper than six feet. However, if the cooling pond were to be formed by excavation, a depth of at least six feet would be specified.

Although systems of ponds in series with long detention times have reported effluent soluble BOD<sub>5</sub> values less than 10 mg/l (12), experience indicates that 10 mg/l would be the lowest soluble BOD<sub>5</sub> to be obtained on a regular basis in the cooling pond effluent.

TABLE 4

ESTIMATED EFFLUENT SOLUBLE BOD<sub>5</sub> DURING THE WINTER FROM THE COMPLETELY MIXED POND RECEIVING VARIOUS WASTEWATER FLOWS WITH AN UNFILTERED BOD<sub>ultimate</sub> of 300 mg/l

Pond Size (acres)	Wastewater Flow (MGD)	Detention Time (Days)	Effluent Soluble BOD <sub>5</sub> (mg/l)	
			10°C (K=0.078)	15°C (K=0.12)
560	5	-	-	-
	10	548	7	5
	15	274	13	9
	20	183	20	13
	30	110	31	21
1400	20	-	-	-
	30	668	6	4
	40	343	11	7
	50	228	16	11
	100	86	40	27

TABLE 5

ESTIMATED EFFLUENT SOLUBLE BOD<sub>5</sub> DURING THE WINTER FOR A COOLING POND WITH ONLY THE TOP 6 FEET EFFECTIVE IN WASTEWATER STABILIZATION (Other Conditions the same as Table 4)

Pond Size (acres)	Wastewater Flow (MGD)	Detention Time* (Days)	Effluent Soluble BOD <sub>5</sub> (mg/l)	
			10°C (K=0.078)	15°C (K=0.12)
560	5	-	-	-
	10	218	17	11
	15	109	32	21
	20	73	45	31
1400	20	-	-	-
	30	273	13	9
	40	137	26	17
	50	91	38	25

\* Volumes of ponds are assumed to be 3360 and 8400 acre-feet, respectively, based on effective depth of 6 feet.



TABLE 6

## AREAL ORGANIC LOADING RATES FOR THE TWO PROPOSED PONDS

Pond Size (acres)	Wastewater Flow (MGD)	Areal Organic Loading Rate (lb BOD <sub>u</sub> /a/d)	
		Raw	Subsequent to Plain Sedimentation *
560	5	22.5	13.5
	10	45.0	27.0
	15	67.5	40.5
	20	90.0	54.0
1400	20	36.0	21.5
	30	54.0	32.5
	40	72.0	43.5
	50	90.0	54.5

\* Sedimentation assumed to remove 40 percent of organic load.

The literature evaluation for facultative waste stabilization ponds indicates, for the climatic conditions of north-central Texas, that ponds with an areal organic loading greater than 50 lbs BOD<sub>ultimate</sub>/acre/day would cause summer time odors and other nuisance conditions. A loading of 50 lbs BOD/day/acre is a conservative value in that the usually-specified range is 50 to 150 lbs BOD/day/acre for warm climates (39,33). The areal organic loading rate for various wastewater flows into the two proposed ponds is presented in Table 6. In order to meet the constraint for nuisance control, the two proposed ponds would be restricted to maximum raw wastewater flows of approximately 10 and 30 MGD, respectively. A plain sedimentation process can remove 60 to 70 percent of the suspended solids in typical municipal sewage, thereby reducing the BOD by 35 to 45 percent. Estimates are presented in Table 6 for the areal organic loading rate for various raw wastewater flows following plain sedimentation. The maximum wastewater flows for such conditions are approximately 20 MGD and 50 MGD for the 560 and 1400 acre ponds, respectively.

### Growth of Microorganisms

The purpose of the microorganisms in the pond is to assimilate the wastewater. However, if a large quantity of the microorganisms leave the pond, there probably would be some detrimental affect on the condenser system as well as on the receiving water body.

The concentration of suspended solids in the effluent of facultative waste stabilization ponds receiving raw wastewater varied from 20 to 300 mg/l with the lower value associated with long detention times and/or low areal organic loading rates (12, 13). For similar ponds receiving secondary wastewater effluent, effluent suspended solids vary from 20 to 50 mg/l (14). Lake Brauning utilizes the San Antonio River which is composed primarily of secondary treated effluent as make-up water. The lake has a suspended solids concentration in its surface water varying from less than 10 to 50 mg/l. Lake Mendota, a eutrophic lake in south central Wisconsin, had phytoplankton counts up to 12 million per liter in 1965. Nine of the monthly samples were less than 50 mg/l with the highest sample being approximately 180 mg/l (15).

In order to avoid eutrophic conditions in lakes, Vollenweider (16) suggested nitrogen and phosphorus loadings of less than 1.0 and 0.07 g/m<sup>2</sup>-year, respectively, in a lake with an average depth of 5 meters. He also indicated that an approximate doubling of the loading would be the other extreme of the transition zone from oligotrophic to eutrophic conditions. These permissible values were translated into a wastewater loading on the proposed pond sizes in Table 7. Comparison of the permissible values in Table 7 with the actual nutrient loading from the wastewater flows in Table 8 showed that even exceptional pretreatment processes for the removal of nitrogen and phosphorus would not be feasible even for a 5 MGD wastewater flow.

Hence, the proposed cooling ponds are expected to have the same characteristics as highly eutrophic lakes. The data obtained from the literature on the suspended solids concentration in effluents from waste stabilization ponds appears appropriate for use.

TABLE 7  
 PERMISSIBLE NUTRIENT LOADING TO AVOID LAKE EUTROPHIC  
 CONDITIONS

Pond Size (acres)	Nutrient Loading (lb/day)	
	Nitrogen	Phosphorus
560	14	1
1400	34	2.5

TABLE 8  
 WASTEWATER NUTRIENT LOADING

Wastewater Flow (MGD)	Raw (lb/day)		Secondary (lb/day)	
	N	P	N	P
5	1460	625	1040	417
20	5840	2500	4170	1665

#### Introduction of Fish Culture

A reduction in the net production of microorganisms may be possible through the introduction of fish into the cooling pond. Fish can be stocked in the ponds to utilize many of the organic nutrients in the food chain through consumption of the plant organisms and also to minimize any mosquito problems which could develop. Although this idea is not proposed for study in the research program, its benefits may be significant. When and if a prototype system is constructed, studies could be more easily carried out.

There are several examples of successful fish cultures within lakes or ponds receiving sewage effluent. In Texas, Lake Brauning maintains a significant crop of game fish which are harvested by anyone wishing to fish for them (5). Elsewhere, fish ponds in Europe and Asia have utilized primary treated sewage for some time for pond fertilization. In Munich, Germany, carp yield has been as high as 500 lb/acre for a seven-month growing season. Game fishing in the proposed ponds, where feasible and publicly acceptable, could offset the recreation benefits lost from the exclusion of body-contact recreation and increase good will for the power company or public utility operating such a combined facility.

#### Nitrogen and Phosphorus Removal

The removal of nitrogen or phosphorus from the pond influent can be defined with certainty only in relation to that associated with effluent suspended solids, i.e. biological incorporation and subsequent physical removal of the microorganisms from the wastewater stream.

Vollenweider (16) in his review reported that nitrogen retention varied from 15 percent in Upper Lake Constance to 89 percent in Lake Tahoe. Phosphorus retention varied from 25 percent in Lake Zurich to 93 percent in Lake Tahoe. Phosphorus compounds retained in the lakes were usually found in the sediments in lakes with aerobic hypolimnions. For anaerobic conditions, phosphorus concentrations increase in the overlying waters. This is only partly true for nitrogen, a significant fraction of which is eliminated from the lake by denitrification processes. Vollenweider (16) estimates that the nitrogen removed by denitrification in five Swiss lakes ranges from 45 to 81 percent of that retained. However, these lakes were not receiving the organic loading which the proposed cooling ponds will.

Nitrogen removals of over 50 percent have been observed in waste stabilization ponds with long detention times (>100 days) (12, 17), but no significant removal was observed in a stabilization pond with a detention time of 20 days (18). Nitrogen leaving these ponds was primarily organic and ammonia with very little in the

nitrate form. Three mechanisms have been postulated but not substantiated; deposition in the sediments, denitrification, and volatilization of ammonia at high pH (19).

Oswald (55, 84) reported on exceptionally high nitrogen removals from a deep, facultative pond at St. Helena, California. Through some unknown mechanism they postulated that 60 percent of the influent nitrogen was converted to nitrogen gas. The only unusual physical characteristic of the pond was its 10-foot depth. With at least a 15-foot depth over much of the envisioned cooling pond, the integrated system may also achieve these nitrogen removals in the pond. This would have great economic significance.

The fate of phosphorus in waste stabilization ponds also is not known. Field studies indicate that in some cases phosphorus removal is significantly greater than that which could be associated with biological removal. In the few cases where data are available phosphorus removal by precipitation occurs in ponds with long detention times and low areal organic loadings.

Because the other nitrogen and phosphorus removal mechanisms often do not occur and are not quantitatively predictable, it is assumed, conservatively for the purpose of this study, that the only nitrogen and phosphorus removal is through wasting of biological solids in the effluent. Assuming that the average suspended solids concentration of the biological growth for the pond receiving raw wastewater is 100 mg/l, the nitrogen and phosphorus removal should be approximately 10 mg/l and 1.5 mg/l, respectively. Likewise, for a wastewater stabilization pond receiving secondary effluent, an effluent suspended solids is 40 mg/l and the nitrogen and phosphorus removals should be approximately 4 and 0.6 mg/l, respectively.

#### Coliform Dieoff

The Report of the Committee on Water Quality Criteria (20) recommended for secondary contact recreational use (not to include swimming) an average not exceeding 2000 fecal coliform per 100 ml except in specified mixing zones adjacent to outfalls. For primary contact (swimming) a log mean of 200 per 100 ml was recommended.

Initial coliform levels for domestic wastewater are reported to vary between  $10^6$  and  $10^8$  per ml (18). Thus, reduction of coliforms to meet the recommended standards represents a significant removal requirement imposed on the overall system.

The cooling pond has been assumed to approach a completely mixed system. Marais and Shaw (10) utilized the following relationship:

$$N_t = \frac{N_o}{Kt + 1}$$

where

- $N_o$  = number coliforms present initially
- $N_t$  = number coliforms present at time t
- t = detention time (days)
- K = reaction rate coefficient,  $\text{day}^{-1}$

Applying this relationship for  $10^7$  coliforms per 100 ml and flows of 15 and 45 MGD in the 560 and 1400 acre ponds, respectively, and assuming a reaction rate coefficient (K) of 2 (9), the effluent coliform level is approximately  $1.8 \times 10^4$  per 100 ml. A 10 to 1 dilution factor with a receiving water will be required to meet the recommended standards for secondary contact and a 100 to 1 dilution for primary contact.

Lake Victor Brauning was filled initially with San Antonio River water over a two year period (from 1963 to 1965) (6). During this period the coliform levels in the river, which were approximately 50 to 60 percent secondary sewage effluent, varied from  $10^4$  to  $10^5$  per 100 ml. The coliform levels within the lake varied from a low of less than  $2 \times 10^2$  to above  $10^4$  per 100 ml.

After 1965, intermittent pumping of San Antonio River water was done to maintain lake level and, therefore, was not done on a regular schedule. During the three year period 1965 to 1968, coliform counts varied. The average MPN exceeded the primary contact standards but did fall within secondary contact standards most of the time. By 1969 coliform counts had decreased to meet primary contact standards.

The experience at Lake Victor Brauning did not show a significant dieoff of coliforms during the filling period, nor for some time thereafter with water containing less coliforms than raw wastewater.

It would be reasonable to assume a similar situation would exist in the proposed cooling pond with a continual inflow of wastewater. Therefore, effluent from the cooling pond will most probably require additional treatment to meet standards, e.g. chlorination.

### Sludge Buildup

The increase in sediment in the cooling pond is a function of the influent suspended solids and the quantity of biological growth which does not leave the pond through the effluent. In the case of raw wastewater, the high input of suspended solids will result in an increase in the sediment thickness near the input areas. If plain sedimentation prior to the pond is utilized to remove some of the suspended BOD to avoid nuisance conditions, the pond probably would receive 60 instead of 200 mg/l. The problem would be further alleviated with secondary effluent. The sludge resulting from the settling of the microorganisms should be somewhat evenly distributed throughout the pond.

The literature review shows that sludge removal from waste stabilization ponds was an infrequent requirement. Although an equilibrium between the rate of sludge deposition and decomposition has been reported (21), the results of some field studies are not conclusive. Moreover, a correlation between the rate of sludge buildup and the depth of the waste stabilization pond is lacking. A buildup of six inches of sludge after 10 years operation of a 2 foot deep waste stabilization pond is significant, but not the case for a cooling pond with an average depth of 15 feet.

Table 9 presents the calculated rate of sludge buildup in the two proposed ponds for expected minimum and maximum flows under severe conditions, i.e. raw wastewater as influent. Studies have shown that one pound of soluble BOD produces one pound of sludge (15). It was also assumed that 100% of the influent suspended solids settled in the pond and that 60 percent of the influent  $BOD_{ultimate}$  was soluble and converted to biomass. The sludge buildup during the year is based on experiences with sludge decomposition in a conventional anaerobic digester at an assumed average annual temperature of 25°C in the cooling pond (22). Partial inhibition of methane fermentation will increase the rate of sludge accumulation in the cooling pond.

TABLE 9

SLUDGE BUILDUP IN STABILIZATION PONDS  
 Raw Wastewater: SS = 200 mg/l, BOD<sub>u</sub> = 300 mg/l

Surface Area (acres)	Flow (MGD)	Sludge (lb/day x 10 <sup>3</sup> )		Volume at 2% (ft <sup>3</sup> /day x 10 <sup>4</sup> )		Required Volume, (acre-ft/day)		Volume for 90 days Retention (inches)		
		SS(1)	Biomass Growth(2)	SS(1)	Biomass Growth (2)	SS(1)	Biomass Growth(2)	SS(1)	Biomass Growth(2)	
560	5	8.30	5.25	0.7	0.4	0.15	0.95	13.9	8.5	0.5
1400	20	33.40	20.95	2.7	1.7	0.61	0.38	54.9	34.2	1.9
1400	20	83.40	20.95	2.7	1.7	0.61	0.38	54.9	34.2	0.8
560	50	83.40	52.20	6.7	4.2	1.53	0.96	137.7	86.4	1.9

(1) Assumes 100% settling of suspended solids (SS).

(2) Assumes 1 lb. of soluble BOD produces 1 lb. of sludge (15).

(3) Assumes total area used for sludge deposition.



### Salt Concentrations

Considering the 1400 acre pond with a capacity of 21,000 acre-feet, the estimated evaporative loss is about 20,000 acre-feet per year which nearly equals the pond volume. With no flow through and assuming no removal of salts, a rapid buildup of total dissolved solids would occur, doubling in the first year.

In reality, the two-fold increase will not occur because some dissolved solids will be removed by biological growth, precipitation, sorption, and seepage. Rainfall and runoff will also decrease the rate of solids buildup. A closed system having an influent but no overflow, Lake Victor Brauning (5), has shown only a slow increase in dissolved solids concentration over its ten-years operation. Current trends indicate that some form of blowdown in excess of the natural processes (whatever they may be) will be required in a few years.

Although removal mechanisms for dissolved solids will occur within the pond, a quantitative assessment of the magnitude is not possible. The minimum flow required to maintain the total dissolved solids concentration in the pond at no more than twice that of the influent would require a flow twice the evaporative loss. For the 560 and 1400 acre ponds, this would be 10 and 40 MGD, respectively.

Previously, maximum flows for other water quality considerations were 20 and 50 MGD for the ponds. Considering the minimum flows described above, the allowable, average flow to these ponds would be 15 and 45 MGD, respectively. These flow rates will be used in subsequent sections of this report to describe process costs.

## PRETREATMENT

### Raw Wastewater Feed

To control pond odors in the summer, the maximum areal  $BOD_u$  loading of the pond has been chosen to be 50 lbs  $BOD_u$ /acre/day as previously discussed. Considering this factor only, a maximum 12 MGD and 28 MGD could be fed directly to the 560 acres and 1400 acre ponds, respectively. Flows in excess of these values will require some form of pretreatment for BOD removal (See Appendix C).

Simple primary sedimentation for partial suspended solids removal typically achieves about forty percent reduction in raw wastewater BOD.

The addition of this process would then allow flows up to 20 and 50 MGD for the two pond sizes. Thus, from a consideration of BOD loading, simple primary sedimentation may be sufficient to allow processing of the maximum expected wastewater flows. An over-riding consideration may be that because of the presence of grease, grit, and other floatables in raw wastewater, conventional primary treatment would be required for aesthetic reasons alone.

Other considerations besides BOD loading and floatables may be significant. The presence and fate of wastewater nitrogen and phosphorus compounds within the pond cannot be predicted accurately. It is recognized that a large percentage of these nutrients can be removed from the wastewater by the pond as previously discussed. Thus, the most economical system would utilize this capability to the fullest. What is unknown, however, is the effect of given nutrient levels within the pond upon condenser performance. Potential problems may arise from excessive biological growth within the condenser, condenser scaling due to high phosphorus levels and condenser corrosion due to high ammonia concentrations.

The conclusions regarding specified pretreatment processes for raw wastewater influents cannot be firm at this time. An optimistic recommendation would indicate that no pretreatment would be necessary for wastewater flows up to 12 and 28 MGD for the two basic pond sizes. Above these flow rates, primary sedimentation alone would allow treatment of 20 and 50 MGD, respectively.

More conservative recommendations would require that either phosphorus or nitrogen removal or nitrogen and phosphorus removal be included. To achieve phosphorus removal, the most likely process would be chemical precipitation-flocculation. This well-known process utilizes inorganic chemicals such as lime, alum, or iron salts to precipitate phosphorus compounds and coagulate organic particulates and colloids. The latter capability increases BOD removal over that of primary sedimentation up to 70 percent (23). Thus, the process could also reduce pond loadings, reduce potential odor problems, and, most likely, lead to a better pond appearance. If lime was the choice of chemical, alkaline fly ash could be available at some power plant sites to supplement the lime dose and reduce costs.

If nitrogen were to be removed from the raw influent, the most inexpensive method would be to combine nitrogen removal with phosphorus removal. In this instance, lime would be used to precipitate phosphorus compounds at about pH 11; then the settled effluent could

be air stripped to release ammonia from the wastewater. The high pH is required so that the inorganic nitrogen compounds will be in the more volatile, ammonia form (24). Recarbonation of stripping tower effluent with CO<sub>2</sub> for pH reduction may be needed depending on pond alkalinity which is an unknown at this time. Nitrogen removal alone is not feasible with raw wastewater.

Figure 1 compares expected performance and cost for combined nitrogen and phosphorus removal with primary sedimentation alone for a flow rate of 45 MGD. Total amortized cost for combined pretreatment is 13.1¢/1000 gallons as compared to 5.8¢/1000 gallons for primary sedimentation alone. If grease, grit, and floatables only were removed, costs could be even further reduced. As previously discussed, the choice is not clear because of the unknown pond response. To answer the question, research must be performed.

#### Secondary Effluent Feed

A pretreatment recommendation for secondary effluent feed must also acknowledge the same unknowns as discussed above. If BOD load applied to the pond were the only criterion, no pretreatment would be required for either of the basic pond sizes within reasonably available wastewater flows. The only cost for pretreatment would be that for pumping. However, again the question arises as to the need for nutrient removal. It does not appear that the 99 percent nutrient removal required to guarantee a non-eutrophic pond is economically feasible. Nevertheless, the considerations relating to condenser problems as indicated above may be valid. Figure 2 shows the costs for combined phosphorus and nitrogen removal along with expected process performance at the 45 MGD flow rate. Costs are similar to those for treatment of raw wastewater because required equipment and chemical dosages will be about the same. Performance is different because of the different character of wastewater. It should be noted that the pretreatment shown in Figure 2 produces an effluent similar to the influent to Lake Brauning, which is a workable closed system.

### POST-TREATMENT

#### Post-Treatment: Cooling Water

The use of secondary effluent in powerplant cooling water systems has been as make-up water for closed systems, i.e. no

FIGURE 1

PRETREATMENT PROCESSES  
(45 MGD FLOW)

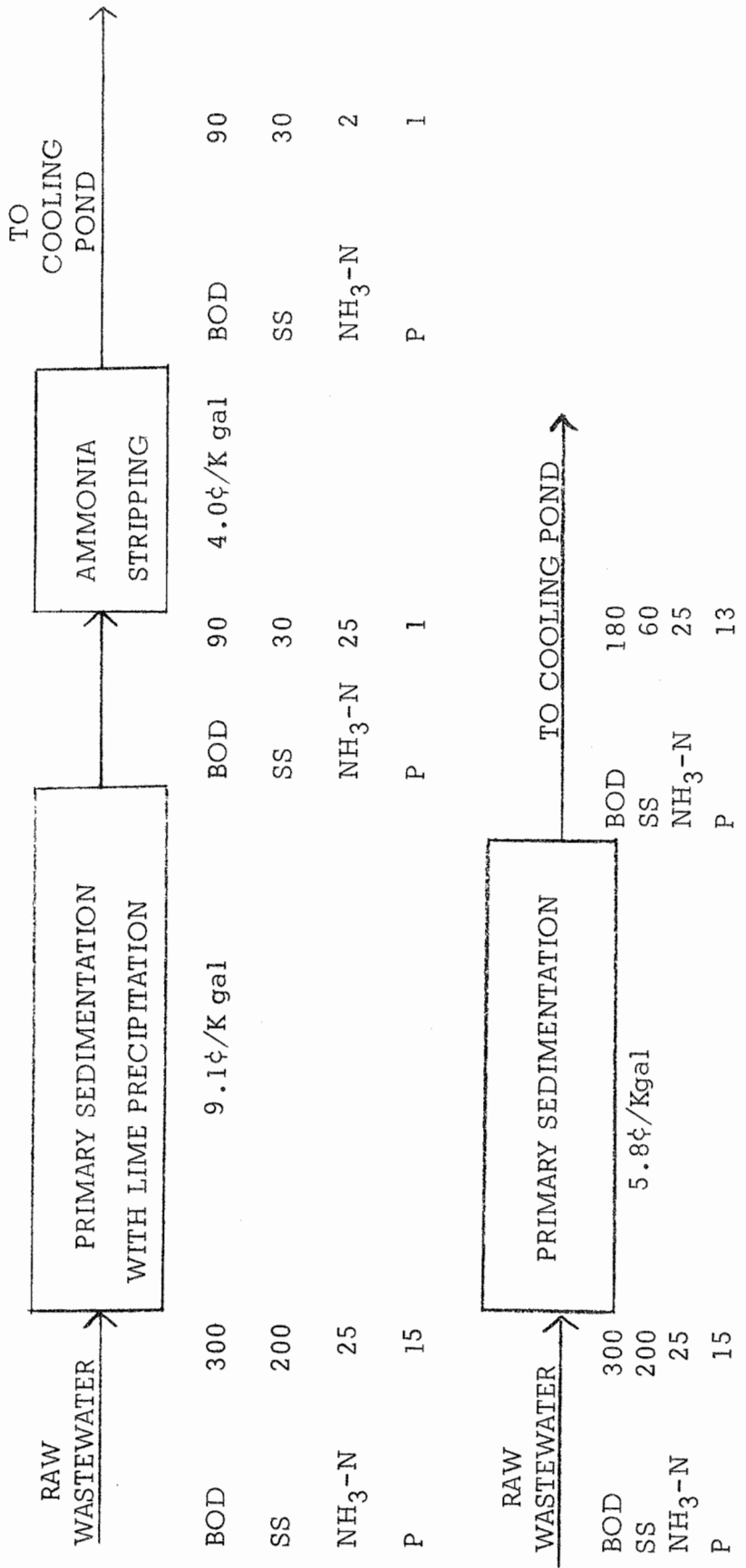
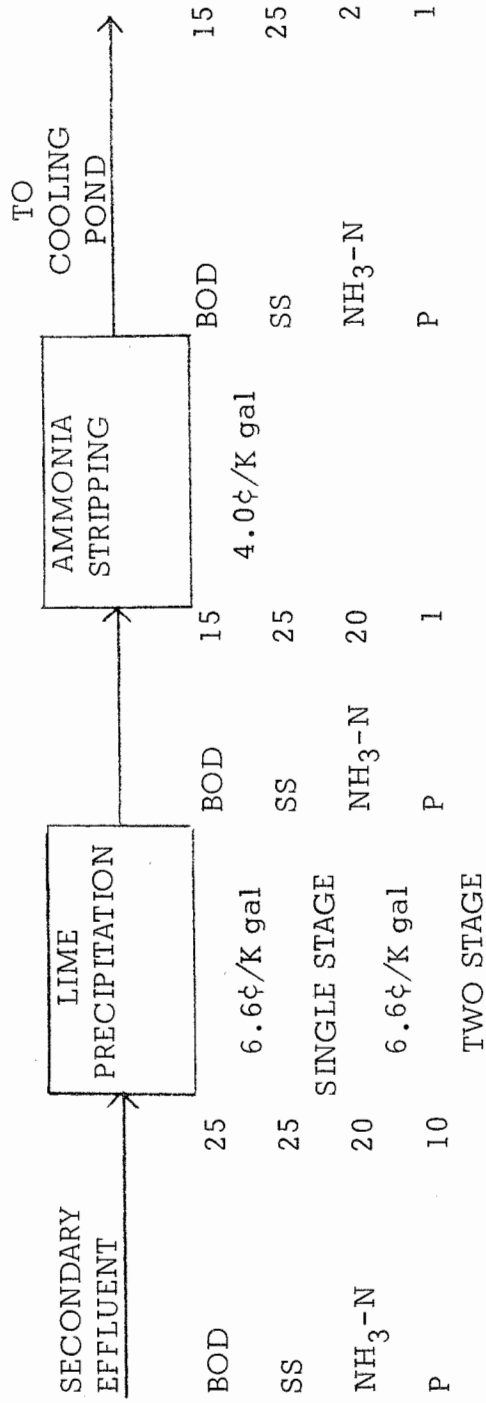


FIGURE 2

PRETREATMENT PROCESSES  
(45 MGD FLOW)



significant system effluent. Thus, any additional treatment beyond secondary is accomplished only on the flow required to sustain system losses. Some problems have occurred that require treatment of the entire system, e.g. biological fouling of condensers, but a closed system minimizes this possibility.

An integrated system is not a closed system (minimum wastewater flows are a special case), and as such, any treatment required to control problems within the condenser system will be a constant requirement. The cooling flow for a 1000 megawatt powerplant is approximately 1350 cfs for conventional fossil fuel. Any treatment process associated with the condenser system would thus be required to accommodate a flow of 875 MGD. The use of nuclear fuel powerplants would have even higher flows reflecting the present reduction in efficiency associated with these systems. It is obvious that a requirement for additional treatment of condenser flow could be prohibitive in cost.

There are some problems that can be expected to occur in all cooling water systems with even minimal nutrient concentrations, in particular biological fouling of condenser surfaces. This can be controlled by intermittent disinfection, usually chlorination. Mechanical methods also can be used such as increased blowdown or injection of scouring devices (6).

Potential problems associated with corrosion and scale deposition or excessive biological fouling must be evaluated with respect to the cooling pond effluent. Antagonistic effects of some problems have been reported. Scale problems may be precluded by biological slime prohibiting hard deposition (8). Selective withdrawal could be used to avoid high concentrations of algae, nitrogen, or phosphorus from the pond. Research in these areas is required for adequate definition of required cooling water quality.

#### Post-Treatment: Effluent

Future effluent standards will require treatment beyond that obtained in the cooling pond. The case in which influent flow and evaporative losses are equal is not feasible for any extended period of time because of salt buildup. Thus, for continuing operation, there will be pond effluent requiring further treatment. Pollutants to be removed include algal cells (suspended solids), soluble organics (BOD), and, in some cases, nitrogen and /or phosphorus.

A specific pond's effluent characteristics will depend upon the initial wastewater characteristics and flow rate, type of pretreatment, temperature, and other secondary variables such as pond configuration, depth, mixing pattern, prevailing winds, location of influent and effluent piping. However, it is possible at this time to predict the unit process most likely capable of treating the range of effluents to an adequate degree.

Removal of suspended algal cells from stabilization pond effluents has been shown to be most readily accomplished by chemical flocculation (see Appendix D). Typical flocculating chemicals are alum, lime, and iron salts along with cationic polyelectrolytes. It is important to note that the same inorganic chemicals are used for phosphorus precipitation. Thus, a most economical system would be one which removed phosphorus from the pond effluent rather than the influent. This would be dependent upon condenser fouling as discussed above.

With the constraint of chemical flocculation being required for effluent treatment, other processing steps follow as indicated in Figure 3. Because of the unknown concentration of ammonia nitrogen in the pond effluent, a removal process such as  $\text{NH}_3$  stripping may or may not be necessary. A possible alternative would be selective ion exchange if  $\text{NH}_3$  concentrations were low. Recarbonation by  $\text{CO}_2$  addition to reduce pH will be necessary when lime is used during the chemical treatment step. If alum were the chemical of choice, no pH adjustment would be needed.

A filtration step is recommended for two reasons. First, suspended solids carryover from the chemical treatment system will occur to some degree. Second, the filter will provide protection against upset conditions in the chemical treatment section when slugs of chemical precipitate escape the unit and could plug the activated carbon beds. The activated carbon beds have been shown to be most efficient in removing soluble, adsorbable organics from secondary effluents (24). Unfortunately the process is expensive as shown in Figure 3.

The requirement for chlorination of the effluent is shown in Figure 3, but this could be eliminated by the use of ozonation. Though long known as an effective bactericide and used mostly in Europe for

water treatment, use of ozone has not been widely practiced in the United States. Within the past decade a renewed interest in this process has emerged and also, the additional use as a wastewater treatment process is undergoing investigation (25). The ability of ozone to oxidize most organic compounds is acknowledged, but efficient methods of applying and maintaining residual concentrations for the required contact time are still in the development stage.

An ozone process would be particularly attractive in the integrated system. The ability to oxidize organics could eliminate the activated carbon process and effluent chlorination. Indeed, future effluent discharge may be prohibited if chlorination is practiced because of the toxicity of chlorinated hydrocarbons. A major cost in ozone processes is electrical energy, but costs may be reduced in an integrated system. Also, the activated carbon process thus eliminated is the highest cost process in the state-of-the-art system. However, detailed investigations on the possible toxicity of ozonated effluents have not been completed.

Effluent salt concentrations for the case shown in Figure 3 will be approximately double that of the influent to the system based upon simplistic considerations. Most likely the dissolved solids will be somewhat less than double that of the influent. The exact level will depend on many interactions as yet to be defined by experimentation. Future discharge requirements may dictate some level of de-ionization and additional processing could be needed.

#### ESTIMATED COSTS AND COMPARISONS

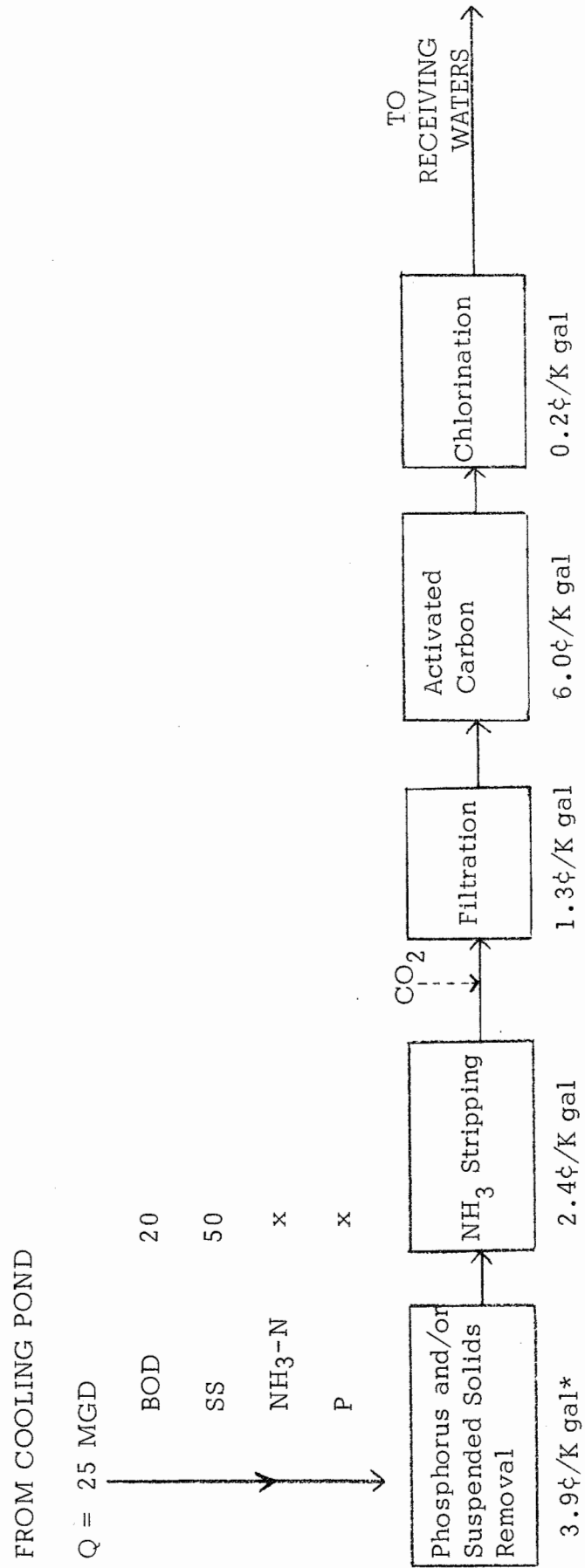
The integrated system has two major advantages which reduce treatment costs below those for a municipality. First, because the ponds must be sized as required by heat dissipation, the conversion of wastewater pollutants by the resident biomass can be assumed to cost nothing. Secondly, because evaporation occurs from the pond surface, the net outflow from the pond is less than the inflow. Thus, effluent treatment equipment can be sized for reduced capacity with savings in capital and operating costs.

Table 10 summarizes the cost comparisons between several possible flow schemes in the integrated system and a municipal



FIGURE 3

POST-TREATMENT BEFORE DISCHARGE TO RECEIVING WATERS  
(45 MGD INFLUENT FLOW, 1000 MW POWERPLANT)



\* Amortized costs based upon 45 MGD influent flow rate.

tertiary treatment plant. Specific details of all case studies are found in Appendix E which includes individual unit costs and flow rates. The municipal plant is assumed to consist of conventional activated sludge treatment followed by tertiary treatment processes. The flow scheme is probably somewhat more costly than some of the new, integrated treatment processes. Unfortunately, reliable full-scale cost data is not available at this time for comparison. For the integrated system, 15 MGD represents the influent of a 400 MW powerplant system. 45 MGD corresponds to the 1000 MW case. Capital (initial investment) cost and total amortized costs (25-year, 6 percent interest) are shown in the table.

In all cases the integrated system was less costly than municipal treatment. To cite one example, a 45 MGD municipal plant would cost about \$29,000,000 for investment and a total amortized cost of 34 cents per 1000 gallons. For a 45 MGD integrated system being fed raw sewage and containing primary pretreatment, costs would be \$17,000,000 for investment and 20 cents per 1000 gallons total amortized cost based upon influent flow rates. A further conclusion drawn from Table 10 is that secondary effluent is much less expensive to process than raw sewage.

Cost figures are based upon the data included in Appendix E. The data represent average costs for many plants and, therefore, any specific plant may have a significantly different cost. Rather than emphasize the absolute magnitude of the individual costs, it may be more realistic to consider the relative differences in costs between cases. For purposes of this report, indicated costs should be sufficient to demonstrate the economic feasibility of the integrated system and provide incentive for further study.

#### PROPOSED RESEARCH

The major objective of the research program is to develop data leading to least-cost design of an integrated powerplant cooling system utilizing municipal wastewater. Review of literature and other operating data previously discussed leaves little doubt that the concept is feasible. Therefore, the research plan becomes one in which an optimum system is to be developed based upon possible variations in wastewater characteristics and available flow rates.

Ponds 2 and 3 are loaded in the first year at maximum loads for raw wastewater and secondary effluent feeds. The rationale behind this priority is that if the maximum load cases are easily controlled and performance is satisfactory, there is no need to study more moderate loadings or the effect of pretreatment for nitrogen and/or phosphorus removal.

4. As Pond 1 will require operation of the phosphorus and nitrogen removal system, it will be closely monitored to provide information concerning required chemical dosages, quantity and composition of waste sludges, influent and effluent parameters of interest. Established art concerning design of the proposed pretreatment processes is highly advanced and, thus, extensive design data for these units need not be developed.

5. Response of condenser fouling as a function of changes in pond biomass will be determined. Because the pond will develop different biomasses as a function of type of influent and its pretreatment, it would be necessary to determine the degree of fouling caused by the range of organisms and their concentrations developed in the ponds. Eventually, upper limits of allowable pond loadings will be established for condenser operation with and without antifouling measures such as intermittent chlorination upstream of the condensers or mechanical scouring.

6. Mass balances on materials entering and leaving the system will be performed on key parameters. Of special importance is the nutrient budget within the system, quantitation of water losses, and the buildup of dissolved solids.

7. Required effluent treatment systems and design data will be determined. Because the pond will produce some net quantity of organisms, the effluent stream must be treated for suspended solids removal. The most likely process is chemical flocculation although other processes will be given cursory examination in the laboratory. It may also be feasible to return the organisms back to the pond. Final treatment for residual soluble BOD removal may best be accomplished by treatment with activated carbon or ozonation. If activated carbon treatment were to be practical, additional chlorine treatment may be necessary to control the coliform concentration. Alternatively, ozonation has the capacity of combining organic destruction with disinfection and is relatively unaffected by the presence of suspended matter.

It is proposed that separate studies on effluent treatment be performed to establish the most economic system. Experimentation on effluent samples will be performed comparing ozonation and activated carbon treatment along with suspended solids removal processes. This experimentation will continue as representative pond effluents become available.

#### Phase 3 (12 months)

If during year 1 the operation of either Pond 2 or 3 indicates that significant improvements in water quality, condenser operation, or pond stability and aesthetics may be achieved by additional pre-treatment or reduced organic load, then year 2 will be devoted to further system optimization based upon benefits derived from nitrogen and/or phosphorus removal by pretreatment. Other research would also continue:

1. Continual updating of the mathematical model based on Pond 1, 2, and 3 operation,
2. Bench-scale effluent treatment studies based on year 2 operation of ponds, and
3. Continuation of condenser fouling studies.

It is proposed that at the end of year 1 operation, a complete summary and data analysis be performed before making decisions on year 2 objectives and scope.

#### Research Budget and Time Schedule

A preliminary estimate of needed research funds follows. Costs are separated for the three phases. Phase 1 estimates are based upon about a six-month period for design, purchasing, construction, and preliminary operations to fill the ponds. During this time, only partial staff will be employed. The following 12 months cover the Phase 2 research period.

At the end of Phase 2, the details of Phase 3 research will be better known. Therefore, cost estimates for Phase 3 must be considered extremely tentative at this time but 12-months duration is anticipated. Depending on where the pilot plant is constructed, a site restoration cost may be required. The research budget includes an estimate for complete restoration which may not be required if future use of the facility is planned.

Proposed Research Budget

PHASE	Phase 1	Phase 2	Phase 3
DURATION	6 months	12 months	12 months
A. Salaries and Wages			
1. Senior Personnel			
a. 33%, Principal Investigator	\$ 3,500	\$ 7,000	\$ 7,000
b. 33%, Faculty Associates	3,500	7,000	7,000
SUB TOTAL	7,000	14,000	14,000
2. Other Personnel (Non-faculty)			
a. 100%, Project Supervisor	6,000	12,000	12,000
b. 50%, Four Graduate Students*	-	17,400	17,400
c. 100%, Lab Technician*	-	7,000	7,000
d. 100%, Operator	4,500	9,000	9,000
e. 100%, Secretary-Bookkeeper	3,000	6,000	6,000
TOTAL SALARIES AND WAGES	\$20,500	\$65,400	\$65,400
B. Staff Benefits (6% S & W)	1,230	3,924	3,924
C. Total Salaries and Wages and Benefits	21,730	69,324	69,324
D. Permanent Equipment			
1. Feed Pretreatment System (10 gpm)	20,000	-	-
2. Feed Storage Tanks	3,000	-	-
3. Condenser and Circulation Systems	8,000	-	-
4. Monitoring and Sampling Equipment	10,000	-	-
5. Lab Equipment: Autoanalyzer, Ozone generator and reactors, Activated carbon column system	10,000	-	-
E. Expendable Equipment and Supplies	3,000	5,000	3,000
F. Travel, Domestic	500	1,500	1,000
G. Publication Costs	-	1,000	1,500
H. Computer Costs	1,000	1,500	1,000
I. Other Costs			
1. Consultants	500	1,500	1,000
2. Communications	200	400	400
3. Pond Excavation, Lining and Piping	15,000	-	-
4. Site Restoration	-	-	15,000
J. Total Direct Costs	\$92,930	\$80,224	\$92,224
K. Indirect Costs, 46% of S & W	9,430	30,084	30,084
L. Total Costs	\$102,360	\$110,308	\$122,308
TOTAL PROJECT COST	<u>\$334,976.00</u>		

\*Personnel not on payroll for the first six months.

## CONCLUSIONS

1. An integrated wastewater treatment and powerplant cooling water system is technically and economically feasible and mutually beneficial if water quality requirements for all types of recreation in the pond are not considered as constraints.
2. A 400 megawatt powerplant in north-central Texas using fossil fuel would require a 560 acre pond and based on the typical topography of the region could receive raw or secondary wastewater from a metropolitan area containing 100,000 to 200,000 people. Likewise, a 1000 megawatt unit would require a 1400 acre pond of the same depth receiving similar municipal wastewaters from 400,000 to 500,000 people.
3. The principal water quality problem constraining both raw and secondary wastewater sources is salt buildup in the pond because of evaporation. Raw wastewater influent rates are also constrained by odor and nuisance problems resulting from high areal organic loading.
4. To meet an effluent water quality of 20 mg/l suspended solids and a log mean of 200 fecal coliforms per 100 ml, suspended solids removal and disinfection must be practiced, no matter what source of wastewater is utilized.
5. Prior to the cooling pond, treatment processes are required to remove grease and floatables as well as suspended solids if raw wastewater is utilized because of probable odor and nuisance conditions in such large water bodies.
6. Nitrogen and phosphorus pretreatment of either wastewater source would not decrease the nutrient loading on the pond below the eutrophic levels reported for natural lakes. However, such pretreatment might be desired to prevent condenser difficulties as regards corrosion, scale and fouling or the degree of unaesthetic quality associated with such large water bodies.
7. To meet probable future effluent standards, nitrogen, phosphorus and carbon removal processes will be required. Effluent salt concentrations may be the overriding constraint.
8. There are no predictive models of waste stabilization ponds which can accurately estimate all effluent water quality parameters. Further, because quantification of mass, energy, and their

transformations are difficult in a six-foot waste stabilization pond, extrapolation of the waste stabilization pond data to deep, circulated cooling ponds is not reliable. Specific research on the proposed system must be performed for more realistic estimates of process requirements and performance.

## APPENDIX A

### PHYSICAL CHARACTERISTICS OF THE COOLING POND

In order to evaluate accurately the capability of the cooling pond as a wastewater treatment process, it is necessary to estimate the surface area of the cooling pond based on effluent temperature requirements and to estimate the minimum wastewater flow to balance evaporation and other losses expected in the north-central region of Texas.

The minimum pond surface area is dictated by the requirements for temperature control. The report of the Committee on Water Quality Criteria (20) recommended that heat should not be added to a stream in excess of the amount that will raise the temperature of the receiving water more than 5°F. For discharge into lakes the temperature of the epilimnion in those areas where important organisms are most likely to be adversely affected should not be raised more than 3°F above levels which existed prior to the addition of artificial heat.

For a 1000 megawatt system the required daily cooling flow is approximately 850 MGD (1350 cfs) based on a 15°F rise across the condensers and 38 percent efficiency (26). The maximum wastewater flow anticipated in this study is 50 MGD. Thus, the maximum cooling pond effluent expected is 50 MGD, assuming no losses, or only 6 percent of the actual cooling flow. Hence, a recirculation system represents a significant decrease in heat loading to the stream when compared to a once-through system.

#### Surface Area Estimates

Thackston and Parker (26) presented a series of contour maps of the United States depicting the effect of geographical location on equilibrium temperatures, heat exchange coefficients, and cooling pond performance. Their data were derived considering a "standard plant" with a capacity of 1000 megawatts, a once-through cooling water flow of 1350 cfs, and a temperature rise across the condensers of 15°F. (This is equivalent to a plant efficiency of 37 to 38 percent.) Based on their data for north-central Texas, the net temperature rise calculated for a 1500 acre pond on July 1 is approximately 4.5°F. A fifty percent increase in pond surface area reduced the net temperature rise to 3.5°F. In January a surface pond area between 1250 and 1500



acres will result in a temperature rise of 7.5°F. (This higher pond temperature probably would be of value to biological systems within the cooling pond during the winter.)

Figure A-1 depicts a relationship between heat added and water-surface temperature noted by Harbeck, et al. (27) at Lake Colorado City. The surface area of Lake Colorado City varied between 1000 and 1200 acres during the study of Harbeck, et al. and is representative of the conditions that would be encountered in the present study, i.e. surface area and the use of recirculation. Extrapolation of this relationship to the heat added by a 1000 megawatt powerplant indicates a temperature rise near 10°F would be expected. However, to establish a temperature rise below 10°F a pond size with a surface area greater than 1200 acres would be appropriate.

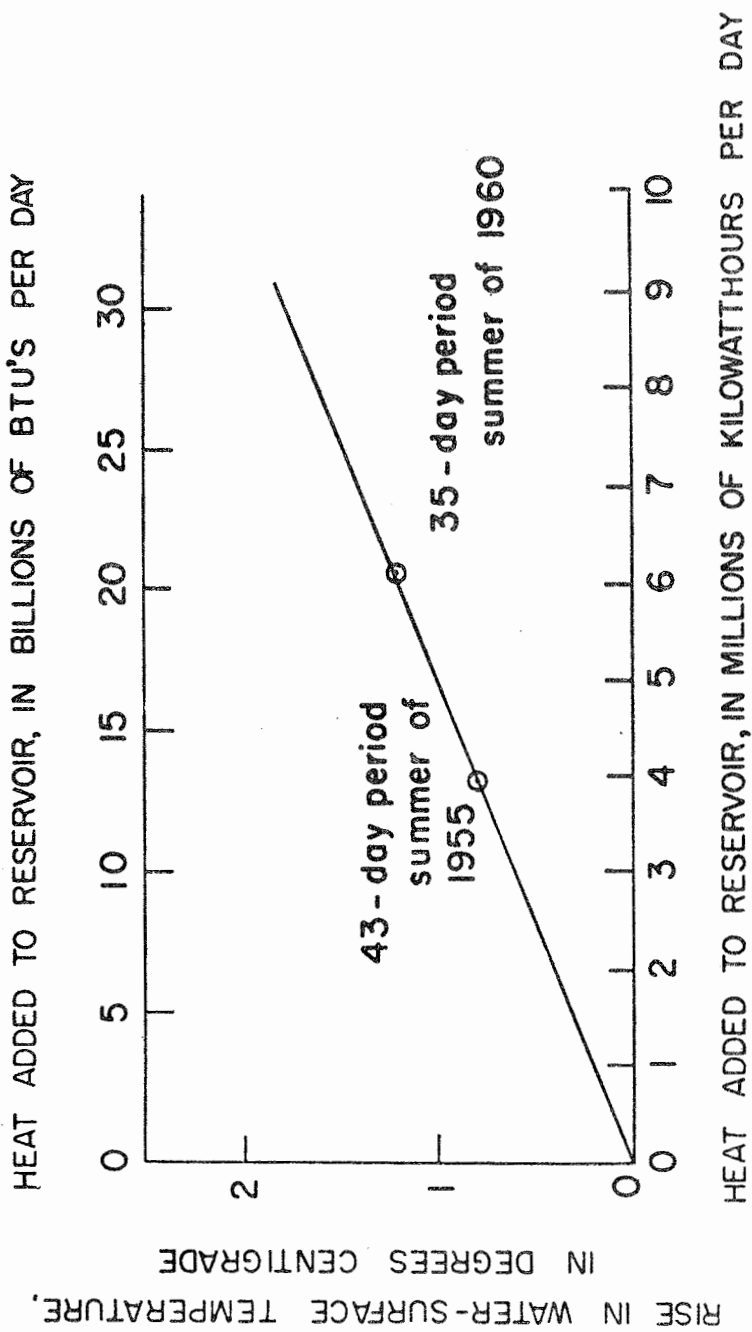
A surface area of 1400 acres will be used during the present study for a 1000 megawatt powerplant. This area has been estimated based on a fossil fuel plant with a 37 to 38 percent efficiency using a recirculation system. Should the powerplant utilize a nuclear fuel source, the efficiency will be reduced to approximately 30-33 percent, and the thermal load on the cooling pond will increase, which would require a larger surface area to maintain the same pond temperature. However, for the purposes of this study the 1400 acre approximation will serve to define the constraints for the biological system within the pond.

The 560 acre estimate for the surface area for the 400 megawatt powerplant was based on the assumption that the scale factor is linear. Though not exact this will serve to define the constraints for biological systems.

#### Minimum Wastewater Flows

The minimum required wastewater flow to sustain the losses in the system was approximated by assuming that only the evaporative loss is significant and that losses from seepage and on site usage are balanced by rainfall and runoff.

Evaporative losses were measured and compared with computed losses by Harbeck, et al. (27) in a field study at Lake Colorado City which is subjected to a thermal load and at a nearby reservoir, Champion



**Figure A-1**  
 Relation Between Heat Added to Reservoir and Water-Surface Temperature  
 Rise for Selected Summer Periods at Lake Colorado City  
 (after Harbeck, et al. 27)

Creek, which was not subjected to a thermal load. During an earlier study these investigators postulated that forced evaporation at Lake Colorado City was directly proportional to the thermal load as follows:

$$E_f = 710 Q_s$$

where

$$\begin{aligned} E_f &= \text{forced evaporation, in acre-feet} \\ Q_s &= \text{thermal load, in billions of kilowatt-hours per year.} \end{aligned}$$

The field results substantiated the above calculation within 5 percent.

The geographical location and physical properties of Lake Colorado City and Champion Creek reservoirs are assumed to be similar to the cooling ponds required for the proposed 400 and 1000 megawatt powerplant (13.5 billion KW-hrs per year at 38 percent efficiency),

$$\begin{aligned} E_f &= 710 \times 13.5 \\ E_f &\approx 10,000 \text{ acre-feet per year.} \end{aligned}$$

The normal evaporative loss without a thermal load was approximately 7800 acre-feet at Lake Colorado City which had a surface area of 1100 acres during this study. The same evaporative rate would result in 9900 acre-feet per year for a pond with a surface area of 1400 acres.

The total evaporative loss will be 19,900 or approximately 20,000 acre-feet per year. This is equivalent to a daily flow of 17 million gallons. As a safety factor the minimum flow will be assumed as 20 MGD.

The 5 MGD estimate for the evaporative loss for the 400 megawatt powerplant was based on the same assumptions used for surface area.

## APPENDIX B

### LITERATURE REVIEW - WASTE STABILIZATION PONDS

The term stabilization pond generally refers to all bodies of water artificially created or employed for the retention of municipal or industrial wastewater until the wastes are rendered stabilized or unobjectionable through biological decomposition and the waters are suitable for disposition either by discharge to receiving water or by way of seepage or evaporation (28). There are three types of waste stabilization ponds:

- 1) anaerobic ponds in which decomposition of organics occurs by fermentation;
- 2) facultative ponds in which stratification causes a predominance of anaerobic reactions in the lower section and of aerobic oxidation in conjunction with photosynthesis in the upper section; and
- 3) high-rate ponds in which photosynthesis is the major source of oxygen for entirely aerobic stabilization of organics.

In the following subsections these ponds are discussed with emphasis on physical characteristics, design criteria and expected effluent quality. Separate sections emphasize the probable nitrogen and phosphorus removal in any type of waste stabilization pond.

#### Anaerobic Waste Stabilization Ponds

The microbial organisms remove the applied BOD primarily in the form of methane, CO<sub>2</sub> and hydrogen in a fashion similar to conventional unheated digestors (12) and anaerobic contact processes (29). Although high BOD<sub>5</sub> removals are obtained under favorable temperature conditions, the effluent suspended solids and BOD<sub>5</sub> are still quite high and do not meet present effluent standards. The anaerobic pond generally is part of a series of at least two ponds and is followed by a facultative pond to produce an acceptable overall effluent (30).

### Depth

Recommended pond depths vary from 3 to 12 feet (12) (14) (29). The depth is chosen to protect the settled sludge undergoing methane fermentation from environmental changes such as dissolved oxygen (30), provide a more compact sludge deposition zone (28), and prevent carry-over of suspended solids in the effluent (14). Oswald (30) also points out that the increased depth reduces the temperature.

### Detention Time

Aguirre and Gloyna (31) recommended that the detention time in anaerobic ponds be kept to a minimum of 5 days. Parker (14) has made similar recommendations. However, Oswald, et al. (12) suggested times of 20 to 30 days. It should be understood that these detention times are based on seasons of the year when temperatures exceed 15°C.

### Organic Loading Rate

The range of organic loading rates is based on two factors:

- 1) a minimum loading to maintain anaerobic conditions, and
- 2) a maximum loading commensurate with aesthetic considerations, e.g. excessive odors or unsightliness acceptable at the site.

As would be expected, the ranges of these loadings vary considerably depending on the geographic location of the pond with maximum values during the warmer times of the year. Oswald (30) considers the minimum loading to maintain anaerobic conditions throughout the pond for summer operation of ponds in California to be approximately 400 lbs BOD/acre/day and in winter conditions possibly 100 lbs BOD/acre/day. Cooper, et al. (29) reported minimum ranges from 220 to 660 lbs BOD/acre/day. Van Eck and Simpson (32) reported that maximum loading rates as high as 2590 lbs BOD<sub>5</sub>/acre/day for domestic wastewaters under summer conditions gave 80 percent BOD removal in South Africa. Parker (14) found 85 percent BOD removal for loading rates of 1800 lbs BOD<sub>5</sub>/acre/day in Australia. However, odors and nuisance problems accompanied these high loading rates. Aguirre and Gloyna reported that loadings up to 900 lbs BOD<sub>U</sub>/acre/day with detention times of 2.3 days and minimum sludge temperatures of 16°C did not cause noticeable odors.

However, a loading of 1600 lbs BOD<sub>u</sub>/acre/day with a detention time of 1.3 days at 28°C did cause noticeable odors (31). Oswald (28) recommends loading rates of 200 to 500 lbs BOD/acre/day to prevent nuisance conditions from odors.

Recirculating effluent from a facultative or aerobic pond back to the anaerobic pond (21) (30) provides dissolved oxygen to dispel odors as well as carry a larger fraction of the BOD load forward to the facultative pond. However, though odors may be eliminated, the loading also is reduced for the anaerobic pond and increased for the following pond (30).

### Temperature

With domestic wastewaters normally 30 to 40 percent BOD<sub>5</sub> removal can be expected from sedimentation alone, and below 15°C this will represent the BOD<sub>5</sub> removal capacity of the pond (33). However, BOD reductions of 40 to 60 percent can be expected for average temperature of 20°C, and reductions exceed 80 percent at temperatures above 25°C. Minimal detention times are required to achieve this removal; Parker (14) reported 65 to 80 percent removals with a 1.2 day detention time during warm conditions and 45 to 60 percent for 5 to 7 days during the cold season in Australia.

### Predictive Models

Assuming an influent and pond temperature of 20°C:

$$L_p = \frac{L_o}{K_n \left( \frac{L_p}{L_n} \right)^n R+1}$$

where

- $L_o$  = influent BOD (mg/l)
- $L_p$  = effluent BOD (mg/l)
- $R$  = detention time for completely mixed system (days)
- $n$  = exponent, to be determined by experimentation  
(for Zambia,  $n = 4.8$ )
- $K_n$  = BOD removal parameter (days<sup>-1</sup>)

An example of the application of the model is presented by Gloyna (33).

### Facultative Waste Stabilization Ponds

For successful operation, a facultative pond requires sufficient depth to maintain aerobic and anaerobic zones. In the aerobic zone bacterial oxidation of organic matter occurs with the major portion of the required oxygen supplied by algae utilizing the CO<sub>2</sub> produced by the bacteria. Part of the influent BOD is converted to biological growth which eventually settles in the pond and undergoes anaerobic decomposition.

Facultative ponds often receive raw wastewater directly and in other cases follow anaerobic ponds. Facultative ponds also serve as polishing ponds for secondary effluents.

Effluent quality from facultative ponds varies according to the previously described circumstances. BOD removals range from 50 to 95 percent based on filtered effluent samples. Unfiltered samples vary in accordance with the effluent suspended solids which range from 20 to 300 mg/l.

#### Depth

A major consideration in selecting the depth for facultative ponds is the requirement to maintain a distinct anaerobic zone. Early facultative ponds were designed with depths from 1.5 to 4.0 feet (34) (35), but difficulties arose with maintaining anaerobic decomposition of the sludge layer (30). At present the recommended depth is 5 to 6 feet (21) (31) (33) (36) (37). Little has been reported about pond depths in excess of the recommended range. Oswald (30) related decreased algal counts with increasing depth. Marais (37) and Hodgson (38) pointed out that, for a given surface area, increased depth will provide greater retention time and additional treatment, but that the rate of treatment will not be increased in direct proportion to the increase in depth. They state that there is little practical advantage in increasing the depth beyond 5 or 6 feet to reduce surface area requirements for BOD removal.

#### Detention Times

Detention times vary considerably with geographical location, climatic conditions and organic loading rate. A detention time of 20 to 45 days is required by most state agencies (39). Parker (14) reported acceptable BOD removal in Australia during summer with a one week retention and a one month retention during the winter.

### Organic Loading Rates

Table B-1 presents the results of a survey of the 50 states of the United States for recommended areal organic loading (39). The recommended rates vary from 16.7 to 50 lbs BOD<sub>5</sub>/acre/day, with the higher loadings used in the southern states.

Oswald (30) has reported ponds in California operating without problems with loadings of 125 lbs BOD<sub>5</sub>/acre/day. Parker, *et al.* (14) suggests loadings of 100 to 60 lbs BOD<sub>5</sub>/acre/day for warm and cold seasons in Australia. Loadings up to 200 lbs BOD<sub>5</sub>/acre/day have produced stabilized effluents from laboratory and pilot ponds operated in southern states (39) (40).

Though a general trend upward in loading rates has occurred, this has not been without problems. As pointed out by Svore (41), that while increased loading rates do not reduce treatment efficiency, they can cause nuisance conditions. During long, hot summer days in southern states the formation of excessive concentrations of blue-green algae lead to odors when their mats decay. This can be controlled but requires the pond operators to break up the mats before they decay.

Other methods can be employed with additional operational requirements. Ponds in South Africa have been operated at loadings of 122 lbs BOD<sub>5</sub>/acre/day when temperatures are moderate and their loading rate has been increased to as high as 250 lbs BOD<sub>5</sub>/acre/day with recirculation of the effluent. Recirculation provides additional oxygen rich water which helps eliminate odors and reduces fluctuations in organic loading (21). It was noted that although recirculation will allow an increase in areal organic loading rates, removal of settled sludge may be required. Recommended recirculation rates were 1:1 with provisions for increasing this to 2:1 when variable and higher loading rates are anticipated.

### Models

Several models have been proposed for facultative waste stabilization pond design (9) (11) (33). These models are based generally on first order BOD removal kinetics and consider either a completely mixed or an intermediate case between plug flow and completely mixed conditions. Organic loading, detention time and temperature are the major variables.



TABLE B-1

QUESTIONNAIRE RESULTS ON ORGANIC LOADING  
AND DETENTION TIME DATA  
(After Carter, et al., 39)

Variable	Value Given in Region		
	North	Central	South
Number of states	18	17	15
Organic Loading (lb BOD <sub>5</sub> /a/d)			
Mean	26	33	44
Range	16.7-40(5)*	17.4-80(1)	30-50(2)
Median	21	33	50
Loading (population/acre)			
Mean	124	189	267
Range	100-200 (7)	100-400(4)	175-300(3)
Median	100	200	295
Detention time (days)			
Mean	117	82	31
Range	30-180(11)	25-180(5)	20-45(9)
Median	125	65	31

\* Number in parenthesis indicates the number of states for which no value was obtained. Note: lb/day/acre x 0.112 = g/day/sq m; pop./acre ÷ 0.405 = pop./ha.

Marais and Shaw (10) proposed a kinetic model based on first-order kinetics for a completely mixed system independent of temperature based on existing pond performance in South Africa. Subsequently Marais (9) incorporated a temperature relationship in the model based on the van't Hoff-Arrhenius theory. The present equation is:

$$S = \frac{S_o}{K_t R + 1}$$

where

- $S_o$  = influent BOD<sub>5</sub> (mg/l)
- $S$  = effluent or pond BOD<sub>5</sub> (mg/l)
- $R$  = detention time (days)
- $K_t$  = first order BOD removal parameter (days<sup>-1</sup>)

Thirumurthi (11) has proposed a kinetic model based on an intermediate case using the Wehner and Wilhelm (42) equation for non-ideal mixing:

$$\frac{S}{S_0} = \frac{4 a e^{1/2d}}{(1+a)^2 e^{a/2d} - (1-a)^2 e^{-a/2d}}$$

where

$$a = \sqrt{1 + 4 Ktd}$$

$$d = \frac{D}{UL} = \frac{Dt}{L^2}$$

t = detention time

K = BOD removal parameter

d = diffusivity constant or dispersion number (dimensionless)

D = axial dispersion coefficient (ft<sup>2</sup>/hr)

U = fluid velocity (ft/hr)

L = characteristic length of travel path of a typical particle

The dispersion number represents the effects of short circuiting, exit and entrance hydraulics, and other mixing characteristics. Values of "d" are zero and infinity, respectively, for plug flow and completely mixed systems. For conventional design waste stabilization ponds, "d" will seldom exceed 1.0 because of low hydraulic loads (11).

Using a laboratory-scale stabilization pond, Thirumurthi found that the BOD removal parameter (K) varied as a function of increased organic concentration (BOD<sub>5</sub>) at a constant hydraulic load. During these studies the areal organic loading rate varied between 16 and 66 lbs BOD/acre/day.

To verify his work, Thirumurthi utilized the Fayette, Mo. lagoon experimental studies (17) on a common wastewater with areal organic loading rates ranging from 20 to 100 lbs BOD/acre/day. While an increase in the BOD<sub>5</sub> concentration at a constant hydraulic load (constant d) decreased the BOD removal parameter (K), the effect of an increased hydraulic load (varying d) at a constant BOD<sub>5</sub> concentration increased K.

A facultative stabilization pond model was developed that included the effect of algal upsets in pond efficiency using small 3 foot deep pilot plant units with 6.8 m<sup>3</sup> volumes (43). This exemplifies the autoflocculation phenomena reported by Oswald, *et al.* (13). The maximum depth of

the experimental ponds was 3.9 feet. Oswald (30) reported that the effects of autoflocculation have been noted many times in excessively shallow ponds, i.e., less than 4 feet. Increased depth alleviates this problem by providing an anaerobic sludge layer in which methane fermentation will occur, thus preventing the release to the pond waters of an organic load associated with the decomposing algae.

Hermann (34) proposed an empirical formulation for facultative stabilization pond design. Using laboratory-scale experiments they derived a basic equation for a design to provide an 80-90 percent removal of BOD<sub>5</sub> in domestic wastewater. Subsequent laboratory experiments (31) (36) plus the results of larger pilot plant studies and over 200 operating ponds (33) resulted in the following equation:

$$V = Q \frac{S_o}{200} t_o \theta^{(T_o - T)}$$

where

- v = volume
- Q = flow
- S<sub>o</sub> = influent BOD<sub>u</sub> (mg/l)
- t<sub>o</sub> = reaction time for corresponding temperature (T<sub>o</sub>)  
for a waste with a BOD<sub>u</sub> of 200 mg/l
- θ = 1.085
- T = temperature (°C)

A depth of 5 feet is assumed to be optimum for soluble waste with an additional foot added for sludge storage if the influent is raw wastewater containing significant suspended solids. As stated by Aguirre and Gloyna (31) a t<sub>o</sub> of 7 days is representative of an intermediate case between plug flow (t<sub>o</sub> = 3.5 days) and a completely mixed case (t<sub>o</sub> = 15 days). The detention time (t<sub>o</sub>) is related to the BOD removal parameter (K) in completely mixed systems at 35°C by the following equation (5).

$$K_{35} = \frac{S_o}{S} - 1 \left( \frac{1}{t_o} \right)$$

The ratio S<sub>o</sub>/200 relates the effect of organic concentration at other than 200 mg/l on the BOD removal parameter K.

Values for K reported by Marais (9) and Thirumurthi (11) are markedly different than those reported by Duarte in Gloyna's review (33).

### Aerobic Ponds

The basic principle of the aerobic pond requires that algal photosynthesis provide the required oxygen for aerobic oxidation of the organic load. As such, a balance between required oxygen and photosynthetic production of oxygen must be attained. To maximize algal production the pond needs to be shallow.

Aerobic conditions are maintained throughout the pond and this requires some form of additional mixing to expose periodically the settled sludge to the oxygen rich supernatant. But the sludge cannot be kept in suspension as this would decrease light penetration, thereby reducing the photosynthetic efficiency. Oswald recommends loadings of 100 to 300 lbs BOD<sub>5</sub>/acre/day. During the warm season algal yields between 100 and 300 mg/l can be expected for these loadings (13).

However, problems have been encountered with defining an economical solution to algal separation; several approaches, including centrifuges, ion exchange and various added flocculants, have been investigated (44) (45) (46) (47). The disposal of the algal solids also poses a problem if the sale of algae, i.e. protein, is contemplated. Such a practice would require a market and no significant outlets have appeared (44).

### Models

Experimentally, Oswald (28) found that algal cell production is related to efficiency of solar radiation conversion:

$$Y_a = 0.125 F I_L$$

where

$Y_a$  = algal yield (kg algae/ha per day)

$F$  = light conversion efficiency (%) (2 to 9)

$I_L$  = light intensity (cal/cm<sup>2</sup> per day)

Oswald (48) obtained  $F$  from empirical formulas based on BOD and temperature. Likewise, algal oxygen production is related to oxygen production:

$$O = 0.22 F I_L$$

where

$O$  = oxygen production (kg/ha per day).

The ratio between oxygen production and algal cell production is about

1.6 to 1. The highest percentage BOD removal occurs when the ratio of oxygen produced to oxygen required is about 1.6.

### Nitrogen Removal

Three ways in which nitrogen can be removed in a wastewater stabilization pond are:

1. incorporation within the biomass and subsequent removal with the harvested biomass;
2. settling of the biomass and influent suspended solids to the bottom of the pond; and
3. removal in a gaseous form either as  $N_2$  or  $NH_3$ .

### Biological Removal

An estimate of the nitrogen removal possible by metabolic uptake can be shown by a simple calculation. Assuming that the range of effluent suspended solids is 50 to 300 mg/l and that the solids are predominantly algae with a nitrogen content of approximately 10 percent, the quantity of nitrogen removed with the harvested biomass is 5 to 30 mg/l N. If the total nitrogen of the raw wastewater input is 35 mg/l, approximately 15 to 85 percent nitrogen removal is obtained. The higher effluent suspended solids concentrations are typically obtained in high-rate aerobic ponds. Oswald, et al. (49) reported that approximately 67 percent of the total nitrogen input was removed by algae in such ponds. Gates and Borchardt (50) reported 50 percent removals were obtained by harvesting algae grown in experimental ponds of the same type.

### Deposition

Nitrogen can be removed by sedimentation in the pond of the biomass (as well as the influent suspended solids). Once in the sediments the fate of nitrogen is not known. Aerobic and anaerobic decomposition of the settled material will release the organic nitrogen to the aqueous system either as soluble organic nitrogen or ammonia. Both of these forms also can be sorbed by the settling material (51).

### N Removal as a Gas

Nitrification, oxidation of reduced nitrogen compounds forming inorganic  $\text{NO}_2$  or  $\text{NO}_3$ , and subsequent denitrification to  $\text{N}_2$  gas has been considered as a removal mechanism. The occurrence of denitrification is acknowledged for nitrate-containing industrial wastewaters and has been reported (52,53). However, the mechanism of denitrification remains unclear and its magnitude undefined in impoundments (51) as well as in stabilization ponds (54,55).

Another gaseous nitrogen removal mechanism is the volatilization of ammonia from the water surface under alkaline conditions. Stratton (19) reported that significant increases in rates of volatilization occurred as the pH increased from 7 to 9 and that these rates exhibited first order kinetics. In bench-scale units at  $19.6^\circ\text{C}$  at pH 8.5 an initial concentration of 20 mg/l  $\text{NH}_3\text{-N}$  was reduced to less than 2 mg/l in 20 days and at pH 9 twelve days were required to reduce 20 mg/l  $\text{NH}_3\text{-N}$  to less than 1 mg/l. No significant evidence of biological activity was observed in the test. Stratton (19) was not able to verify his results in field tests. Attempts to verify this removal mechanism in field studies were not successful with  $\text{NH}_3\text{-N}$  removals much less than predicted; about 6 percent of the predicted rate was measured. However, a total nitrogen loss of approximately 12 mg/l was noted over a 6-day period from an impounded sample of holding pond waters at the Santee Water Reclamation Plant, Santee, California. The pH of the water was initially 8.2 and rose to 10.2 by the second day.

Oswald (84) reported that without chemical separation of the algae 92 percent nitrogen removal was obtained in a pond system involving a 10 feet deep facultative pond followed by a 3 feet deep aerobic pond. Approximately 60 percent of the nitrogen removal was by conversion to nitrogen gas by some unknown mechanism.

### Field Studies

Table B-2 presents the results of a field study at Fayette, Mo., covering a two year period (56). Nitrogen removals were greater than 140 mg/l in all units, far in excess of that which would be associated with incorporation in the biomass. (No effluent suspended solids data were presented.) However, other nitrogen removal mechanisms were not identified. Effluent nitrogen concentrations were still too high for probable future effluent standards.

TABLE B-2

## FATE OF NITROGEN IN FACULTATIVE STABILIZATION POND

Source	Organic Loading lbs BOD <sub>5</sub> /acre/day	Depth ft.	Det. Time Days	Influent		Effluent		pH
				Organic N mg/1	NH <sub>3</sub> -N mg/1	Organic N mg/1	NH <sub>3</sub> -N mg/1	
(3) Fayette, Mo.	20 (1)	2.5	87	129	31.4	5.6	5.8	(2)
12 mo. average	40	2.5	44	129	31.4	6.7	7.1	
1957-58	60	2.5	29	129	31.4	10.0	8.4	
	80	2.5	22	129	31.4	8.7	8.7	
	100	2.5	17	129	31.4	8.5	8.6	
(1) Average BOD concentration over test period was 274 mg/1.								
(2) pH of ponds varied from 7.2 to 11.5, not averaged.								
(3) Fayette, Mo.	60	2.5	29	122	31	3.5	3.5	(2)
10 mo. average	60	5.0	29	122	31	5.0	4.9	
1958-59	100	2.5	17	122	31	4.8	4.8	
	120	2.5	14	122	31	6.6	6.3	
	2.5 (3)	2.5	17	4.8	4.8	2.4	2.4	
(3) This pond received effluent from 100 lbs BOD/acre/day pond.								
(13) City of Esparto, Calif.	62.5	8	48	10	25	12	8	(2)
April 1965 average	33.6	8	54	12	8	7.5	4.5	
	21.5	4	90	7.5	4.5	1.7	0.3	
City of Esparto, Calif.	(1)	8	(1)	9.0	20.0	8.9	10.8	8.18
March 1966 average		8		8.9	10.8	5.2	7.5	8.22
(1) No loading or detention data, flow increased approximately 10 percent with a BOD of 119 versus 203 for 1965 data.		4		5.2	7.5	5.0	0.48	9.04
(2) No pH data.								
(11) City of Herzliya, Israel	172	2.8	13	76.4	(3)	167(4)	69.8	(3)
(3) No NH <sub>3</sub> -N data.	32	2.8	13	69.8	102	47.1		8.06
(4) Volatile Suspended solids.								

Total nitrogen removal for the Esparto ponds (17) also is in excess of that associated with the effluent suspended solids, indicating a non-biological removal mechanism also was involved. The total nitrogen removal for the Israel ponds (11) cannot be evaluated for lack of ammonia data.

### Phosphorus Removal

Three ways in which phosphorus can be removed in a wastewater stabilization pond are:

1. incorporation within the biomass and subsequent removal with the harvested biomass;
2. settling of the biomass and influent suspended solids to the bottom of the pond; and
3. chemical precipitation.

### Biological Removal

The quantity of phosphorus that can be removed by harvesting of the biomass in the pond effluent can be estimated by a simple calculation. The range of suspended solids present in pond effluents is 50 to 300 mg/l. Assuming the average phosphorus content is 2 percent, the range of phosphorus removal is 1 to 6 mg/l. If the total phosphorus in the raw wastewater is 15 mg/l, approximately 7 to 40 percent phosphorus removal is obtained.

### Deposition

Phosphorus removal by deposition of the biomass and the influent suspended solids cannot easily be estimated. The fraction of organics entering a stabilization pond as BOD, that becomes biomass deposited in the sediments as sludge has been estimated to equal or slightly exceed the influent concentrations (28), i.e. 100 mg of BOD will produce 100 mg of sludge. If 2 percent of the sediments is assumed, 300 mg/l BOD in the raw wastewater will remove 6 mg/l of phosphorus. However, the sludge deposited is subject to decomposition with the eventual release of this incorporated phosphorus to the wastewater, particularly under anaerobic decomposition (16). There is also removal by sorption (57) or ion exchange (58) within the sediments, but this is a function of the particular sediments and cannot be quantitatively defined.



### Chemical Precipitation

Phosphorus removal not associated with biological activity can occur by chemical precipitation. The theoretical solubility of phosphorus in the pH range encountered in natural waters (7 to 9) with Al or Fe is in the  $10^{-5}$  to  $10^{-6}$  M range ( $< 0.3$  mg/l) and with calcium is approximately  $10^{-8}$  to  $10^{-9}$  ( $< 0.3$  mg/l). The actual solubility is a function of other ionic species present (59, 60, and 61). In general most wastewaters contain concentrations of precipitating cations (particularly calcium) to effect significant chemical precipitation, but this rarely occurs in conventional wastewater treatment processes. Several reasons for this have been postulated.

Ferguson and McCarty (59) reported that magnesium may exert an effect on calcium-phosphorus solubility by inducing the formation of  $\beta$  tri-calcium phosphate in lieu of hydroxyapatite causing a significant increase in phosphorus solubility. Some doubt exists as to whether this is more of a long-term kinetic effect than a difference in the solid species formed (60, 62). Other investigations have shown that the carbonate ion retards the precipitation of calcium-phosphorus compounds (63, 64).

Morgan (62) reported that precipitation of phosphorus with calcium was completely inhibited in short-term tests (5 days) by soluble organics at concentrations of carbon and calcium phosphorus representative of wastewater e.g. 100 mg/l COD, 2 mM (80 mg/l) Ca and 0.33 mM (10 mg/l) P at pH 7.6. A COD of 50 mg/l increased the residual phosphorus concentration compared to a system with no organics present from 0.05 mM (1.55 mg/l) to 0.185 mM (5.75 mg/l). Yu and Mark (65) reported that the presence of blue-green algae inhibited the formation of apatite and induced an amorphous form at pH 10.5. The change in residual soluble phosphorus was not reported but the solubility of amorphous forms of a precipitate is higher (66). The soluble phosphorus present in a body of water also may be strongly affected by the detention time (61). The formation of apatites is reported to be a slow process and controversy exists as to whether or not equilibrium is obtained (67).

The pH of the water also exerts a significant effect, particularly above pH 9. The pH range between 7 and 9 exhibits little change in the solubility of apatite because of competitive precipitation of calcium carbonate, but decreases rapidly above pH 9 (59).

Although the chemical precipitation of phosphorus is possible in stabilization ponds, it is influenced by many factors. These

factors can cause significant differences in the soluble phosphorus present in the effluents from different stabilization ponds. The results of several field studies of operating ponds bear this out.

### Field Studies

Table B-3 presents the results of an experimental study at Fayette, Mo. over a two year period (56). The removal of phosphorus from the influent wastewater is significantly greater than that which would be associated with biological incorporation for the average organic content of the wastewater. Thus, in these ponds deposition of phosphorus by chemical precipitation can be assumed to have occurred. However, phosphorus concentrations in the pond effluent are still quite high.

Examination of the Israel data indicates that the only phosphorus removal is by incorporation by the biomass.

The 1966 data for the Esparto lagoons (12) indicate that phosphorus removal in the first pond after approximately 40 days was associated with metabolic uptake (0.7 mg/l P with 54 mg/l SS). Phosphorus removal in the second pond after an additional 40 days, 7 mg/l, far exceeds the metabolic requirements for the effluent suspended solids. The high phosphorus removal in the second pond occurred essentially at the same pH, a difference of only 0.04 pH units.

### Sludge Deposition

The deposition of sludge in stabilization ponds is a function of the influent suspended solids and the settling characteristics of the biomass. As such, in the case of raw wastewater, an input of suspended solids will result in the settling of the solids in or near the input point and will cause a buildup in these immediate areas. The settling biomass will be more evenly distributed throughout the pond.

Anaerobic decomposition should remove most of the organic carbon in the sludge in the gaseous form (30). Initially, sludge accumulation proceeds at a faster rate than decomposition, but once methane fermentation reaches full development an equilibrium is established and no net accumulation occurs (68). Should methane fermentation be inhibited by dissolved oxygen or changes in temperature

TABLE B-3  
FATE OF PHOSPHORUS IN FACULTATIVE STABILIZATION POND

Source	Organic Loading lbs BOD <sub>5</sub> /acre/day	Depth ft.	Det. Time Days	Influent		Effluent			
				Organic P mg/1	PO <sub>4</sub> -P mg/1	Organic P mg/1	PO <sub>4</sub> -P mg/1		
(56) Fayette, Mo.	20	2.5	87	12.2	10	2.2	2.0		
12 mo. average	40	2.5	44	12.2	10	2.5	2.4		
1957-58	60	2.5	29	12.2	10	3.6	3.3		
(1) Average BOD concentration over test period was 274 mg/1.									
(2) pH of ponds varied from 7.2 to 11.5, not averaged.									
(56) Fayette, Mo.	60	2.5	29	13	10.1	1.8	1.5		
10 mo. average	60	5.0	29	13	10.1	2.0	1.6		
1958-59	100	2.5	17	13	10.1	2.3	1.9		
	120	2.5	14	13	10.1	3.1	2.5		
	2.5(3)	2.5	17	2.3	5.7	6.9	0.75		
(3) This pond received effluent from 100 lbs BOD/acre/day pond.									
(4) No data on SS									
(12) City of Esparto, Calif.	62.5	8	48	Total P (2)		SS	Total P (1)	PO <sub>4</sub> -P	pH (3)
April 1965 average	33.6	8	54		22	90	(1)	-	
City of Esparto, Calif.	21.5	4	90		-	70		-	
March 1966 average	(1)	8	(1)	(2)	-	6	(1)	2.94	
(1) No loading or detention data, flow increased approximately 10 percent with a BOD of 119 versus 203 for 1965.		8		(2)	11.2	54	(1)	10.5	8.18
(2) No total P data.		8			10.5	52		3.4	8.22
(3) No pH data.		4			3.4	34		2.6	9.04
(18) City of Herzliya, Israel	172	2.8	13	10.4	(4)	167(5)	7.7	(4)	8.1
	32	2.8	13	7.7		102	8.3		8.06
(4) No PO <sub>4</sub> -P data.									
(5) Volatile suspended solids.									

and pH, sludge decomposition will impose an additional BOD upon the pond from the by-products of partial fermentation (30). Equilibrium under these conditions can take several years (21).

Parker, et al. (14) reported that sludge removal from both anaerobic and aerobic ponds was an infrequent requirement. The Murtcain ponds, average depth 21 in., were inspected after 10 years of operation. The anaerobic pond had a sludge thickness from 3 to 6 inches and over 80 percent of the facultative pond had from 2 to 3 inches. Near the inlet from the anaerobic pond the sludge was from 18 to 24 inches thick due to solids carryover from the anaerobic pond. The overall results indicate a frequency of removal of once in 6 to 10 years for the relatively shallow ponds.

After 2 years of operation, examination of a facultative pond receiving raw wastewater at Fayette, Mo., the sludge buildup was almost a tenth of a foot (56). However, no data on influent suspended solids were reported.

## APPENDIX C

### TREATMENT PROCESSES

#### Sedimentation

The use of sedimentation in wastewater treatment can be grouped into two major areas: removal of untreated settleable solids by gravity, and gravity settling of chemically-coagulated solids.

A major form of treatment given to raw sewage is termed primary treatment and usually includes bar screening, grit removal, grease removal, gravity sedimentation of settleable, putrescible solids, and sludge disposal facilities (69). As a minimum, the integrated system should have some degree of primary treatment when processing raw sewage because of esthetic problems with floatables, large solids and rags, and grit. Removal efficiencies vary widely and are a function of many factors, e.g. type of wastewater, suspended solids content, type of suspended solids, temperature. Burns and Roe, Inc. (23) found that the range from several sources (69,70,71,72 and 73) indicates a removal efficiency of 35 to 75 percent for suspended solids and 25 to 40 percent for BOD.

Costs for this process have been well defined and reasonable estimates can be found from the literature (74). No unusual methods are associated with primary treatment in relation to the integrated wastewater and cooling water system.

Improvement in removal efficiency for suspended solids and BOD can be obtained by the addition of a chemical coagulant. Suspended solids removals can be increased to between 60 and 90 percent. Therefore, as the suspended and colloidal matter in raw sewage contributes to BOD the BOD removal efficiency can increase to between 40 to 60 percent (23). Soluble organics are not removed to any significant extent by chemical coagulation. A further benefit when using inorganic coagulants is the simultaneous precipitation of inorganic phosphates and coagulation of organic phosphorus containing particulates. Total phosphorus removals down to a residual of one to two mg/l of P are readily achieved without filtration (75). Coagulant dosages which produce good phosphorus removal will also result in good suspended solids removal if polymers are used to aid in flocculating fine precipitates (23). The interrelationship of phosphorus removal with coagulants to improve suspended solids removal make the combination of these processes attractive.

The inorganic coagulants most commonly used are alum (Al), lime (Ca), and iron such as ferric chloride, ferric sulfate or ferrous salts. The choice of a coagulant is dependent on the particular wastewater, chemical cost, and the subsequent fate of the treated water.

Aluminum and the Fe compounds react with the alkalinity present in the waters. The pH range for best coagulation with these coagulants ranges from 5.5 to 8.5 for alum, from 3.5 up for ferric iron, and ferrous ion at pH greater than 9. Good removals occur through most of these ranges (23). As the alkalinity of the water is consumed by these coagulants, the resultant pH rarely exceeds 8.0. Thus the use of an ammonia stripping process requiring a much higher pH range, ~pH 11, will require the addition of hydroxide alkalinity, e.g. lime, sodium hydroxide.

Hydrated lime,  $\text{Ca}(\text{OH})_2$ , when added to wastewaters, raises the pH and converts bicarbonate alkalinity to carbonate alkalinity. When carbonate species are formed, the solubility product of  $\text{CaCO}_3$  is exceeded and precipitation commences. Along with  $\text{CaCO}_3$ , when pH is elevated to about 11,  $\text{Mg}(\text{OH})_2$ , calcium phosphates, and other polyvalent metal ions are also precipitated. The system is somewhat more complex at lower pH's where inhibition and selective precipitation can be made to occur (59,60). However, control techniques have not, as yet, been developed to provide reliable performance with rapidly fluctuating wastewater characteristics. As mentioned previously, coagulant dosages providing good phosphorus removal also provide good suspended solids removal, and dosages will be discussed under the subsection dealing with phosphorus removal.

### Phosphorus Removal

Chemical removal of phosphorus is defined as the precipitation of a solid form which can be removed physically from the wastewater stream. This subsection deals with specific processes, i.e., controlled precipitation, coagulation, flocculation and gravity settling; it does not include precipitation of phosphorus that may occur within the pond as a result of natural conditions.

The three major cations associated with chemical precipitation of phosphorus are aluminum, calcium and iron. All form relatively insoluble compounds with phosphorus within particular concentration and pH ranges. In addition, all three function as chemical coagulants;

aluminum and iron form metallic hydroxides, and calcium, added as lime ( $\text{Ca}(\text{OH})_2$ ), reacts with carbonate, and at high pH ( $\sim 11$ ), magnesium hydroxide also will be precipitated to act as an effective coagulant.

Required dosages of aluminum and iron are a function of the amount of phosphorus to be removed, i.e. a stoichiometric requirement. In practice the stoichiometric dosages are not adequate and from 1.5 to 3 times are required to develop clear effluents (24). Also, aluminum and iron remove hydroxyl ions with a resulting decrease in pH. In some instances, if the alkalinity of the wastewater is not sufficient, base must be added to remain within the pH range for coagulation. These ranges were discussed in the subsection on sedimentation.

The lime dose required for phosphorus removal is independent of the amount of phosphorus present in wastewaters of typical ionic character. Rather, it is a function of the alkalinity of the wastewater. This is related to the desire to form a dense  $\text{CaCO}_3$  precipitate which exhibits rapid settling and a clear effluent. To obtain  $\text{Mg}(\text{OH})_2$ , and the effect of this gelatinous precipitate to remove colloidal particles, requires pH 11 or above (75). Burns and Roe (23) present a graphical summary of lime requirements as a function of alkalinity.

Two processes employing lime (calcium) as the precipitating cation have been investigated (75): 1) single stage in which a low pH, approximately 9.5, is used, and 2) two-stage in which a high pH, approximately 11, is achieved in the first reactor followed by a second stage where first-stage effluent is recarbonated with  $\text{CO}_2$  to pH less than 10. The second stage removes excess calcium as calcium carbonate and stabilizes the water if pH is reduced to about 8.5. The choice of process is dependent on alkalinity of the wastewater as well as the required removal of phosphorus. With high alkalinity wastewaters, above 200 mg/l, well settling floc will form at a pH as low as 9.5 (75), but the effluent will contain some colloidal precipitate. Without additional treatment, e.g. filtration, the phosphorus content of the effluent will be greater than the high pH process. The use of single stage phosphorus removal appears to be limited to an alkalinity of 150 mg/l or greater in wastewaters with significant organic particulates. Above 200 mg/l alkalinity, settleability is likely to be satisfactory (75).

Another consideration of single versus two-stage processes is the additional cost of recarbonation and an additional clarifier. At the same time, the high pH of the two-stage process provides the pH range for ammonia stripping. This has been done and found to work well at South Tahoe (76).

Chemical removal of phosphorus from wastewaters can be accomplished either by chemical addition during primary treatment or, in the case of a stabilization pond, by chemical treatment of the effluent. Should removal of suspended solids from the raw wastewater be required, the addition of a coagulant would increase the effectiveness of solids removal and also accomplish a significant removal of phosphorus.

Incremental costs for adding phosphorus removal to primary sedimentation are mostly due to chemical cost. Equipment costs add a relatively small amount to the total, approximately 0.15¢/K gal for alum and Fe Cl<sub>3</sub> and 0.3¢/K gal for lime at flows of 100 MGD (75). As a basis for estimating chemical costs, lime at a dosage of 300 mg/l as CaO with a cost of \$20/ton add approximately 2.8¢/K gal. For alum and FeCl<sub>3</sub> one may assume an influent phosphorus concentration and an 80 percent phosphorus removal with 1.5 molar ratios defining the dosage. With alum at \$60/ton (17.1 percent Al<sub>2</sub>O<sub>3</sub>), iron at \$90/ton (as Fe Cl<sub>3</sub>) and the addition of a polymer at a rate of 0.5 mg/l costing \$1.50/lb the costs would be 3.6¢/K gal for iron and 4.2 ¢/K gal for alum (75).

Although lime has the least chemical cost, the alkalinity of the wastewater will affect the required dosage, i.e. 450 mg/l CaO would be 4.2¢/K gal. Additionally, sludge handling problems would be larger with lime compared to the other chemicals. Some cost reduction may be achieved with the use of alkaline fly ash from lignite power plants. At this time there is no basis to estimate the savings, but the scope of proposed research includes this evaluation.

Chemical treatment of the cooling pond effluent also requires a system including feeding equipment and clarifiers. Removal at this point is the same as tertiary processes associated with advanced wastewater treatment systems (76). Consideration of the following processes as well as peculiarities of the effluent associated with stabilization ponds is necessary. Appendix D will deal with algal removal from the effluent.

The choice of chemical for effluent treatment is restricted normally to alum and lime because of residual iron remaining in the treated water. Alum doses of 50 to 100 mg/l are normally sufficient to achieve phosphorus removal and effluent clarification (75). Again, lime dosage is dependent on alkalinity of the wastewater.

With respect to lime treatment of effluent, recarbonation will be required before discharge to most receiving waters because of high



pH. The presence of high organic concentrations in the sludge from primary sedimentation does not allow recalcination but the lower organic content of pond effluent will permit recalcination. This effects a saving in lime costs as well as reduces the volume of sludge produced to a small fraction and reduces attendant cost of sludge handling. Based on recalcining lime, the costs for a 45 MGD flow is 4.7¢/K gal for single-stage and 6.8¢/K gal for two-stage. The addition of dual media filtration is recommended to insure meeting discharge requirements and will add 1.8¢/K gal to both (75). The estimated cost for phosphorus removal with alum, including filtration, is 7.1 ¢/K gal for a 100 MGD flow (75). Ultimately, the choice of where and how phosphorus removal will be accomplished depends on the performance of the cooling pond as a waste stabilization pond.

Sludge disposal problems associated with phosphorus removal in primary sedimentation can pose problems. In natural gas or nuclear powerplants this will be an added cost, but in coal or lignite powerplants this sludge could be added to the fly ash sludge. A 1000 megawatt plant will produce approximately 1500 tons of fly ash per day and the addition of approximately 200 tons per day from primary sludge will not exert a significant effect. The use of powerplant heat also may be advantageous with respect to recalcination should lime be used for phosphorus removal.

#### Nitrogen Removal-Ammonia Stripping

The inclusion of a nitrogen removal process will be dependent on possible future discharge requirements. Should this requirement exist, the lowest cost process would be air stripping (24) of the wastewater at elevated pH to remove ammonia. The process is described in detail in several reports (27,76). In general, ammonia stripping follows a high pH phosphorus removal process to provide the pH range required, approximately 11. In this way costs are minimized; however, the use of any source of caustic alkalinity that can provide the pH range is all that is required.

The percent reduction of ammonia is a factor that should be considered. Slechta and Culp (76) reported costs of \$13.85/MG for 95 percent removal, but only \$8.80/MG for 80 percent removal, mostly due to lower air stripping requirements. Ammonia stripping during pretreatment could take advantage of a lower percent removal because the pond could remove most of the remaining nitrogen compounds.

The major cost saving in this process in the integrated system would be in reduced electrical power costs which represent close to 50 percent the total cost. The use of stack gases from fossil fuel powerplants could be considered, but the problems encountered with delivering the required gas flow would have to be considered, i.e. ducting, distance, etc. Also, precipitate formation because of high CO<sub>2</sub> concentrations in the stack gases could cause severe operating problems. Moreover, effluent dissolved solids would increase.

Basic limitations to the ammonia stripping process include freezing and poor removal during cold weather, calcium carbonate deposition on the tower packing, and the possible pollution of receiving waters by re-deposition of NH<sub>3</sub> from atmosphere into surrounding water bodies. For the envisioned locations of the integrated system, only the scaling problem will remain which is simply a maintenance problem. The southern climate rarely reaches freezing temperatures for more than a few days a year. Also, powerplant locations containing the integrated system most likely will not be near large oligotrophic water bodies or within large metropolitan cities. If for some reason ammonia stripping is unacceptable for a specific location, alternative processes are ammonia removal by selective ion exchange (76) and biological nitrification-denitrification (76). Unfortunately, these latter processes are several-fold more expensive than ammonia stripping.

### Activated Carbon

Adsorption processes utilizing granular activated carbon in columns or reactors have been well studied in large-scale equipment for the removal of soluble organic materials from wastewaters. Applications in municipal wastewater treatment have been to remove refractory organic materials not removed by biological processes (24), and as a major component in new, complete, physical-chemical treatment processes.

Accurate design requirements can only be based on a knowledge of specific influent wastewater characteristics. As an estimate, however, Culp and Culp indicate a reduction of 55 percent COD is typical for a secondary effluent and can be obtained in about 15 minutes contact time (24). Further reduction in effluent COD will require greater contact times, but this requires increased capital and carbon costs. The same effect occurs if the influent COD concentration is doubled;

thus a balance between pretreatment and required contact time must be decided. Table C-1 presents the effects of pretreatment on required carbon dosage.

Costs for activated carbon treatment are also dependent on the influent wastewater. The South Tahoe plant uses activated carbon to treat a wastewater that has received pretreatment as described in the last column of Table C-1. With this level of pretreatment, the total costs for March, 1968 through February 1969 for a 7.5 MGD plant were 3.61¢/K gal. This cost level can be considered as the minimum and requires a high level of pretreatment. Smith (74) reported a cost of approximately 8.8¢/K gal adjusted to June 1967 for a 7.5 MGD plant. Costs based on EPA 150 index (January 1971) now show a cost of approximately 14¢/K gal (2).

It is apparent that the authors based their costs on different influent wastewater quality. Effluent from a stabilization pond will require additional treatment prior to carbon adsorption, and the level of this treatment must be balanced with the carbon processes. Ultimately, the costs for activated carbon are dependent on this pre-

TABLE C-1

THE EFFECTS OF PRETREATMENT ON CARBON DOSAGE AND CARBON COLUMN EFFLUENT QUALITY. (After Culp and Culp)

	Primary Downflow series		Secondary, Plus Plain Filtration Downflow	Chemically Flocculated and Filtered Second- ary Effluent Upflow Counter- current-17 min
Pretreatment	2 beds	4 beds	4 series beds	
Carbon contact	15 min	30 min	20 min	
Carbon dosage lb/mg	1,200	800	500	250
SS, mg/l	10	5	<1	<1
BOD, mg/l	20	10	<1	<1
COD, mg/l	65	45	12	12
TOC, mg/l	20	10	3	3
Color, units	-	-	4	4
Turbidity, JU	-	-	1.5	0.5

treatment which can most accurately be evaluated at this time only on the basis of further research.

An important phase of the activated carbon process is carbon regeneration. This requires drying and baking of the adsorbates between 212 and 1500°F followed by reactivation of the carbon at temperatures above 1500°F. Fossil fuel plants could provide this regeneration at slightly reduced costs, but this is not significant in overall costs. For the South Tahoe plant, fuel was approximately 6 percent of the regeneration costs which were 20 percent of the total cost (24). Thus, fuel costs for regeneration represent approximately 1.2 percent of overall costs.

The use of powdered activated carbon adsorption may prove advantageous if carbon requirements are low enough to allow one-time use. Advantages of powdered activated carbon are increased rates of adsorption, and the possibility of dosing the wastewater in the same equipment used for chemical flocculation (77). Much research is being performed at this time on the use and regeneration of powdered activated carbon. Preliminary results are encouraging and, therefore, the proposed research plan will include efforts to evaluate its use.

### Ozonation

Because the integrated system is intimately associated with a nearby source of electrical energy, generation of ozone for disinfection and residual organic destruction may be an attractive alternative to chlorination and activated carbon treatment of final effluent. Recent pilot studies undertaken by Air Reduction Company for The Environmental Protection Agency indicate that treatment of secondary effluents by ozonation is a viable alternative to activated carbon adsorption (78). Sixty percent COD reduction (35 mg/l to 15 mg/l) was achieved at an ozone dosage of 50 mg/l. Cost comparisons between ozonation and activated carbon adsorption indicated that for a 10 MGD plant and 80 percent ozone utilization efficiency, operating costs were estimated at 7.7¢/1000 gal. More than half of the operating cost is due to electric energy requirements. For comparison, activated carbon adsorption operating cost was quoted as being 8.3¢/1000 gal.

Other benefits accrue from ozonation as compared to activated carbon adsorption. In order of importance, they are: disinfection, aeration to increase dissolved oxygen, no odor as during anaerobic

column operation, and turbidity reduction. Disinfection with ozonation is particularly important as it occurs automatically and without additional cost. Also, it appears that chlorination of wastewater effluents may not be acceptable in the future because of the chlorine-induced toxicity (79). Studies are underway to compare the toxicity of several disinfection schemes including ozonation (80).

APPENDIX D

REMOVAL OF ALGAE FROM WASTE STABILIZATION

POND EFFLUENTS - A STATE OF THE ART

by V. Kothandaraman and Ralph L. Evans

Circular 108

Illinois State Water Survey

Urbana, Illinois

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## REMOVAL OF ALGAE FROM WASTE STABILIZATION POND EFFLUENTS — A STATE OF THE ART

by V. Kothandaraman and Ralph L. Evans

### Introduction

Treatment of municipal, industrial, and agricultural wastes employing stabilization ponds or lagoons has found increasing application within the past 20 or 30 years. Where land values are not excessive, the low cost of construction and operation and the demand for less technical competence in their operation compared with more sophisticated treatment facilities make lagooning, in one form or the other, the method of choice for the stabilization of many different types of waste materials.

Regardless of whether the lagoon is an oxidation pond, an anaerobic cell followed by an aerobic polishing pond, or a facultative lagoon, the effluent from each facility is likely to contain a significant concentration of algae. In some areas, notably the state of Michigan, the complete retention of wastewater in oxidation ponds is required except for a few occasional discharges during periods of high water in the receiving streams.<sup>1</sup> Where the total inflow into lagoons exceeds the evaporation losses, there is bound to be effluent discharges unless adequate capacities are provided. The planktonic algae discharged from lagoons have the potential for oxygen production in the receiving stream, but they also represent a significant load of energy-rich organic matter to the receiving stream.

With the adoption of wastewater effluent standards by water pollution abatement agencies, particularly with respect to suspended solids and biochemical oxygen demand, it becomes imperative either to design the oxidation pond facilities on a total wastewater retention basis or to provide means for separating the algae from pond effluents and disposing of the harvested residue.

This report summarizes the investigations of other research workers concerning methods of harvesting algae and disposing of that harvest, and should be helpful to consulting engineers and stream pollution abatement agencies. It was prepared by the Water Quality Section of the Illinois State Water Survey under the general supervision of Dr. William C. Ackermann, Survey Chief. Since several of the separation techniques employed in these investigations are physiochemical processes, a brief introduction to the algal cell wall characteristics is presented before discussing the algal removal methods.

### CELL WALL CHARACTERISTICS

Ives<sup>2</sup> first demonstrated experimentally that algal cells carry a negative electric charge and developed expressions for charge density variations. The charge density was found to be a function of the viscosity and dielectric constant of the disperse medium, temperature of the medium, and the concentration and valency of the ionic species in the medium. A suspension of *Chlorella* in distilled water exhibited marked pH dependence of charge densities, as shown in figure 1. The charge density was lowest at a pH of around 7. However, the algae remained electro-negative at all pH values investigated.

Ives<sup>2</sup> postulated that the mechanism of algal removal by chemical coagulants was charge neutralization of the negatively charged algae by the positively charged metal

and hydroxy metal ions and subsequent agglomeration and sedimentation. Other previously held theories like mechanical enmeshment, adsorption, and the protogel theory developed by Hay were considered by Ives as secondary in importance for the precipitation of algae.

Golueke and Oswald<sup>3</sup> conducted a series of experiments to investigate the relation of hydrogen ion concentrations to algal flocculation. The relationship between pH levels and the tendency of the algae to flocculate measured by the increase in clarity of the supernatant is shown in figure 2. A pH value of about 3 seems to be a critical point since flocculation was most extensive in this region. The flocculation cells were found to form compact clumps in a fashion indicating direct surface attraction. They postu-

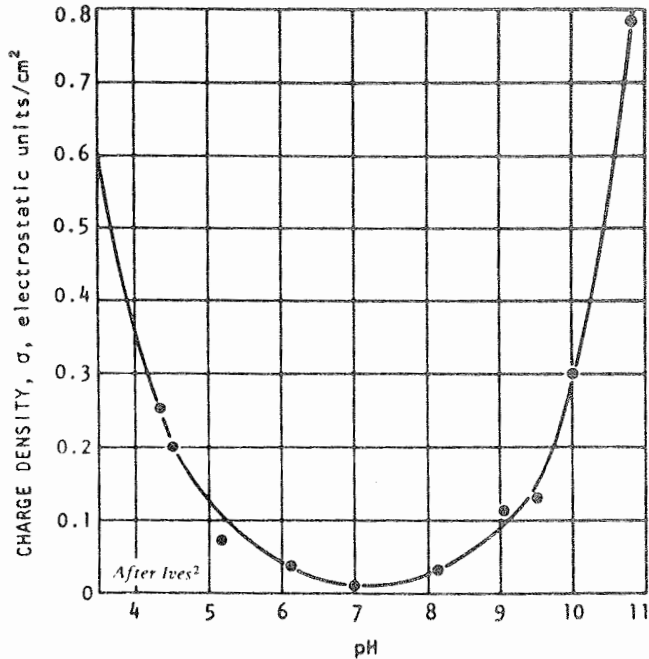


Figure 1. Chlorella — charge density-pH graph

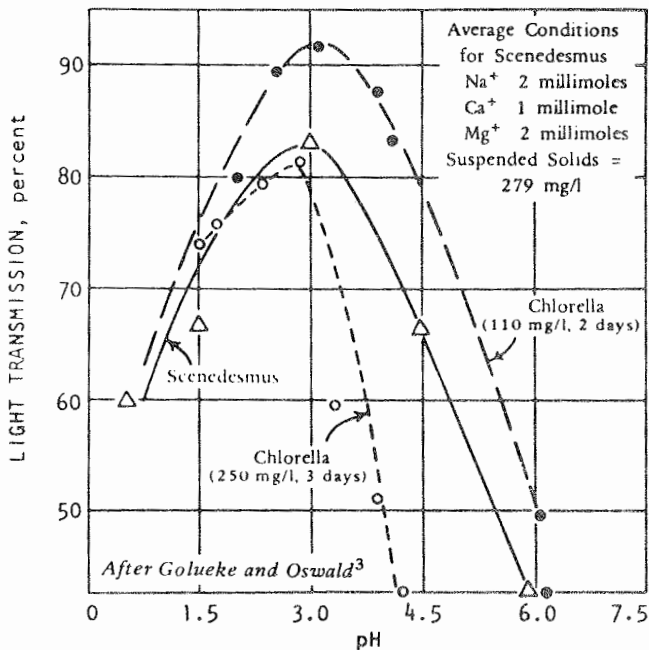


Figure 2. Effect of pH level on tendency of algae to flocculate as shown by the increase in clarity of the supernatant

lated that the free  $H^+$  not only served to satisfy the surface charge of the algal cells but also acted as a bonding agent. The greater the density of surface charge, the more pronounced was bonding. An indication that the  $H^+$  is bound by algal cells was inferred from the fact that an increase in ion concentration was required for maximum precipitation when the algal concentration was increased.

Although harvesting of algae by passage through ion-exchange columns is uneconomical,<sup>3,4</sup> results of ion-exchange column studies reveal much about their surface properties. Golueke and Oswald<sup>4</sup> found that pond algal culture when passed through strong and weak anion exchange resin columns resulted in no algal removal. However they found that algae could be removed by passing an algal suspension through a column either of a strong or weak cation exchange resin. The mechanism of removal apparently involved flocculation that resulted from a change in surface charge of the algal cells brought about by the charge of the resin.

This was demonstrated by the failure of the exchange columns to remove algae after their exchange capacity had been exhausted and their renewed ability following regeneration. The changed characteristics imparted to the algal cells persisted even after the algae had been removed from the columns by backwash. Algae in the backwash water promptly coagulated and settled. As the columns lost their ion-exchange capacity, the algae also lost their tendency to coagulate. The ion-exchange columns were effective, as far as algal removal was concerned, only when they were regenerated with  $H^+$ . Use of  $Na^+$  or of any cations other than  $H^+$  interfered with the algal removal capacity of the columns. Operating characteristics such as throughput rates, regeneration of columns, column lengths, methods of improving the regeneration efficiency, etc., are discussed in detail by Golueke and Oswald.<sup>4</sup>

It appears that the mechanism by which the cells were agglomerated in the presence of free  $H^+$  involves a combination of electrostatic forces and physical changes in the membrane of the cell. It is a physiological axiom that the pH of the medium in which algae are suspended exerts a strong influence on the permeability of the algal cell walls and may change the nature of the walls and affect the surface physicochemical make up of the walls.



## ALGAL SEPARATION

Methods and cost of algae removal assume great importance when planktonic algae must be removed from the effluent of a conventional stabilization pond before discharge into a receiving body of water. The mode of final disposal of the harvested algae in turn dictates the methods of algal removal from pond effluents and further processing. Algae intended for animal or human consumption must be processed so that they can be stored for long periods of time without deterioration. When the algae are harvested for their food values, a careful choice of chemicals must be made so that the chemicals do not have any toxic or other deleterious effects when ingested. Gradation in actual and in permissible costs of harvest will range from the lowest for the mere removal of algae, to higher for processing of the algae as a livestock feed, and to the highest for processing algae that are to be used as human food.

Technical and economic problems in algal harvest are largely due to the size, specific gravity, and morphology of the algal cells, their limited concentration and low market value. A combination of small size (5 to 15 microns) and low specific gravity results in a settling rate that is too slow to permit the use of settling as a routine procedure for harvesting algal cells. The small size of algal cells also necessitates the use of screens or filters having a pore size within the micropore range. The limited concentrations, 200 to about 4000 mg/l in oxidation ponds, involve handling of large volumes of liquid in order to recover a comparatively small amount of product.

Harvesting of algae generally involves three steps.<sup>3,5</sup> The first step, concentration or removal, increases the solids concentration from about 0.02 to 0.40 percent by weight to about 1 to 4 percent. The second step is dewatering which then brings the solids to 8 to 20 percent. Finally in the third step, the algal mass is dried to 85 to 92 percent solids by weight.

Relatively little work has been done on algal separation, especially at pilot scale levels. Oswald and Golueke<sup>3,6</sup> present a comprehensive review of the results of several years of work on the problems of separating algae grown on secondary sewage effluent. Also recently, the California Department of Water Resources<sup>5</sup> reported on the feasibility of nitrate removal from agricultural drainage by an algal system including harvesting of algae, based on pilot scale studies. In general, their results indicate that algae could be most economically concentrated by coagulation, flocculation, and sedimentation. Dewatering was accomplished by centrifugation, with final drying in the open. An alternative to separate dewatering and drying steps was to spread the concentrated slurry on sand beds, which brought about the desired solids concentration without the intermediate

step of dewatering. North American Aviation, as indicated in the California Department of Water Resources report,<sup>5</sup> found sand bed dewatering and drying feasible in the harvesting of sewage grown algae. In their studies also, the algae were concentrated by sedimentation after coagulation and flocculation and then spread on sand beds.

### Chemical Precipitation

Several investigators have reported on the efficacy of using chemicals, both mineral and organic, for coagulating and precipitating algae from suspension based on laboratory scale investigations.<sup>3,5,7,8,9</sup> Lin, Evans, and Beuscher<sup>7</sup> found that an overall algal reduction in excess of 85 percent in Illinois River water could be achieved using aluminum sulfate at a concentration of about 30 mg/l. Tenny et al.,<sup>9</sup> using mixed cultures of algae obtained from laboratory reactors and organic flocculants, found that algal flocculation occurred only with the addition of cationic polyelectrolytes and not with the addition of anionic and nonionic polymers. They postulated that a bridging phenomenon between discrete algal cells and the linearly extended cationic polymer chains, forming a three dimensional matrix that is capable of subsiding under quiescent conditions, was the possible mechanism involved. A concentration of the cationic polymer (C-31 of Dow Chemical Co.) in the range of 2 to 3 mg/l at a pH range of 2 to 4 was most effective.

Oswald and Golueke<sup>3</sup> and McGarry<sup>8</sup> found that anionic and nonionic polymers were ineffective in causing coagulation and precipitation of algae. However, the California Department of Water Resources found, on the basis of laboratory scale studies, that a few anionic polymers were experimentally beneficial and economically feasible aids in coagulation. These anionic polymers were not used in pilot scale evaluation.

Golueke and Oswald<sup>3</sup> reported that cationic polyelectrolytes, in the concentration range of 2.5 to 3.0 mg/l, resulted in an algal removal of about 80 to 90 percent. Complete removal of algae was achieved at a concentration of about 10 mg/l. The efficiency of algal removal by the cationic polyelectrolytes investigated were not affected in the pH range of 6.0 to 10.0.

The California Department of Water Resources<sup>5</sup> reported that out of 60 polyelectrolytes tested, 17 compounds were found to be effective coagulants and their costs were economically competitive when compared with mineral coagulants alone. Generally less than 10 mg/l of the polyelectrolytes was required for effective coagulation. A significant feature of this report is that a daily addition of 1 mg/l of ferric chloride to the algal growth pond resulted in significant reductions in the required dosages of both organic

and inorganic coagulants. After commencing the addition of ferric salt to the algal growth pond, 90 percent of the algae could be removed with 0.5 mg/l of Cat-Floc (cationic polyelectrolyte, Calgon Corp.). The cost of treating 1 million gallons of algal pond effluent at this concentration of Cat-Floc was estimated to be about \$2.00. Results of the reduction in mineral coagulant dosages due to the addition of ferric salt to the growth pond were presented also.

McGarry<sup>8</sup> has reported results of factorially designed experiments to optimize the process of chemical coagulation in harvesting algae obtained from a pilot high rate pond. Tests were performed to identify the economically feasible polyelectrolytes used as primary coagulants alone, or in combination with alum, and to investigate the independent variables which affected the flocculation process. Among the conclusions of the author, the significant ones are: 1) alum was effective for separation of algae from high rate oxidation pond waters and 2) the overall minimum cost per unit of algal yield was obtained with alum alone in the dosage range of 75 to 100 mg/l of alum. The polyelectrolytes used in the study did not reduce the overall costs of algal separation.

Speedy, Fisher, and McDonald<sup>10</sup> investigated the effectiveness of various prototype unit processes in a water treatment plant in removing algae from the raw water supply. They came to the conclusion that the use of alum as a coagulant is moderately effective in algal removal. However, use of lime as a coagulant appeared to be much more efficient. They found that different steps of treatment removed different kinds of algae at different rates.

Van Vuuren and Van Duuren<sup>11</sup> reported on the results of removal of algae from the Pretoria (South Africa) wastewater maturation pond effluent. They found that chemical coagulation with either alum or excess lime applied to algae-laden pond effluent yielded an acceptably clear and colorless water. They found also that all the polyelectrolyte coagulant aids at varying concentrations, tested in conjunction with lime or alum, failed to give improved results. The authors chose high concentrations for the primary coagulants, namely 110 mg/l of alum and 220 mg/l of lime, which probably masked the effectiveness of the several coagulant aids tested.

The California Department of Water Resources<sup>5</sup> evaluated the use of lime, alum, and ferric sulfate as primary coagulants during their laboratory and pilot scale studies for harvesting algae. Their experience shows that the concentration required of each of these chemicals to remove the desired amount of algae varied according to the operating conditions of the growth units and was relatively independent of the initial algal concentration.

The concentration of lime needed for 90 percent removal of algae varied from 20 to 200 mg/l. Another benefit as a result of the use of lime as a coagulant was the almost complete removal of phosphorous. However, the pH of the lime treated effluent may have to be brought back to acceptable levels before final discharge to the receiving waters. Alum concentrations of about 50 to 200 mg/l were required to bring about 90 percent algal removal. The requirement of ferric sulfate was found to be about 140 mg/l.

The dramatic effect of the daily addition of 1 mg/l of ferric chloride to the growth pond on required dosages of all three chemicals is illustrated in figure 3. Without any additional chemical, the removal level was increased from 40 to 70 percent, presumably as a result of coagulation by the ferric chloride added for growth. The effective level of  $\text{Fe}_2(\text{SO}_4)_3$  dosage was reduced from 140 to 5 mg/l. The concentrations of both lime and alum were also reduced, with alum being effective at about 20 mg/l and lime at 40 mg/l. However, a steady build up of iron in the supernatant, due to the addition of ferric chloride to the growth pond, was noticed. Shown in table 1 are cost estimates of removing 90 percent of the algae from 1 million gallons of pond effluent.<sup>5</sup> Figures in the table show that the cost of these chemicals for separation of algae may be reasonable and that ferric sulfate is the cheapest of the primary flocculants tested.

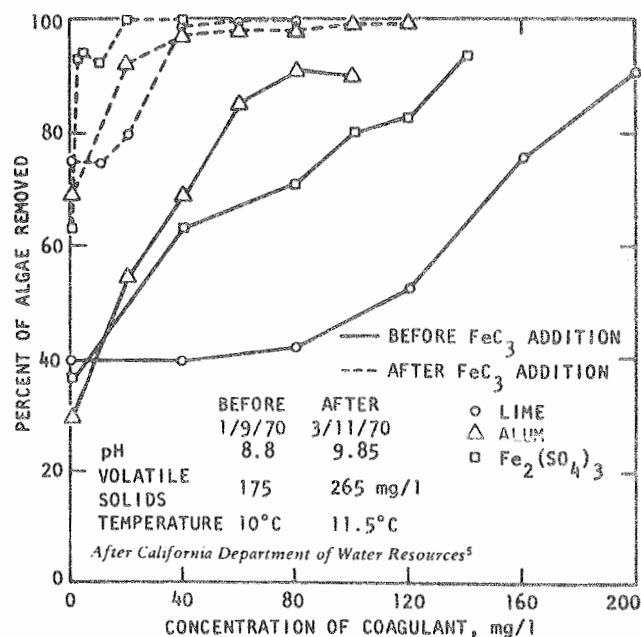


Figure 3. Concentrations of lime, alum, and ferric sulfate required for 90 percent algae removal — before and after addition of ferric chloride to rapid growth pond

**Table 1. Estimated Cost of Lime, Alum, and Ferric Sulfate to Remove 90 Percent Total Suspended Solids**

Date	Volatile solids (mg/l)	(Dollars/million gallons)		
		Alum	Lime	Ferric sulfate
11/19/69	250	11.30	16.80	14.40
12/03/69	200	12.30	14.60	
12/11/69	100	14.00	20.00	25.00
1/09/70	175	13.30	20.80	23.80
1/16/70	71	11.90	15.90	22.30
1/23/70	91	12.70	8.20	24.40
2/11/70*	114		3.50	11.00
2/18/70	210	13.00	5.40	1.50
2/25/70	417	2.35	1.30	0.25
3/11/70	265	3.46	3.28	0.40
3/18/70	229	3.50	3.23	0.62

\*Commenced 1 mg/l daily addition of Fe<sup>+++</sup> to pond on 1/26/70

In their pilot studies, the settling chamber consisted of a module of settling tubes inclined at an angle of 7½° upward in the direction of flow. This was a part of the self-contained water treatment plant called the "Water Boy" manufactured by Neptune Microfloc, Corvallis, Oregon. The slurry taken directly from the tubes contained from 1.1 to 1.6 percent solids. Overnight settling of the slurry increased the percent solids to 4 to 6 percent. However, tests of settling as a function of time indicated that 80 percent of the settling occurred in the first 6 to 8 hours.

Golueke and Oswald<sup>3</sup> reported on the efficacy of algal removal using aluminum sulfate based on laboratory, pilot, and field scale studies. Laboratory studies were concerned with determining the relation between dosage and pH level, the effect of floc aids, of stirring and settling times, the amount of aluminum in the precipitated product, and ways of removing aluminum from the harvested product and of determining the quality of the supernatant. Best removal of algae was obtained when the pH of the suspension was 6.5. Removal became increasingly poorer as the pH was raised above 7.0. A mixing time of 3 minutes was found to be adequate at a blade tip velocity of 12 inches per second. A settling time of about 15 minutes was sufficient.

The greatest yield of algae per milligram of alum was obtained at a dosage of 70 mg/l. However, highest clarity of the supernatant was obtained in the dosage range of 90 to 100 mg/l. In their experiments in which flocculant aids were added in addition to the alum coagulant, no improvement with respect to settling characteristics of the floc par-

ticles, clarity of supernatant, or amount of required dosage was noted over that obtained with alum alone.

In the pilot and field scale studies, similar results were obtained. The sediment concentration in the flocculation-sedimentation unit with a detention time of 2 to 3 hours averaged 1.5 percent by wet weight.

The authors found that most of the algae could be removed by raising the pH to 10.6 or above with lime. Very little algal precipitation occurred at pH levels from 9.5 to 10.5. Their results showed that the use of ferrous sulfate as an additive brought about a distinct improvement in precipitation, the extent of which was in direct proportion to FeSO<sub>4</sub> dosage until a critical point of 40 mg/l was reached. Above this concentration further gain in clarity was not accomplished. The required dosage of Ca(OH)<sub>2</sub> was reduced correspondingly. A combined dosage of 40 mg/l FeSO<sub>4</sub> and 120 mg/l Ca(OH)<sub>2</sub> resulted in a supernatant clarity having 86 percent light transmission; whereas at 200 mg/l Ca(OH)<sub>2</sub> and no FeSO<sub>4</sub>, it was only 78 percent.

The advantage in reduction of cost of chemicals that could result from the use of an iron salt in conjunction with a primary coagulant has to be weighed by the disadvantages resulting from the production of an algal slurry and a supernatant containing iron. Iron in the floc would constitute an undesirable element in algal product used as a feedstuff for livestock, and its excess in the supernatant would make the latter undesirable for discharge into receiving waters.

### Centrifugation

Oswald and Golueke<sup>3,6</sup> experimented with centrifuges to determine the effect of feed throughput rates, cell concentration, rotational velocity, underflow discharge rates, power requirements, etc.

Removal of algae from the influent algal culture (concentration 200 mg/l) ranged from 84 percent at a throughput rate of 100 gpm to about 64 percent at 385 gpm at rotational velocities of 3000 to 3300 rpm. The details of the effect of throughput rate, the disc angle and the rotational speed of bowl on power requirements are given in the reference. The authors estimated that the minimum power requirement for concentrating algal culture at a concentration of 200 mg/l to be about 2.7×10<sup>3</sup> kwh per ton (dry weight) of algae. The power requirements could be halved by doubling the initial algal concentration.

The California Department of Water Resources<sup>5</sup> reported the results of the evaluation of the De Laval self-cleaning centrifuge both as a primary concentration and as a dewatering device. Used as a primary concentrator, the unit removed up to 95 percent of the influent algae (concentra-

tion 800 mg/l). The flow through the unit was 6 gpm and the effluent slurry contained about 10 to 12 percent solids. A problem in plugging was encountered because of incomplete discharge of material in the bowl. This problem was solved by an operational procedure called "double shoot." After the change in the operational procedure, a product with 17 percent solids was obtained, but the percent removal dropped to about 80. The self-cleaning centrifuge, used as a dewatering device, was tested with an influent containing 20,000 to 30,000 mg/l solids. At a flow rate of 2.75 gpm, the slurry contained about 10 percent solids. At that loading, the centrifuge removed more than 98 percent of the influent suspended solids.

Although centrifugation offers the advantage of simplicity and continuity of operation and the production of material high in quality and devoid of additive reagents, it has the obvious economical disadvantages of high initial cost, and relatively high demand of electrical power.

### Flotation

Golueke and Oswald<sup>3</sup> reported on laboratory scale flotation experiments. Of 18 different flotation reagents tried in their experiments, appreciable concentration of algae was obtained only with two reagents. Even with those two, the extent of removal was too small to be practical. However, Levin et al.<sup>12</sup> reported a flotation method in which the cell concentration of the harvest is a function of pH, feed concentration, age of the culture, and height of foam in the processing column. A pH level of about 3.0 was found to give best results. Levin et al.<sup>12</sup> used synthetic medium to raise 6 different algal species in their laboratory scale batch-type harvesting studies. Without any chemical additives, they were able to concentrate algae in the foam to the extent of 5 to 8 percent of solids content.

A major disadvantage of this method is the need to reduce the pH of the pond liquid to the required low level and subsequent readjustment to an acceptable level before discharge to receiving waters.

### Microstraining

Though successful application of microstrainers in the removal of algae from raw water supplies has been reported,<sup>13,14</sup> their use with pond effluents appears to be very much limited. Golueke and Oswald<sup>3</sup> carried out pilot scale experiments to evaluate the use of microstrainers in algae removal. Flow rates varied from 50 to 100 gpm and the microstrainer was rotated at 10, 20, and 30 rpm. Only very small amounts of algae were removed even with the addition of filter aids, a decrease of flow rates, and slowing of the rotational speed of the filter.

The California Department of Water Resources<sup>5</sup> reported that screens of pore sizes 25 and 35 microns were ineffective in algal removal. Removals up to 30 percent were obtained, but most of this was due to algal settling in the influent and effluent chambers.

### Upflow Clarifiers

Upflow clarifier with detention times of about 1 hour was found to be effective in concentrating algae from pond effluents.<sup>5</sup> With an addition of 100 to 200 mg/l of sodium hydroxide, the upflow clarifier was found to remove as much as 95 to 100 percent of suspended solids and produced a slurry containing 2 to 10 percent solids.

Although the unit did an effective job of removing the algae, many operational problems arose which prevented long run times. The most serious of the problems was caused by the consolidation of the algal mat, which caused increased hydraulic pressure resulting in sloughing of the algal floc. When this occurred, normal operation of the unit did not resume until the mat built up again. Also, the pH of the effluent from the clarifier should be readjusted to an acceptable level before final disposal.

### Filtration

Borchardt and O'Melia<sup>15</sup> carried out laboratory scale studies on the removal of uni-algal cells by sand filtration. The effects of variables such as algal concentration in the inflow, sand size, sand depth, flow rate, etc. on head loss and the efficiency of algal removal were reported. The removal efficiency was found to decrease with time and the algal cells were found to penetrate through the sand bed even in the early stages of each of their filtration experiments.

Foess and Borchardt,<sup>16</sup> in laboratory scale studies, reported that the attachment mechanism for discrete algae in sand filtration involved a surface interaction between the particle and the sand grain which can be chemically controlled. Stock cultures of *Chlorella* or *Scenedesmus* in required concentrations were used in their studies. Solution pH was determined to be an effective parameter for controlling the filtration process. When the pH was lowered, making interaction energies between diffuse layers of the sand and algal particles more favorable for adsorption of the two particles, removals significantly increased.

Golueke and Oswald<sup>3</sup> reported on the laboratory scale experiments on filtration with the use of a Buchner funnel and a 3-cm diameter filter leaf. Diatomaceous earth, corn meal, corn starch, and calcined rice hulls were used as filter aids. Effectiveness of paper, fine mesh metal, nylon, cotton, and woolen screens of a wide variety of porosities, as well as teflon cloth (14-20 micron pore size), was evalu-

ated. In the absence of filter aids, algae were able to pass through all filter media tried. Complete removal of cells was obtained when diatomaceous earth, corn starch, and calcined rice hulls were used as filtering aids.

Concentration of algae by gravity filtration does not appear to be promising because of clogging of the filter media with the concomitant increase in head loss and decrease in throughput rates. Frequency of backwashing has to be increased to maintain a desirable filtration rate. No pilot scale or field scale operation of algal removal by filtration has been attempted yet.

### Miscellaneous Methods

Golueke and Oswald<sup>3</sup> reported the phenomenon of natural flocculation and precipitation of oxidation pond algal cells under certain conditions. They termed this phenomenon as autoflocculation. The required conditions were an actively photosynthesizing algal mass in a shallow pond, sunlight, and a relatively warm day. Separation of algae by autoflocculation alone would necessitate a large surface area

## DEWATERING METHODS

Algal concentration resulting from the first step of algae separation from the pond effluent varies from 1 to 4 percent by weight. For economical handling of the algae for final disposal, it is necessary to dewater so that the solids concentration could be increased to at least 10 to 20 percent by weight. Pilot scale studies of vacuum filtration, centrifugation, and gravity filtration were reported as dewatering steps by Golueke and Oswald<sup>3</sup> and the California Department of Water Resources.<sup>5</sup>

### Vacuum Filtration

Attempts to dewater an algal slurry with a vacuum filter were found to be unsuccessful by Golueke and Oswald<sup>3</sup> because of the inability to form a cake of sufficient thickness to permit its removal. The California Department of Water Resources<sup>5</sup> experimented with a continuous belt vacuum filter. It was found that at a belt speed of 2.9 ft/min and with a vacuum of 15 to 20 inches of mercury, the unit could produce sludge containing 18 to 25 percent solids and remove 90 to 95 percent of the influent suspensions. Average concentration of solids in the effluent was about 300 mg/l which necessitated recycling of the effluent from the unit.

### Centrifugation

Golueke and Oswald<sup>3,6</sup> reported on the results of de-

watering algal slurry using four different centrifuges, namely, the Byrd solid bowl, the Tolhurst solid bowl, the De Laval, and the MercoBowl (Dorr-Oliver) centrifuges. Except for the Byrd centrifuge, excellent results were obtained. At a feed rate of 2 to 4 gpm with an initial solids concentration of about 1100 mg/l, the centrifuged slurry was found to have a solids concentration of about 12 percent. The percent removal was about 78 percent. The experience of the California Department of Water Resources in dewatering by centrifuges has been mentioned earlier.

The same authors examined the possibility of algal separation by sonic vibration and by the passage of the pond effluent through a charged field. When algae were exposed to ultrasonic waves of 15,000 cycles and up, they were dispersed effectively at all of the frequencies tried. Algal separation by passage through an electric field was also tried. Excellent separation of algae occurred when aluminum or copper electrodes were used because of the formation of copper or aluminum hydroxide due to the release of  $\text{Cu}^{++}$  or  $\text{Al}^{3+}$  from the electrodes. No separation took place when two carbon electrodes were used. Copper, carbon, and aluminum electrodes were used either as pairs of only one of the materials, or as pairs composed of two different materials. Distance between the electrodes varied from 1/8 to 1/2 inch. Tested flow rates ranged from 0.05 to 1.2 gpm/cu ft of electrolyte cell volume. Current was varied from 0 to 900 milliamperes. Applicability of this method on a large scale was not investigated.

watering algal slurry using four different centrifuges, namely, the Byrd solid bowl, the Tolhurst solid bowl, the De Laval, and the MercoBowl (Dorr-Oliver) centrifuges. Except for the Byrd centrifuge, excellent results were obtained. At a feed rate of 2 to 4 gpm with an initial solids concentration of about 1100 mg/l, the centrifuged slurry was found to have a solids concentration of about 12 percent. The percent removal was about 78 percent. The experience of the California Department of Water Resources in dewatering by centrifuges has been mentioned earlier.

### Gravity Filtration

Conventional types of filter media such as those used in sludge dewatering, namely, nylon, wool felt, canvas, paper, and paper backed with sponge, were tried by Golueke and Oswald.<sup>3</sup> All media proved to be capable of retaining the algae and suspended solids contained in the algal slurry. Nylon filters were easy to clean and proved durable. Wool felt was an effective medium but the felt fibers became entangled in the algal cake. Consequently, it was not found suitable as a filter medium. All the industrial filter papers proved satisfactory.

In pilot scale studies by Golueke and Oswald,<sup>3</sup> algal slurry was dewatered on sand beds framed with boards and covered with an industrial grade filter paper. In these studies it was shown that at an initial slurry depth of 2 inches, the

algal slurry could be dewatered and air dried on the paper to a 12 to 15 percent moisture content within 24 to 30 hours. The paper could be used repeatedly.

The process of dewatering algal slurry on sand beds by drainage and allowing it to dry by evaporation was found to be the cheapest means of dewatering and drying algal slurries.<sup>3</sup> In pilot plant studies the sand used had particle sizes such that all of it passed through a 50-mesh screen and all but 11 percent was retained on a 140-mesh screen. A slurry was applied to the bed at a depth of 4.5 to 5 inches. Spoilage was found to occur when the applied

depth was greater than 5 inches. After 24 to 48 hours the dewatered material had a solids content of 7 to 10 percent. Five to seven days of drying were required to bring the moisture content down to 15 to 20 percent. When dried, the algae formed chips. Most of the sand adhering to the dried algal chips could be removed by sieving the flakes over a 1/16-inch mesh screen. Firmly attached sand constituted from 2 to 3 percent of the dry weight of algal chips. Golueke and Oswald<sup>3</sup> estimated that about 6400 square feet of drying bed per acre of pond (algal concentration 200 mg/l) would be required. Also, a loss of about 15 percent of the sand per year should be expected.

## DISPOSAL OF HARVESTED ALGAE

Harvested algae must be disposed of ultimately in a manner which does not create nuisance conditions or health hazards. The California Department of Water Resources considered the following alternatives for the final disposal of harvested algae.

### Animal Food Supplement

Recently, there has been interest in the possible uses of unicellular algae as a protein source, especially for livestock feeding. As reported by the California Department of Water Resources,<sup>5</sup> the North American Aviation Company, Hintz et al., Leveille et al., Foree and McCarty, and others have experimented with sewage grown algae as livestock feed. Their investigations indicate that algae can be used as a substitute for such protein supplements as soybean, cottonseed, or fish meal especially in poultry feeding. A

lucrative and sustained market for algae as a food supplement is yet to develop in this country.

### Soil Conditioner

The California Department of Water Resources is of the opinion that, because algal products have a higher nitrogen content than dried activated sludge, they may find a receptive market as a soil conditioner, particularly for lawn, golf greens, etc. Presence of various salts in the algal product combined with its slow rate of decomposition, make algae a desirable lawn conditioner.

### Gas Production

Suggestions have been made that an algal product can be converted to usable energy through methane fermentation with subsequent use of the methane gas to provide heat and/or electric power.

## SUMMARY

With exacting requirements for treated waste effluents, particularly with regard to the suspended solids, the problem of algal removal from oxidation ponds and lagoons has become increasingly important.

Uni-algal cells have been found to carry a negative charge in the pH range of 2 to 11. They possess high charge densities at pH of 2 and 11, and a very low negative charge density at a pH of around 7. The chemical precipitation of algae has been postulated to be due to charge neutralization, agglomeration, and sedimentation. Even in sand filtration, surface interaction between algae and sand particles was found to be more significant than physical straining.

The handling of algae, in the removal process, consists essentially of three steps, namely, concentration, dewater-

ing, and drying. In the first step, concentration of algae in the pond effluent varying from 200 mg/l to 4000 mg/l is increased to 1 to 4 percent by weight. In the dewatering step, the concentration of algae is increased to 8 to 20 percent; and in the final step it is increased to 85 to 90 percent.

Chemical coagulants, primarily cationic polymers, lime, alum, and ferric salts are effective in bringing about the coagulation and sedimentation of algal cells. Addition of small quantities of ferric salts either to the algal growth ponds or in the coagulation step, along with the primary coagulants like lime, alum, and cationic polymers, greatly enhanced the removal efficiency at much lower dosage rates of the primary coagulants compared with results for the primary coagulants alone.



Dewatering and drying of the algal slurry obtained from the concentration step could be most economically carried out by sand bed application. Vacuum filtration has been found to be only partially successful. High initial cost and recurring power costs render centrifugation only marginally attractive.

Use of sewage-grown algae as livestock feed and particularly as poultry feed appears to be promising, but a market has yet to develop. Use as soil conditioner, particularly for lawns, has been suggested. Anaerobic digestion, land fill, and other conventional methods of solids disposal from municipal waste treatment plants should be explored.

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## APPENDIX E

### COSTS OF POTENTIAL SYSTEMS

Tables E-1 through E-5 summarize the capital and total amortized costs used for economic evaluation in this study. The cost estimating procedures and the process system cost elements are delineated in Tables E-6 and E-7, respectively. Figures E-1 through E-7 summarize the data compiled by Southwest Research Institute (2).



Table E-1  
Raw Wastewater  
560 Acre Pond

Process	Flow, MGD Through Units	Total Amortized Costs		Capital Cost \$ x 10 <sup>6</sup>
		¢/K gal of Plant Inflow	Annual Cost \$ x 10 <sup>3</sup>	
Primary	15	7.9	430	2.8
Pond	15	-	-	-
SS + P Removal*	10	5.3	290	0.8
NH <sub>3</sub> Stripping	10	3.3	180	1.1
Filtration	10	2.4	130	0.7
Activated Carbon	10	8.3	460	2.2
Chlorination	10	0.7	40	0.1
<b>TOTAL</b>		<b>27.9</b>	<b>1,530</b>	<b>7.7</b>

Raw Wastewater  
1400 Acre Pond

Primary	45	5.8	950	6.4
Pond	45	-	-	-
SS + P Removal*	25	3.9	640	1.7
NH <sub>3</sub> Stripping	25	2.4	390	2.5
Filtration	25	1.3	220	1.2
Activated Carbon	25	6.0	990	5.0
Chlorination	25	0.6	90	0.2
<b>TOTAL</b>		<b>20.0</b>	<b>3,280</b>	<b>17.0</b>

\* Two-stage lime treatment plus chemical costs

Table E-2  
Raw Wastewater  
560 Acre Pond

Process	Flow, MGD Through Units	Total Amortized Costs		Capital Cost \$ x 10 <sup>6</sup>
		¢/K gal of Plant Inflow	Annual Cost \$ x 10 <sup>3</sup>	
Primary* (w/lime addition)	15	11.3	620	3.1
NH <sub>3</sub> Stripping	15	4.6	250	1.6
Pond	15	-	-	-
SS + P Removal**	10	5.3	290	0.8
Filtration	10	2.4	130	0.7
Activated Carbon	10	8.3	460	2.2
Chlorination	10	0.7	40	0.1
<b>TOTAL</b>		<b>32.6</b>	<b>1,790</b>	<b>8.5</b>

Raw Wastewater  
1400 Acre Pond

Primary * (w/lime addition)	45	9.1	1,500	7.0
NH <sub>3</sub> Stripping	45	4.0	660	3.2
Pond	45	--	-	-
SS + P Removal**	25	3.9	640	1.7
Filtration	25	1.3	220	1.2
Activated Carbon	25	6.0	990	5.0
Chlorination	25	0.6	90	0.2
<b>TOTAL</b>		<b>24.9</b>	<b>4,100</b>	<b>18.3</b>

\* Primary treatment plus lime addition equipment and chemical costs.

\*\* Two-stage lime treatment plus chemical costs.

Table E-3  
 Secondary Effluent  
 560 Acre Pond

Process	Flow, MGD Through Units	Total Amortized Costs		
		¢/K gal of Plant Influent	Annual Cost \$ x 10 <sup>3</sup>	Capital Cost \$ x 10 <sup>6</sup>
Pond	15	-	-	
SS + P Removal*	10	5.3	290	0.8
NH <sub>3</sub> Stripping	10	3.3	180	1.1
Filtration	10	2.4	130	0.7
Activated Carbon	10	8.3	460	2.2
Chlorination	10	0.7	40	0.1
<b>TOTAL</b>		<b>20.0</b>	<b>1,100</b>	<b>4.9</b>

Secondary Effluent  
 1400 Acre Pond

Pond	45	-	-	-
SS + P Removal*	25	3.9	640	1.7
NH <sub>3</sub> Stripping	25	2.4	390	2.5
Filtration	25	1.3	220	1.2
Activated Carbon	25	6.0	990	5.0
Chlorination	25	0.6	90	0.2
<b>TOTAL</b>		<b>14.2</b>	<b>2,330</b>	<b>10.6</b>

\* Two-stage lime treatment plus chemical costs.

Table E-4  
Secondary Effluent  
560 Acre Pond

Process	Flow, MGD Through Units	Total Amortized Costs		Capital Cost \$ x 10 <sup>6</sup>
		¢/K gal of Plant Inflow	Annual Cost \$ x 10 <sup>3</sup>	
Phosphorus Removal *	15	7.1	390	1.1
NH <sub>3</sub> Stripping	15	4.6	250	1.6
Pond	15	-	-	-
SS Removal*	10	5.3	290	0.8
Filtration	10	2.4	130	0.7
Activated Carbon	10	8.3	460	2.2
Chlorination	10	0.7	40	0.1
<b>TOTAL</b>		<b>28.4</b>	<b>1,560</b>	<b>6.5</b>

Secondary Effluent  
1400 Acre Pond

Phosphorus Removal *	45	6.6	1,100	2.8
NH <sub>3</sub> Stripping	45	4.0	660	3.2
Pond	45	-	-	-
SS Removal *	25	3.9	640	1.7
Filtration	25	1.3	220	1.2
Activated Carbon	25	6.0	990	5.0
Chlorination	25	0.6	90	0.2
<b>TOTAL</b>		<b>22.4</b>	<b>3,700</b>	<b>14.1</b>

\* Two-stage lime treatment plus chemical costs.

Table E-5

## Municipal Costs

15 and 45 MGD Flows

Process	Flow, MGD Through Units	Total Amortized Costs		
		¢/K gal of Plant Influent	Annual Cost \$ x 10 <sup>3</sup>	Capital Cost \$ x 10 <sup>6</sup>
Activated Sludge (includes primary)	15	12.0	660	5.5
P Removal *	15	7.1	390	1.1
NH <sub>3</sub> Stripping	15	4.6	250	1.6
Filtration	15	3.0	160	0.8
Activated Carbon	15	12.0	660	3.1
Chlorination	15	1.1	60	0.2
TOTAL		39.8	2,180	12.3
* Two-stage lime treatment plus chemical cost				
Activated Sludge (includes primary)	45	10.5	1,700	13.0
P Removal *	45	6.6	1,100	2.8
NH <sub>3</sub> Stripping	45	4.0	660	3.2
Filtration	45	2.1	340	1.7
Activated Carbon	45	10.0	1,650	8.3
Chlorination	45	0.9	150	0.3
TOTAL		34.1	5,600	29.3

Table E-6  
Cost-Estimating Procedures  
(after Southwestern Research Institute, 2)

Capital costs include plant and equipment, land, and indirect capital costs. Indirect capital costs of 20 percent are included to cover interest during construction and contingency costs. These are adjusted to coincide with the EPA Treatment Plant Cost Index of 150.

Operating and maintenance costs include salaries, wages, electricity, chemicals, and other supplies, plus miscellaneous items, and are adjusted to the U.S. Department of Labor Average Earnings for Nonsupervisory Workers in Water, Steam, and Sanitary Systems Index of 3.50.

Amortization of capital costs is based on a 25-year loan at 6 percent interest, as suggested by the EPA. Pipeline costs are amortized on a 50-year loan period.

Total unit costs for each process are the sum of the amortized capital investment and operation and maintenance costs stated on a unit cost basis of ¢/1000 gal of product water processed.

## Table E-7

## Process System Cost Elements

(after Southwestern Research Institute, 2)

Primary Treatment: Costs include screening, comminution, main-lift pumping, grit removal, and primary clarification of waste waters. Sources of capital costs are from Smith (74); operating and maintenance costs are from Michel, Pelmoter and Palange (81).

Activated Sludge Treatment: Costs include primary treatment, followed by an activated sludge treatment process consisting of aeration, sedimentation, clarification, and sludge handling (81, 82, 74).

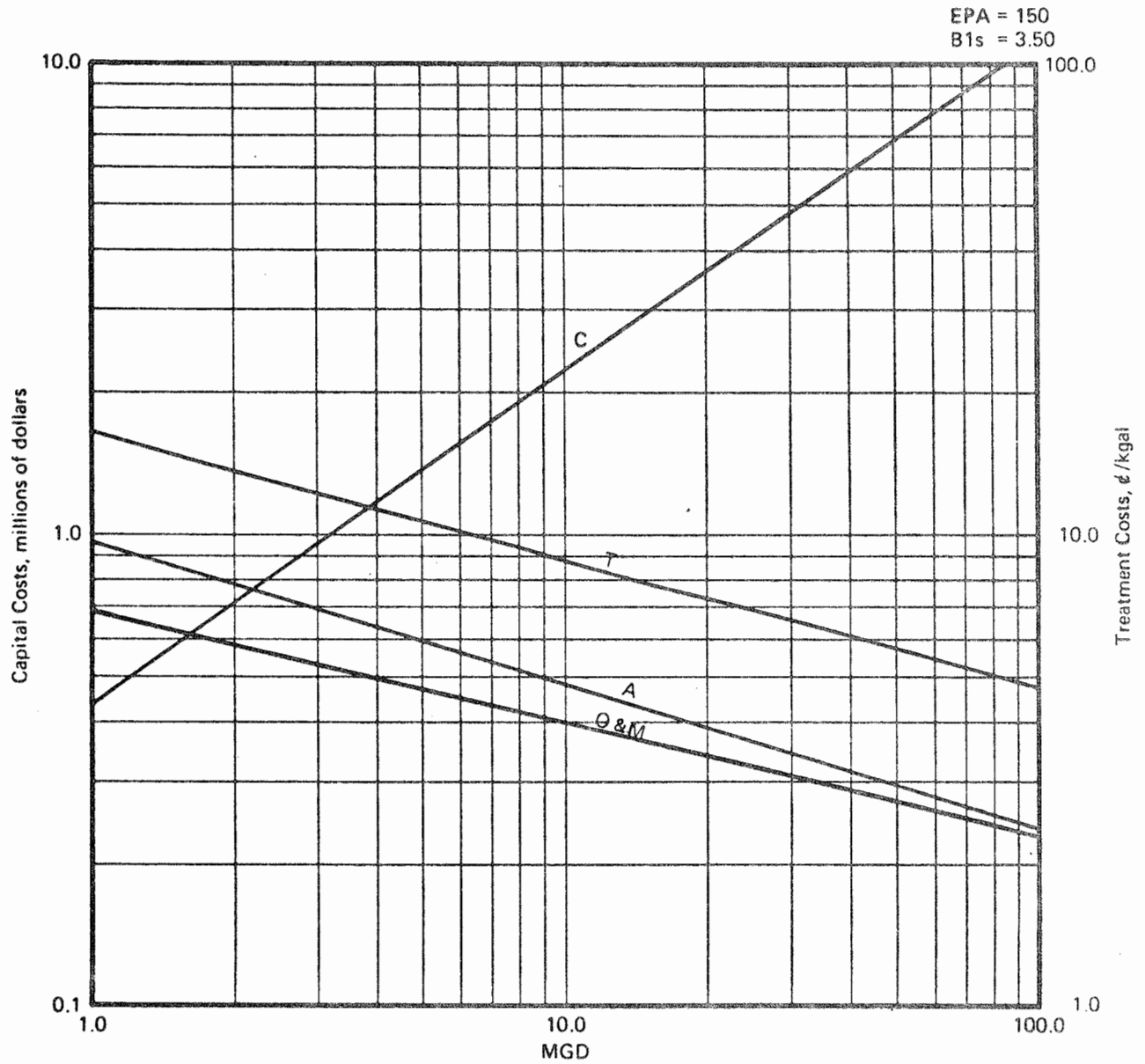
Lime Treatment Without Chemicals: The costs shown include lime addition facilities, solids contact clarifiers, and sludge concentrating equipment. For purchased lime and  $\text{FeSO}_4$ , 2.7¢/K gal should be added to the O & M cost (83).

Ammonia Stripping: The costs associated with this process are for stripping towers, complete with fans and lift pumps (74, 83).

Rapid Filtration Through Sand or Graded Media: Costs shown are for filtration at a rate of 4 gpm/sq ft with associated backwash equipment (74, 83).

Granular Carbon Adsorption: Costs include carbon adsorption columns, initial carbon charge, backwash and regeneration, and ash disposal facilities required for treatment of secondary effluent (74, 83).

Chlorination of Secondary Effluent: Costs are based on 15-min contact time with the addition of 8 mg/l (74).



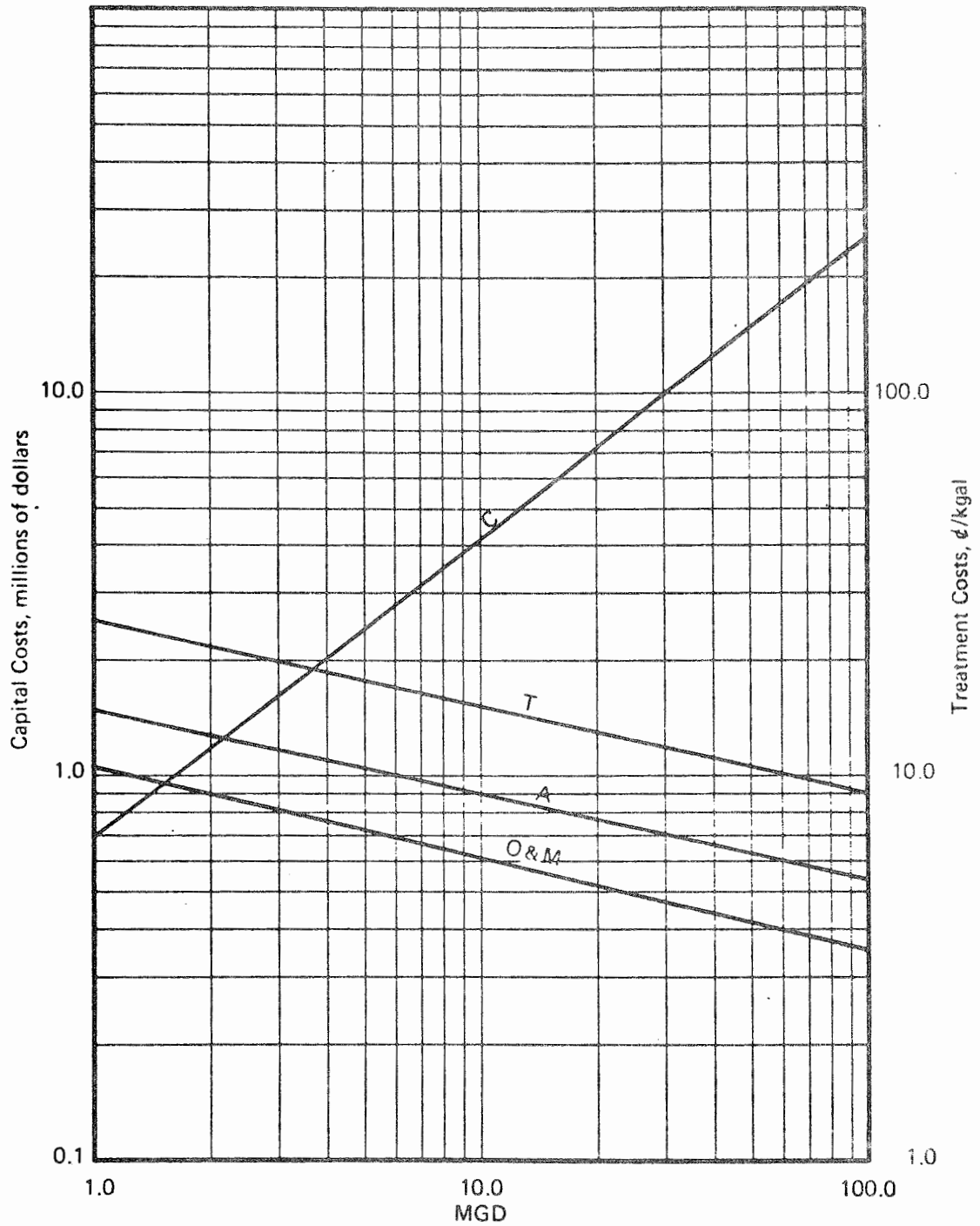
- C = Capital Cost, millions of dollars
- A = Debt Service, ¢/1000 gal (6% - 25 yr)
- O&M = Operating and Maintenance Cost, ¢/kgal
- T = Total Treatment Cost, ¢/1000 gal

Source: Southwest Research Institute (2)

Figure E-1  
PRIMARY TREATMENT



EPA = 150  
B1s = 3.50



- C = Capital Cost, millions of dollars
- A = Debt Service, ¢/1000 gal (6% - 25 yr)
- O & M = Operating and Maintenance Cost, ¢/kgal
- T = Total Treatment Cost, ¢/1000 gal

Source: Southwest Research Institute (2)

Figure E-2  
ACTIVATED SLUDGE TREATMENT  
(Includes Primary Treatment and Sludge Handling)

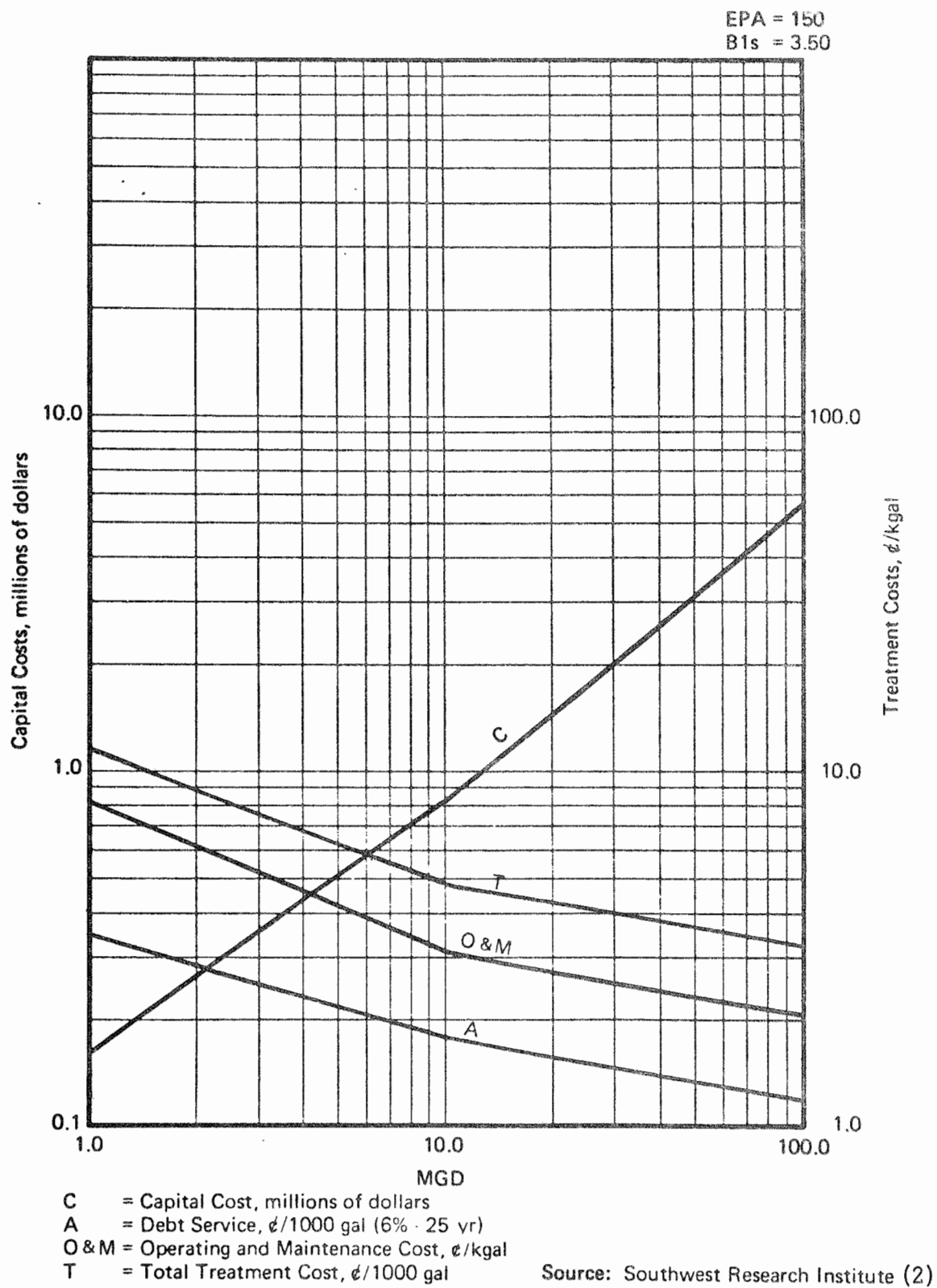
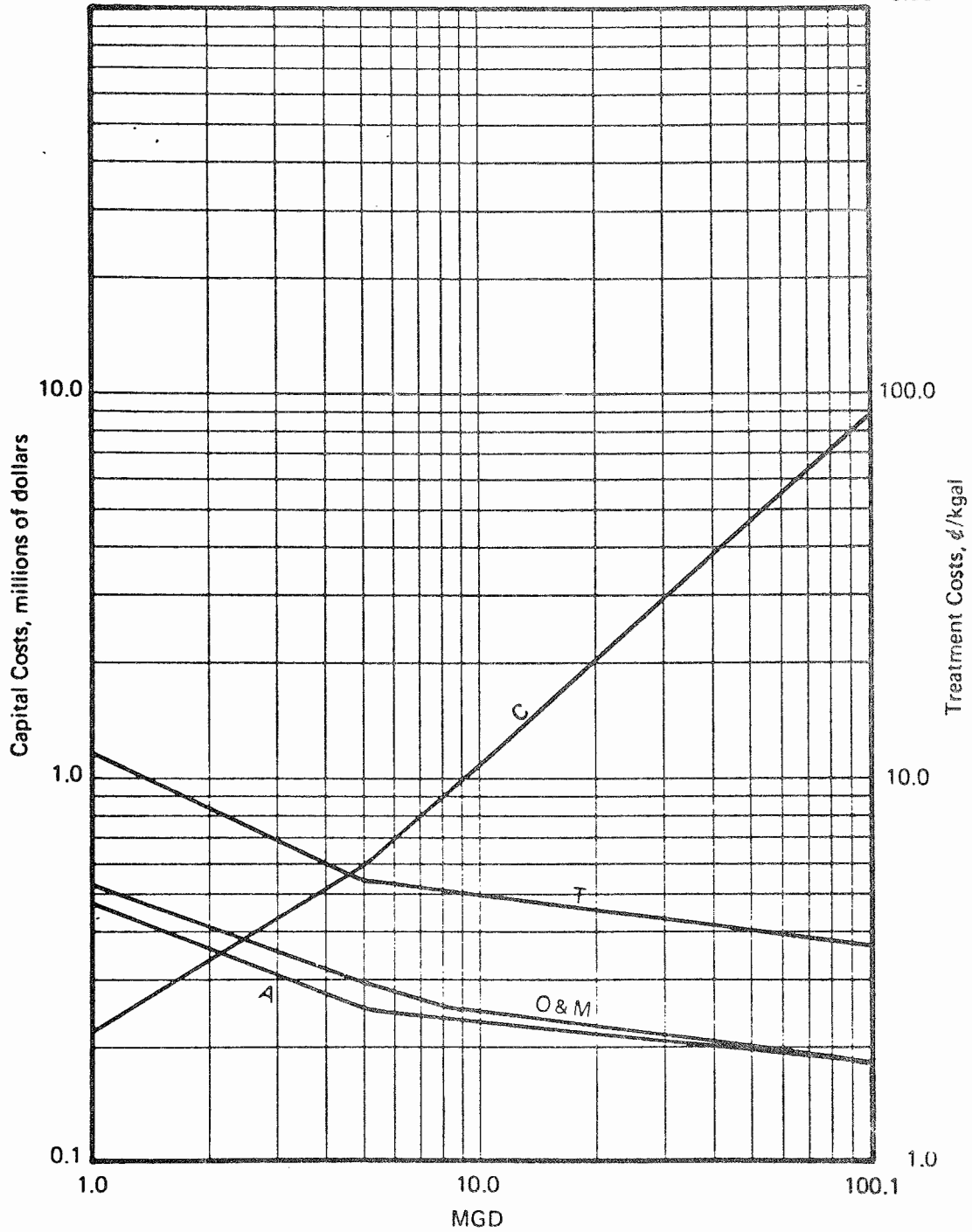


Figure E-3  
TWO CLARIFIER LIME TREATMENT WITHOUT CHEMICALS

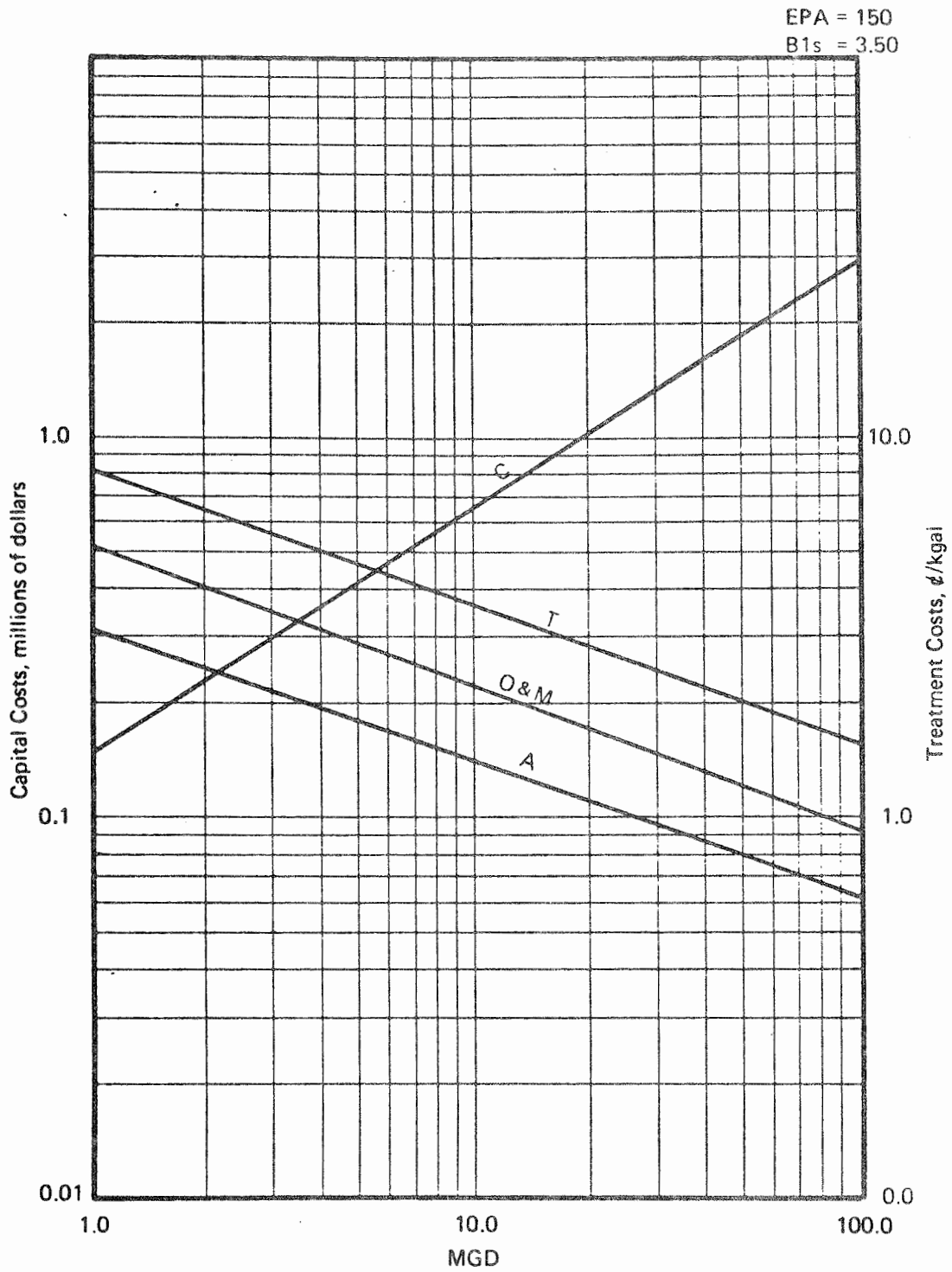
EPA = 150  
B1s = 3.50



- C = Capital Cost, millions of dollars
- A = Debt Service, ¢/1000 gal (6% - 25 yr)
- O&M = Operating and Maintenance Cost, ¢/kgal
- T = Total Treatment Cost, ¢/1000 gal

Source: Southwest Research Institute (2)

Figure E-4  
AMMONIA STRIPPING

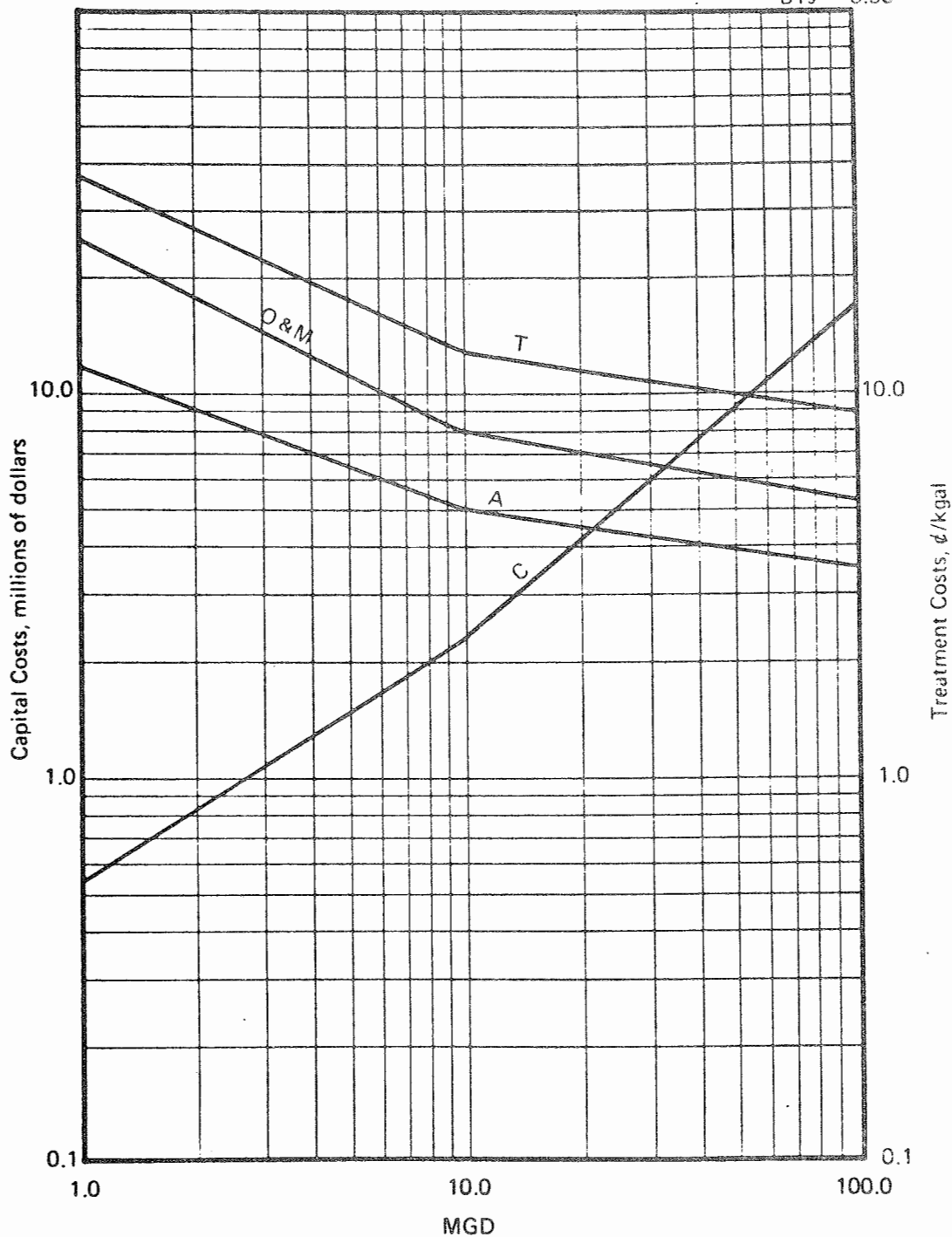


- C = Capital Cost, millions of dollars
- A = Debt Service, ¢/1000 gal (6% - 25 yr)
- O & M = Operating and Maintenance Cost, ¢/kgal
- T = Total Treatment Cost, ¢/1000 gal

Source: Southwest Research Institute (2)

Figure E-5  
RAPID FILTRATION (4 gpm/ft<sup>2</sup>)  
THROUGH SAND OR GRADED MEDIA

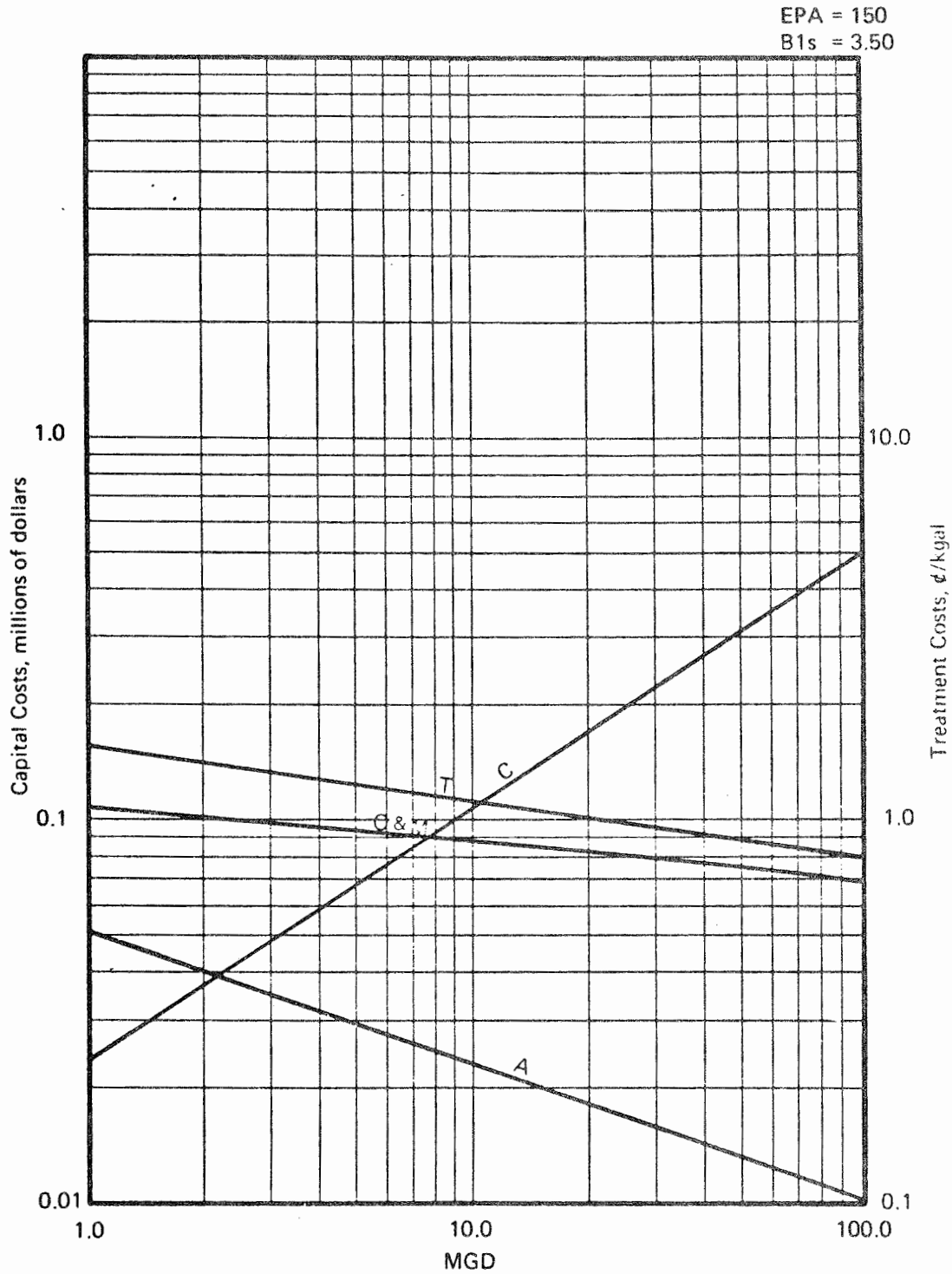
EPA = 150  
B1s = 3.50



- C = Capital Cost, millions of dollars
- A = Debt Service, ¢/1000 gal (6% - 25 yr)
- O&M = Operating and Maintenance Cost, ¢/kgal
- T = Total Treatment Cost, ¢/1000 gal

Source: Southwest Research Institute (2)

Figure E-6  
GRANULAR CARBON ADSORPTION



- C = Capital Cost, millions of dollars
- A = Debt Service, ¢/1000 gal (6% - 25 yr)
- O & M = Operating and Maintenance Cost, ¢/kgal
- T = Total Treatment Cost, ¢/1000 gal

Source: Southwest Research Institute (2)

Figure E-7  
 CHLORINATION OF SECONDARY EFFLUENT  
 (8 mg/l, 15-min contact)

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