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Design Manual

Municipal Wastewater Stabilization Ponds

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DESIGN MANUAL

MUNICIPAL WASTEWATER STABILIZATION PONDS

U.S. ENVIRONMENTAL PROTECTION AGENCY

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NOTICE

This document has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for publication. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

FOREWORD

The formation of the Environmental Protection Agency marked a new era of environmental awareness in America. This Agency's goals are national in scope and encompass broad responsibility in the areas of air and water pollution, solid wastes, pesticides, hazardous wastes, and radiation. A vital part of EPA's national pollution control effort is the constant development and dissemination of new technology.

It is now clear that only the most effective design and operation of pollution control facilities using the latest available techniques will be adequate to ensure continued protection of this Nation's natural resources. It is essential that this new technology be incorporated into the contemporary design of pollution control facilities to achieve maximum benefit from our expenditures.

The purpose of this manual is to provide the engineering community and related industry a new source of information to be used in the planning, design, and operation of present and future stabilization ponds treating municipal wastewaters. It is the intent of the manual to supplement the existing body of knowledge in this area.

This manual is one of several available from Technology Transfer to describe technological advances and present new information.

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CONTRACTOR-AUTHORS

Major Author:	Dr. E. Joe Middlebrooks, Newman Chair of Natural Resources Engineering, Clemson University
Contributing Authors:	James H. Reynolds, James M. Montgomery Consulting Engineers, Inc.
	Charlotte Middlebrooks, Middlebrooks & Associates, Inc.
	R. Wane Schneiter, Kennedy/Jenks Engineers
	Richard J. Stenquist, Brown & Caldwell Consulting Engineers
	Bruce A. Johnson, CH_2M Hill Engineers

CONTRACT SUPERVISORS

Project Officer:	Denis J. Lussier, EPA-CERI, Cincinnati, OH
Reviewers:	Edwin F. Barth, Jr., EPA-MERL, Cincinnati, OH
	Ronald F. Lewis, EPA-MERL, Cincinnati, OH
	Sherwood C. Reed, COE-CRREL, Hanover, NH
	Richard E. Thomas, EPA-OWPO, Washington, DC

TECHNICAL PEER REVIEWERS

Dr. Ernest F. Gloyna - University of Texas-Austin, Austin, TX
George W. Mann - City of Kissimmee, Kissimmee, FL
Dr. Walter J. O'Brien - Black & Veatch Engineers, Dallas, TX
Dr. A. T. Wallace - University of Idaho, Moscow, ID

Review comments were compiled and summarized by Mr. Torsten Rothman, Dynamac Corp., Rockville, MD.

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CHAPTER 1

INTRODUCTION

1.1 Background and History

Stabilization ponds have been employed for treatment of wastewater for over 3000 years. The first recorded construction of a pond system in the United States was at San Antonio, TX, in 1901. Today, almost 7000 stabilization ponds are utilized in the United States for treatment of wastewaters (1). They are used to treat a variety of wastewaters from domestic wastewater to complex industrial wastes, and they function under a wide range of weather conditions, from tropical to arctic. Ponds can be used alone or in combination with other wastewater treatment processes. As understanding of pond operating mechanisms has increased, different types of ponds have been developed for application to specific situations.

1.2 Manual Objective and Scope

This manual provides a concise overview of wastewater stabilization pond systems through discussion of factors affecting treatment, process design principles and applications, aspects of physical design and construction, suspended solids (SS) removal alternatives, and cost and energy requirements.

Chapter 2 provides a review of physical and biological factors associated with wastewater stabilization ponds. Empirical and rational design equations, and their ability to predict pond performance, are also discussed.

Actual design examples employing the rational equations presented in Chapter 2 are outlined in Chapter 3. These examples encompass essentially all types of wastewater ponds currently in use in the United States.

Physical design and construction criteria are discussed in Chapter 4. These criteria are vital to effective pond performance regardless of the design equation employed and must be considered in facility design.

High SS concentrations in pond effluents have traditionally represented a major drawback to their use. Alternatives for SS removal and control are presented in Chapter 5.

Chapter 6 contains cost and energy requirements information to aid in process selection and justification.

An evaluation of various facultative and aerated pond design methods is presented in an Appendix. This evaluation was performed using data collected on the four facultative and five aerated pond systems presented in Chapter 2.

1.3 Types of Ponds

Wastewater pond systems can be classified by dominant type of biological reaction, duration and frequency of discharge, extent of treatment ahead of the pond, or arrangement among cells (if more than one cell is used). The method used in this manual is based on that provided by Oswald (2) and is believed to be the most flexible approach to pond classification.

Ponds are classified below on the basis of dominant biological reaction, types of influent, and outflow conditions. Classification according to flow pattern (e.g., series, parallel) and the amount and type of recirculation is discussed in Chapter 4.

Table 1-1 summarizes information on pond application, loading, and size for each of the pond types discussed in this section.

1.3.1 Biological Reactions

The most basic classification involves description of the dominant biological reaction or reactions that occur in the pond. Four principal types are:

1. Facultative (aerobic-anaerobic) ponds
2. Aerated ponds
3. Aerobic ponds
4. Anaerobic ponds

1.3.1.1 Facultative Ponds

The most common type of pond is the facultative pond. Other terms which are commonly applied are oxidation pond, sewage (or wastewater treatment) lagoon, and photosynthetic pond. Facultative ponds are usually 1.2 to 2.5 m (4 to 8 ft) in depth, with an aerobic layer overlying an anaerobic layer, often containing sludge deposits. Usual detention time is 5 to 30 days. Anaerobic fermentation occurs in the lower layer and aerobic stabilization occurs in the upper layer. The key to facultative operation is oxygen production by photosynthetic algae and surface reaeration. The oxygen is utilized by the aerobic bacteria in stabilizing the organic material in the upper layer. Algae present in pond effluent represent one of the most serious performance problems associated with facultative ponds.

TABLE 1-1

WASTEWATER STABILIZATION PONDS

Pond Type	Application	Typical Loading Parameters	Typical Detention Times	Typical Dimensions	Comments
Facultative	Raw municipal wastewater Effluent from primary treatment, trickling filters, aerated ponds, or anaerobic ponds	22-67 kg BOD_5/ha/d	25-180 d	1.2-2.5 m deep 4-60 ha	Most commonly used waste stabilization pond type May be aerobic through entire depth if lightly loaded
Aerated	Industrial wastes Overloaded facultative ponds Situations where limited land area is available	8-320 kg BOD_5/1000 m^3/d	7-20 d	2-6 m deep	Use may range from a supplement of photosynthesis to an extended aeration activated sludge process Requires less land area than facultative
Aerobic	Generally used to treat effluent from other processes, produces effluent low in soluble BOD_5 and high in algae solids	85-170 kg BOD_5/ha/d	10-40 d	30-45 cm	Application limited because of effluent quality Maximizes algae production and (if algae is harvested) nutrient removal High loadings reduce land requirements
Anaerobic	Industrial wastes	160-800 kg BOD_5/1000 m^3/d	20-50 d	2.5-5 m deep	Odor production usually a problem Subsequent treatment normally required

Facultative ponds find the most widespread application. They are used for treatment of raw municipal wastewater (usually small communities) and for treatment of primary or secondary effluent (for small or large cities). They are also used, in industrial applications, following aerated ponds or anaerobic ponds to provide additional stabilization prior to discharge. The facultative pond is the easiest to operate and maintain, but there are definite limits to its performance. Effluent BOD_5 values range from 20 to 60 mg/l, and SS levels will usually range from 30 to 150 mg/l. It also requires a very large land area to maintain areal BOD_5 loadings in a suitable range. An advantage, where seasonal food processing wastes are received during summer, is that allowable organic loadings are generally much higher in summer than in winter.

The total containment pond and the controlled discharge pond are forms of facultative ponds. The total containment pond is applicable in climates where the evaporative losses exceed the rainfall. Controlled discharge ponds have long hydraulic detention times and the effluent is discharged once or twice per year when the effluent quality is satisfactory.

1.3.1.2 Aerated Ponds

In an aerated pond, oxygen is supplied mainly through mechanical or diffused air aeration rather than by photosynthesis and surface reaeration. Many aerated ponds have evolved from overloaded facultative ponds that required aerator installation to increase oxygenation capacity. Aerated ponds are generally 2 to 6 m (6 to 20 ft) in depth with detention times of 3 to 10 days. The chief advantage of aerated ponds is that they require less land area.

In some cases, both photosynthesis and mechanical aeration can be effective in providing oxygen. At Sunnyvale, CA, for example, mechanical cage-type aerators were installed at the effluent points to eliminate local anaerobic conditions during seasonal increased loads from a canning plant. Photosynthesis and surface reaeration provide the necessary oxygen in the remaining areas of the pond (3).

Aerated ponds can also be classified by the amount of mixing provided. If energy input is sufficient to keep all solids in suspension, and if secondary clarification with sludge return is utilized, the system approaches an activated sludge process with the associated high BOD_5 and SS removal. Power costs for this system become very high, however, and operation and maintenance complexity increases.

Aerated ponds are used in both municipal and industrial wastewater treatment applications. For the former situation, they are often resorted to when an existing facultative system becomes overloaded and there is minimal land available for expansion. For industrial wastes, they are sometimes used as a pretreatment step before discharge to a municipal sewerage system. In both municipal and industrial applications, aerated ponds may be followed by facultative ponds.

1.3.1.3 Aerobic Ponds

Aerobic ponds, also called high rate aerobic ponds, maintain dissolved oxygen (DO) throughout their entire depth. They are usually 30 to 45 cm (12 to 18 in) deep, allowing light to penetrate the full depth. Mixing is often provided to expose all algae to sunlight and to prevent deposition and subsequent anaerobic conditions. Oxygen is provided by photosynthesis and surface reaeration, and aerobic bacteria stabilize the waste. Detention time is short, three to five days being usual.

High-rate aerobic ponds are limited to warm, sunny climates. They are used where a high degree of BOD_5 removal is desired but land area is limited. The chief advantage of the high-rate aerobic pond is that it produces a stable effluent with low land and energy requirements and short detention times. However, operation is somewhat more complex than for a facultative pond and, unless an algae removal step is provided, the effluent will contain high SS. Short detention times also mean that very little coliform die-off will result. Because of their shallow depths, paving or covering the bottom is required to prevent weed growth.

1.3.1.4 Anaerobic Ponds

Anaerobic ponds receive such a heavy organic loading that there is no aerobic zone. They are usually 2.5 to 5 m (8 to 15 ft) in depth and have detention times of 20 to 50 days. The principal biological reactions occurring are acid formation and methane fermentation. The smaller of the two Sunnyvale, CA, cells was originally an anaerobic cell providing treatment of seasonal cannery wastes. Effluent from the anaerobic cell enters the larger and shallower facultative cell.

Anaerobic ponds are usually used for treatment of strong industrial and agricultural wastes, or as a pretreatment step where an industry is a significant contributor to a municipal system. Because they do not have wide application to the treatment of municipal wastewaters, they are not discussed further in this manual.

An important disadvantage to anaerobic ponds is the production of odorous compounds. Sodium nitrate has been used to combat odors, but it is quite expensive and in some cases has not proven effective. Another common approach is to recirculate water from a downstream facultative or aerobic pond to maintain a thin aerobic layer at the surface of the anaerobic pond, preventing transfer of odors to the air. Crusts have also proven effective, either naturally formed as with grease, or formed from styrofoam balls. A further disadvantage of the anerobic pond is that the effluent must usually be given further treatment prior to discharge.

1.3.2 Pretreatment

Ponds can also be characterized by the degree of pretreatment which the wastewater receives before discharge into a pond system:

1. None. Ponds receiving raw untreated wastewater direct from the municipal sewer are often used by small communities to avoid the added expense of pretreatment. Care must be taken to ensure that odors do not occur from anaerobic conditions, mats of rising sludge near the pond inlet, or from greasy scum at the shoreline.

2. Screening. Screening or comminution may be employed; however, this is not common practice.

3. Primary sedimentation. Where primary sedimentation is used, the pond provides a form of secondary treatment, usually at a much lower cost than other forms of biological treatment. The Davis, CA, pond system is an example of sedimentation before pond treatment.

4. Secondary treatment. Ponds receiving secondary effluent are normally associated with trickling filter and activated sludge effluents. Effluent BOD_5 concentrations from high-rate rock media trickling filters may be 40 to 75 mg/l, making ponds practical for further removing and stabilizing the organic material. Ponds can also be considered for use after trickling filters when loading increases or discharge requirements are tightened. Activated sludge and trickling filter effluents can be further treated by ponds for nutrient removal.

1.3.3 Discharge Conditions

Ponds may also be classified on the basis of discharge conditions:

1. Complete retention. These systems rely on evaporation and/or percolation to reduce the liquid volume at a rate equal to or greater than the influent accumulation. Favorable geologic and/or climatic conditions are prerequisite.

2. Controlled discharge. These systems have long hydraulic detention times, and effluent is discharged when receiving water quality will not be adversely affected by the discharge. Controlled discharge ponds are designed to hold the wastewater until the effluent and receiving water quality arecompatible.

3. Continuous discharge. These systems have no provision for regulating effluent flow and the discharge rate essentially equals the influent rate.

1.4 Nutrient Removal Aspects

Both nitrogen and phosphorus in wastewater are affected by passage through waste stabilization ponds. Nitrogen can undergo a number of chemical and physical processes, including settling (in organic particulate form), assimilation into algae cells, ammonification (conversion of organic nitrogen to ammonia), nitrification, and denitrification. Phosphorus is removed by assimilation into algae cells and by precipitation. When the alkalinity increases during the daylight hours, phosphate is precipitated and will settle out of the wastewater. A reduction in alkalinity at night can result in some of the phosphorus being dissolved from the sediment. In general, the pond effluent phosphorus concentration is less than half of the influent wastewater concentration.

Nutrient removal can be accomplished in ponds using water hyacinths, duckweed and other plants. Details on the use of aquaculture in wastewater treatment are presented in Chapter 5 and elsewhere (4).

1.5 References

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CHAPTER 2

PROCESS THEORY, PERFORMANCE, AND DESIGN

2.1 Biology

Enumeration of types of organisms occurring in wastewater ponds possesses some inherent limitations. Species identification reflects the selective nature of the isolation method, as well as the particular interests of the researcher. Also, seasonal changes in pond operation and influent wastewater characteristics produce variations in the microbial populations. Despite these limitations, general observations pertaining to macro- and micro-organisms in wastewater ponds are valuable in understanding the process theory.

2.1.1 Bacteria

2.1.1.1 Aerobic Bacteria

Bacteria found in an aerobic zone of a wastewater pond are primarily the same type as those found in an activated sludge process or in the zoogleal mass of a trickling filter. The most frequently isolated bacteria include *Beggiatoa alba*, *Sphaerotilus natans*, *Achromobacter*, *Alcaligenes*, *Flavobacterium*, *Pseudomonas*, and *Zoogloea* spp. (1) These organisms decompose the organic materials present in the aerobic zone into oxidized end products.

2.1.1.2 Acid Forming Bacteria

Acid forming bacteria are heterotrophs that convert complex organic material into simple alcohols and acids. These acids are primarily acetic, propionic, and butyric (2). The activity of these bacteria is important since they provide one of the substrates for the final reduction of the organic material into methane gas by the methanogenic bacteria. Acid forming bacteria do not limit the rate of the anaerobic decomposition and do not require thermophilic temperatures for optimum growth as do the methanogenic bacteria. Additionally, the acid forming bacteria are able to maintain a near-optimum pH range for their existence through their own acid production.

2.1.1.3 Cyanobacteria

Cyanobacteria are commonly known as blue-green algae. Like algae, cyanobacteria are able to assimilate simple organic compounds while utilizing carbon dioxide as the major carbon source, or to grow in a completely inorganic medium. Cyanobacteria produce oxygen as a by-product of photosynthesis, thus providing an oxygen source for other organisms in the ponds. Ability of the cyanobacteria to utilize atmospheric nitrogen accounts for very broad distribution in both the terrestrial and aquatic environment. Cyanobacteria appear in very large numbers as blooms when environmental conditions are suitable (3).

2.1.1.4 Purple Sulfur Bacteria

Many species of Chromatiaceae, the purple sulfur bacteria, are actually purple, but others may be dark orange to brown or various shades of pink or red. Purple sulfur bacteria may grow in any aquatic environment to which light of the required wavelength penetrates, provided that carbon dioxide, nitrogen, and a reduced form of sulfur, or hydrogen, are available. Purple sulfur bacteria occupy the anaerobic layer below the algae, cyanobacteria, and other aerobic bacteria in a pond. Wavelengths of light used by the purple sulfur bacteria are different from those used by the cyanobacteria or algae; thus the sulfur bacteria are able to grow using light that has passed through the surface layer of water or sediment occupied by aerobic photosynthetic organisms. Purple sulfur bacteria are commonly found at a specific depth, in a thin layer where light and nutrition conditions are optimum (3). Conversion of odorous sulfide compounds to elemental sulfur or sulfate by the sulfur bacteria is a significant factor for odor control in facultative and anaerobic ponds.

2.1.1.5 Pathogenic Bacteria

Pathogenic bacteria frequently discussed with reference to wastewater ponds include *Salmonella*, *Shigella*, *Escherichia*, *Leptospira*, *Francisella*, and *Vibrio*. (Viruses and certain protozoa are also pathogenic, but scant information exists.) Water is not the natural environment of the pathogenic bacteria, but instead a means of transport to a new host. Pathogenic bacteria are usually unable to multiply or survive for extensive periods in the aquatic environment. The decline in numbers of microbial pathogens with time, within the aquatic environment, involves sedimentation, starvation, sunlight, pH, temperature, competition, and predation (1).

The most probable number (MPN) of coliform organisms present in a given volume of a waste is the commonly accepted index of the pathogenic quality of an effluent. Most literature concerning the performance of ponds report very high reductions in coliform bacteria, often as high as 99.9 percent, in facultative ponds. Caution should be exercised in interpreting these figures, however, as large absolute numbers of coliform may still be present.

2.1.2 Algae

Algae constitute a group of aquatic organisms that may be unicellular or multicellular, motile or immotile, of which practically all have photosynthetic pigments. Being autotrophic, algae utilize inorganic nutrients such as phosphate, carbon dioxide, and nitrogen. Algae do not fix atmospheric nitrogen, but require inorganic nitrogen in the form of nitrate or ammonia. Also, some algal species are able to use amino acids and other organic nitrogen compounds. Inorganic nutrients utilized by algae are photosynthetically converted into cellular organic materials, with oxygen produced as a by-product.

Algae are generally divided into three major groups, based on the color imparted to the cells by the chlorophyll and other pigments involved in photosynthesis. Green algae include unicellular, filamentous, and colonial forms. Brown algae are unicellular and flagellated, and include the diatoms. Certain brown algae are responsible for toxic red blooms. Red algae include a few unicellular forms, but are primarily filamentous (3). Green and brown algae are common to wastewater ponds, with the red algae occurring infrequently. The predominant algal species at any given time is thought to be primarily a function of temperature, although the effects of predation, nutrient availability, and toxins are also important.

It has been generally accepted that algae and bacteria together comprise the essential elements of a successful stabilization pond operation. Bacteria break down the complex organic waste components aerobically and anaerobically into simple products which are then available for use by the algae. Algae, in turn, produce the oxygen necessary for maintaining the aerobic environment necessary for the bacteria to perform oxidative functions.

Due to the cyclic biochemical reactions of biodegradation and mineralization of nutrients by bacteria, and synthesis of new organics in the form of algae cells, it is feasible that a pond effluent could contain a higher total organic content than the influent. However, this could occur only under the most optimum algae growth conditions, which would seldom occur in practice.

2.1.3 Animals

Although bacteria and algae are the primary organisms through which waste stabilization is accomplished, higher life forms are of importance as well. Planktonic Cladocera and benthic Chironomidae have been suggested as the most significant fauna in the pond community, in terms of stabilizing organic matter. The Cladocera feed on the algae and promote flocculation and settling of particulate matter. This in turn results in better light penetration and algal growth at greater depths. Settled matter is further broken down and stabilized by the benthic feeding Chironomidae. Predators, such as rotifers, often control the population levels of certain of the lower forms in the pond, thereby influencing the succession of predominating species.

Mosquitoes do present a problem in some ponds. Aside from their nuisance characteristics, certain mosquitoes are also the vector for such diseases as encephalitis, malaria, and yellow fever, and constitute a hazard to public health which must be controlled if ponds are to be utilized. The most effective means of control is the control of emergent vegetation. Gambusia, or mosquito fish, have beeen successfully employed to eliminate mosquito problems in some ponds in warm climates (4)(5).

2.2 Biochemical Interactions

2.2.1 Photosynthesis

Photosynthesis is the process whereby organisms are able to grow utilizing the sun's radiant energy to power the fixation of atmospheric CO_2 and subsequently provide the reducing power to convert the CO_2 to organic compounds. Photosynthesis is usually associated with the green plants; however, certain bacteria as well as algae carry out photosynthesis. In wastewater ponds, the photosynthetic organisms of interest are the algae, cyanobacteria (blue-green algae), and the purple sulfur bacteria (5).

Photosynthesis may be classified as oxygenic or anoxygenic depending on the source of reducing power used by a particular organism. In oxygenic photosynthesis, water serves as the source of reducing power, with oxygen being produced as a by-product. The equation representing oxygenic photosynthesis is:

$$H_2O \xrightarrow{\text{sunlight}} \frac{1}{2} O_2 + 2H^+ + 2e^- \qquad (2\text{-}1)$$

Oxygenic photosynthesis occurs in green plants, algae, and cyanobacteria. In ponds, the oxygenic photosynthetic algae and cyanobacteria convert carbon dioxide to organic compounds that serve as a source of chemical energy, in addition to organic waste matter, for most other living organisms (3). More importantly, the by-product oxygen produced in oxygenic photosynthesis allows the aerobic bacteria to function in their role as primary consumers in degrading complex organic waste material.

Anoxygenic photosynthesis produces no oxygen as a by-product, thus occurring in the complete absence of oxygen. The bacteria involved in anoxygenic photosynthesis are largely strict anaerobes, with reducing power supplied by reduced inorganic compounds. Many anoxygenic photosynthetic bacteria utilize reduced sulfur compounds or elemental sulfur in anoxygenic photosynthesis according to the following equation:

$$H_2S \rightarrow S^o + 2H^+ + 2e^- \tag{2-2}$$

2.2.2 Respiration

Respiration is a physiological process in which organic compounds are oxidized mainly to carbon dioxide and water. However, respiration does not only lead to the production of carbon dioxide, but to the synthesis of cell material as well. Respiration is an orderly process, catalyzed by enzymes such as the cytochromes and consisting of many integrated step reactions terminating in the reduction of oxygen to water (6). Aerobic respiration, common to species of bacteria, protozoa, and higher animals, may be represented by the following simple equation:

$$C_2H_{12}O_6 + 6O_2 \xrightarrow{\text{enzymes}} 6CO_2 + 6H_2O + \text{new cells} \tag{2-3}$$

The bacteria involved in aerobic respiration are primarily responsible for wastewater stabilization in ponds.

In the presence of light, respiration and photosynthesis can occur simultaneously in algae. However, the respiration rate is low compared with the photosynthesis rate, resulting in a net consumption of carbon dioxide and production of oxygen. In the absence of light, algal respiration continues while photosynthesis stops, resulting in a net consumption of oxygen and production of carbon dioxide.

2.2.3 Dissolved Oxygen (DO)

Oxygen is a partially soluble gas, and solubility varies in direct proportion to the atmospheric pressure at any given temperature, and in inverse proportion to temperature for any given atmospheric pressure. DO concentrations of approximately 8 mg/l are usually considered the maximum available under ambient conditions. In mechanically aerated ponds, the limited solubility of oxygen is a significant factor, since it determines the oxygen absorption rate and, therefore, the cost of aeration (7).

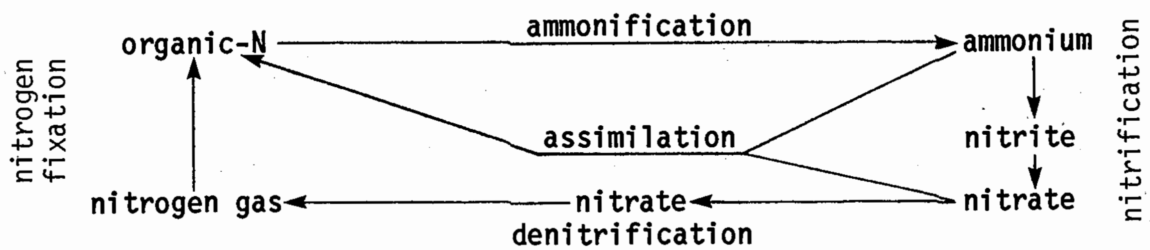
The two natural sources of DO in ponds are surface reaeration and photosynthetic oxygenation. In areas of low wind activity, surface reaeration may be relatively unimportant, depending on the water depth. Where surface turbulence is created from excessive wind activity, surface reaeration is significant. Observation has shown that DO in wastewater ponds varies almost directly with the level of photosynthetic activity, being low at night and early morning and rising during daylight hours to a peak in the early afternoon. At increased depth, the effects of surface reaeration and photosynthetic oxygenation decrease, since the distance from the water-atmosphere interface increases and light penetration decreases. This can result in a vertical gradient in DO concentration accompanied by a segregation of microorganisms.

2.2.4 Nitrogen Cycle

A simplified nitrogen cycle is represented by Figure 2-1.

FIGURE 2-1

THE NITROGEN CYCLE



In a wastewater pond, organic nitrogen and ammonium nitrogen enter with the influent wastewater. Organic nitrogen in fecal matter and other organic materials in the wastewater undergo conversion to ammonia and ammonium ion by microbial activity. The ammonium in turn is nitrified to nitrite by Nitrosomonas and then to nitrate by Nitrobacter. The overall nitrification reaction is:

$$NH_4^+ + 2O_2 \rightarrow NO_3^- + 2H^+ + H_2O \qquad (2\text{-}4)$$

The nitrate produced in the nitrification process, as well as a portion of the ammonium produced from ammonification, can be assimilated by organisms as a nutrient to produce cell protein and other nitrogen-containing compounds. The nitrate is utilized in denitrification to form nitrite and then nitrogen gas. Several bacteria may be involved in the denitrification process including Pseudomonas, Micrococcus, Achromobacter, and Bacillus. The overall denitrification reaction is:

$$6NO_3^- + 5CH_3OH \rightarrow 3N_2 + 5CO_2 + 7H_2O + 6OH^- \qquad (2\text{-}5)$$

Nitrogen gas is "fixed" to form organic nitrogen by cyanobacteria, thus making the nitrogen available as a nutrient and completing the cycle (8).

Nitrogen removal in facultative wastewater ponds can occur by any of the following processes: (1) gaseous ammonia stripping to the atmosphere, (2) ammonium assimilation in algal biomass, (3) nitrate uptake by plants and algae, and (4) biological nitrification-denitrification. Ammonium assimilation into algal biomass depends on the biological activity in the system and is affected by several factors such as temperature, organic load, detention time, and wastewater characteristics. The rate of gaseous ammonia losses to the atmosphere is primarily a function of pH, surface to volume ratio, temperature, and the mixing conditions. An alkaline pH shifts the equilibrium of ammonia gas and ammonium ion towards gaseous ammonia production, while the mixing conditions affect the magnitude of the mass transfer coefficient.

2.2.5 pH and Alkalinity

In wastewater ponds, the hydrogen ion concentration, expressed as pH, is controlled through the carbonate buffering system represented by the following equations:

$$CO_2 + H_2O \rightleftarrows H_2CO_3 \rightleftarrows HCO_3^- + H^+ \qquad (2\text{-}6)$$

$$M(HCO_3)_2 \rightleftarrows M^{++} + 2HCO_3^- \qquad (2\text{-}7)$$

$$HCO_3^- \rightleftarrows CO_3^= + H^+ \qquad (2\text{-}8)$$

where: M = metal ion

$$CO_3^= + H_2O \rightleftarrows HCO_3^- + OH^- \qquad (2\text{-}9)$$

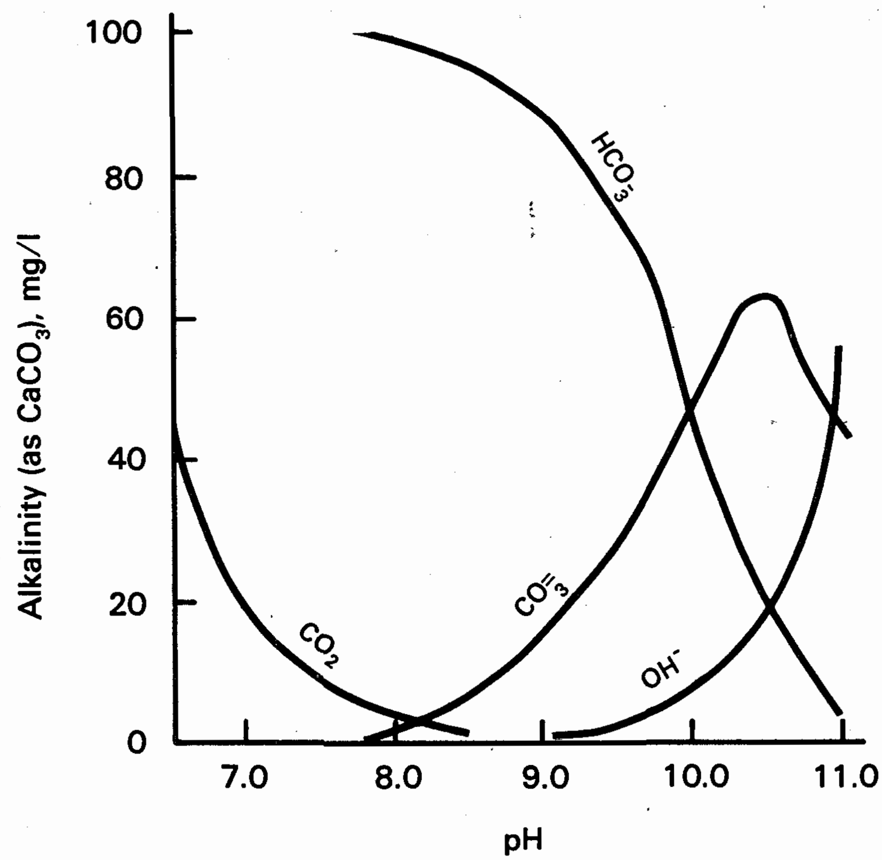
$$OH^- + H^+ \rightleftarrows H_2O \qquad (2\text{-}10)$$

The equilibrium of this system is affected by algal photosynthesis. In photosynthetic metabolism by algae, carbon dioxide is removed from the dissolved phase, forcing the equilibrium of the first expression to the left. This tends to decrease the hydrogen ion concentration and also decrease bicarbonate alkalinity. The effect of the resultant decrease in bicarbonate (HCO_3^-) concentration is to force the third equation to the left and the fourth to the right, both of which decrease total alkalinity. Figure 2-2 shows a typical relationship between pH, CO_2, HCO_3^-, $CO^=$, and OH^-.

The decreased alkalinity associated with photosynthesis will simultaneously reduce the carbonate hardness present in the waste. Because of the close relationship between pH and photosynthetic activity, considerable diurnal fluctuation in pH is observed.

FIGURE 2-2

CALCULATED RELATIONSHIP AMONG pH, CO_2, $CO_3^=$, HCO_3^-, and OH^- (7)



2.3 Controlling Factors

2.3.1 Light

The intensity and spectral composition of light penetrating a pond surface significantly affects all resident microbial activity. The available light determines, to a large degree, the level of photosynthetic activity and, hence, oxygen production. Oxygen availability to the aerobic bacterial organisms is vital. In general, photosynthetic activity increases with increasing light intensity until the photosynthetic system becomes light saturated. The rate at which photosynthesis increases in proportion to an increase in light intensity, as well as the level at which an organism's photosynthetic system becomes light saturated, depends upon the particular biochemistry of the species (1). In ponds, photosynthetic oxygen production has been shown to be relatively constant within the range of 5,380 to 53,800 lumen m^2 (500 to 5,000 foot-candles) light intensity with a reduction occurring at higher and lower intensities (5).

The spectral composition of available light is also crucial in determining photosynthetic activity. The ability of photosynthetic organisms to utilize available light energy primarily depends upon their ability to absorb the available wavelengths. This absorption ability is determined by the specific photosynthetic pigment of the organism. The main photosynthetic pigments are the chlorophylls and the phycobilins. Bacterial chlorophyll differs from algal chlorophyll in both chemical structure and absorption capacity. These differences allow the photosynthetic bacteria to live below dense algal layers where they can utilize light not absorbed by the algae (1).

The quality and quantity of light penetrating the pond surface to any depth depends on the presence of dissolved and particulate matter as well as the water absorption characteristics. The organisms themselves contribute to water turbidity, further limiting the depth of light penetration. Because of these restrictions, photosynthesis is significant only in the upper pond layers. This region of net photosynthetic activity is called the euphotic zone (1).

Light intensity from solar radiation varies with the time of day and difference in latitudes. In cold climates, light penetration can be drastically reduced by ice and snow cover--thus the prevalence of mechanical aeration in these regions.

2.3.2 Temperature

Temperature is a very important factor in the aerobic environment of a pond and will, at or near the surface, determine the succession of predominant species of algae, bacteria, and other aquatic organisms. Algae can survive at temperatures of 5°C to 40°C. Green algae show most efficient growth and activity at temperatures near 30°C to 35°C. Aerobic bacteria are viable within the temperature range of 10°C to 40°C, with 35°C to 40°C being optimum for cyanobacteria (10)(11).

Solar radiation is a major source of heat, generally resulting in a temperature gradient with respect to depth. This can influence the anaerobic decomposition of solids settled to the bottom of the pond. The bacteria responsible for anaerobic degradation, ideally requiring temperatures within the range of 15°C to 65°C, are exposed to the lowest temperatures, thus greatly reducing their activity. The other major source of heat is the influent. In sewerage systems having no major inflow or infiltration problems, the influent temperature is higher than that of the pond contents. Cooling influences are exerted by evaporation, contact with cooler groundwater, and wind action.

Overall effects of temperature, combined with light intensity, are reflected in the fact that nearly all investigators report improved performance during summer and autumn months when both temperature and light are at their maximum. The maximum practical temperature of wastewater ponds is likely less than 30°C, indicating that most ponds operate at less than optimum temperature for anaerobic activity (12).

Temperature changes in nonaerated ponds result in vertical stratification during certain seasons of the year. Stratification results because of an increase in water density with depth caused by a decrease in temperature. During the summer, the upper waters are warmed and the density decreased, and stratification results. The temperature of the upper layer of water is relatively uniform because of mixing by the wind. Temperatures change rapidly in the thermocline, and the zone is very resistant to mixing. As temperatures decrease during the fall, stratification is decreased and the pond is mixed by wind action. This phenomenon is referred to as the fall overturn.

The density of water decreases as the temperature falls below 4°C, and a winter stratification can occur. As ice cover breaks up and the water warms, a spring overturn can also occur.

During both the spring and fall overturns, significant odors caused by anaerobic material being brought to the surface can stimulate complaints from neighbors. Overturn phenomena are the reasons for regulatory requirements that nonaerated ponds be located downwind (based on prevailing winds during overturn periods) and away from dwellings.

2.3.3 Nutrient Requirements and Removal

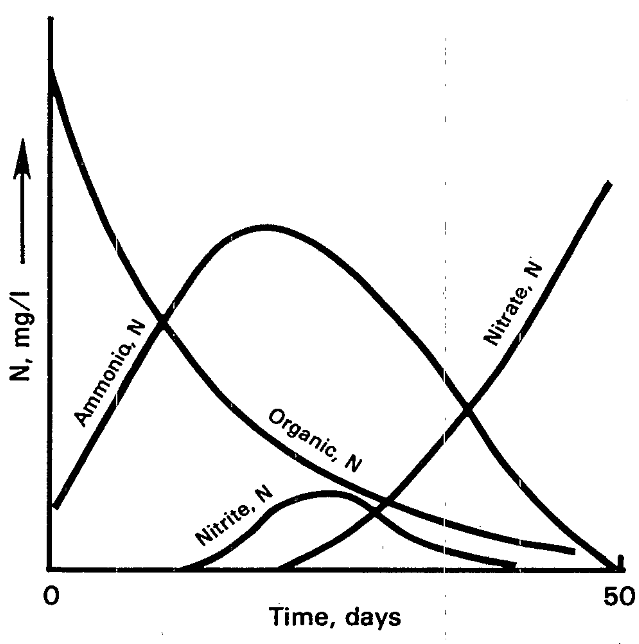
Growth and, to some extent, activity of microorganisms is controlled by the availability of essential nutrients such as carbon, nitrogen, phosphorus, and sulfur and a variety of other substances required in small quantities. These nutrients may be classified as inorganic or organic. Nitrogen, phosphorus, and sulfur represent the inorganic nutrients, while organic carbon compounds represent the organic nutrients.

2.3.3.1 Nitrogen

Nitrogen can be a limiting nutrient for primary productivity involving algae. Figure 2-3 represents the various forms that nitrogen typically assumes over time in wastewater ponds. The conversion of organic nitrogen to various other nitrogen forms results in a net loss in total nitrogen (13). This nitrogen loss may be due to either algal uptake for metabolic purposes or to bacterial action. It is likely that each mechanism contributes to the overall total nitrogen reduction. As previously discussed, another factor contributing to the reduction of total nitrogen is gaseous ammonia stripping under favorable environmental conditions. Regardless of the specific removal mechanism involved, ammonia removal in facultative wastewater ponds may approach 99 percent, with the major removal occurring in the primary cell of a multicell pond system (9).

FIGURE 2-3

CHANGES OCCURRING IN FORMS OF NITROGEN PRESENT IN A POND ENVIRONMENT UNDER AEROBIC CONDITIONS (7)



2.3.3.2 Phosphorus

Phosphorus is most often the growth-limiting nutrient in aquatic environments. Municipal wastewater, however, is normally quite rich in phosphorus even though current restrictions on phosphorus compounds in detergents has resulted in reduced concentrations. Nonetheless, the concentration is still adequate to stimulate growth in aquatic organisms.

In aquatic environments, phosphorus occurs in three forms: (1) particulate phosphorus, (2) soluble organic phosphorus, and (3) inorganic phosphorus. Inorganic phosphorus, primarily in the form of orthophosphate, is readily utilized by aquatic organisms. Some organisms may store excess phosphorus as polyphosphates for future use. At the same time, some phosphate is continuously lost to sediments, where it is locked up in insoluble precipitates (1).

Phosphorus removal in ponds may result from physical mechanisms such as adsorption, coagulation, and precipitation. The uptake of phosphorus by organisms in metabolic functions as well as for storage can also add to phosphorus removal. Phosphorus removal in wastewater ponds has been reported to range from 30 to 95 percent (13).

Like nitrogen, phosphorus held by algae discharged in the final effluent may be introduced to receiving waters as organic phosphorus. Excessive algal "afterblooms" observed in waters receiving effluents have, in some cases, been attributed to nitrogen and phosphorus compounds remaining in the treated wastewater. Phosphorus removal in aquaculture is discussed in Chapter 5.

2.3.3.3 Sulfur

Sulfur is a vital nutrient for microorganisms, but is usually plentiful in natural waters. Because sulfur is rarely limiting, its removal from wastewater is usually not considered necessary, Ecologically, sulfur is particularly important since compounds such as hydrogen sulfide and sulfuric acid are toxic, and since the oxidation of certain sulfur compounds is an important energy source for some aquatic bacteria (1).

2.3.3.4 Carbon

The decomposable organic carbon content of a waste is traditionally measured in terms of its biochemical oxygen demand (BOD), or the amount of oxygen required under standardized conditions for the aerobic biological stabilization of the organic matter in a waste. Since the time required for complete stabilization by biological oxidation, depending on the organic material and the organisms present, can be several weeks, the standard practice is to use the five-day biochemical oxygen demand (BOD_5) as an index of the organic carbon content or organic strength of a waste. The removal of BOD_5 is also a primary criterion for evaluating treatment efficiency.

BOD_5 reductions in wastewater ponds ranging from 50 to 95 percent have been reported in the literature. Factors affecting the reduction of BOD_5 are numerous. A very rapid reduction in BOD_5 occurs in a wastewater pond during the first five to seven days. Later reductions take place at a sharply reduced rate. BOD_5 removals are generally much lower during winter and early spring than in summer and early fall. This is due primarily to lower temperatures during these periods. Many regulatory agencies recommend that ponds be so operated as to prevent discharge during cold periods.

2.4 Performance and Design of Ponds

2.4.1 Facultative Ponds

Performance results from existing pond systems located at Peterborough, NH; Kilmichael, MS; Eudora, KS; and Corinne, UT, are presented in this section (14-17). These studies encompassed 12 full months of data collection, including four separate 30-consecutive-day, 24-hour composite sampling periods, once each season.

2.4.1.1 Site Description

a. Peterborough, New Hampshire

The Peterborough facultative waste stabilization pond system consists of three cells operated in series during the evaluation of the performance described in this chapter. The option to operate the cells in parallel or combination of series and parallel is available. The total surface area is 8.5 ha (21 ac) and the effluent is chlorinated. A schematic drawing of the facility is shown in Figure 2-4. An effluent chlorine residual of 2.0 mg/l is maintained at all times. The facility was designed in 1968 on an areal loading basis of 20 kg BOD_5/d/ha (18 lb BOD_5/d/ac) with an initial average hydraulic flow of 1,890 m^3/d (0.5 mgd). At the design depth of 1.2 m (4 ft), the theoretical hydraulic detention time for the system would be 57 days. The results of the study conducted during 1974-1975 indicated an actual mean area loading of 15 kg BOD_5/d/ha (14 lb BOD_5/d/ac) and a mean hydraulic flow of 1,011 m^3/d (0.27 mgd). The theoretical hydraulic detention time based upon the flow entering the plant was 107 days. The loading rates and detention times for the first cell in the series are shown in Table 2-1.

FIGURE 2-4

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE FACULTATIVE POND SYSTEM AT PETERBOROUGH, NEW HAMPSHIRE

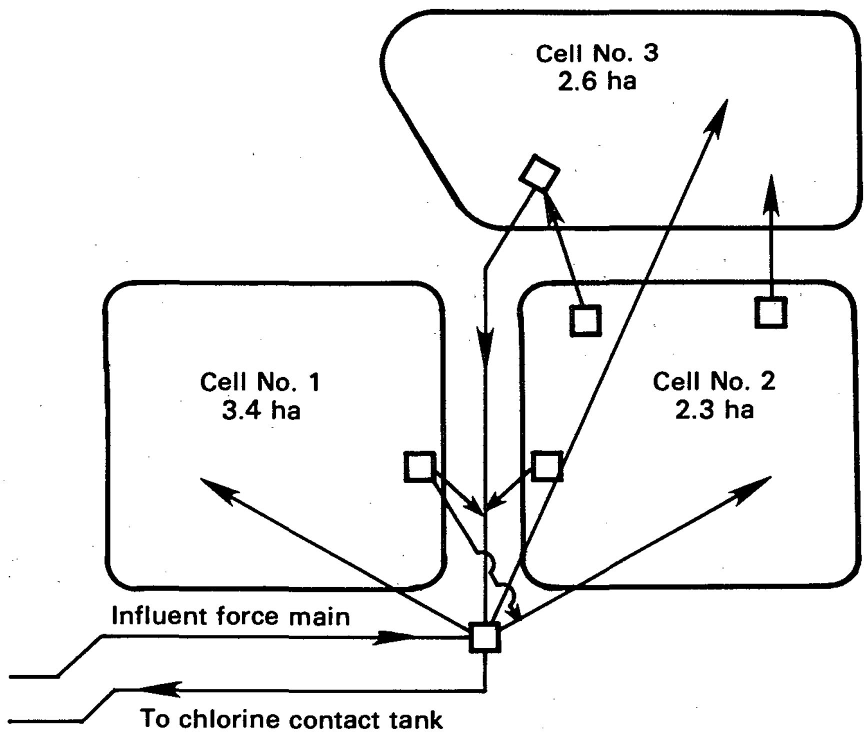


TABLE 2-1

DESIGN AND ACTUAL LOADING RATES AND DETENTION TIMES FOR SELECTED FACULTATIVE PONDS

Location	Organic Loading Rate			Theoretical Hydraulic Detention Time	
	Design	Actual		Design	Actual
		Total System	First Cell		
	kg BOD_5/ha/d			days	
Peterborough, NH	20	15	36	57	107
Kilmichael, MS	67[a]	15	23	79	214
Eudora, KS	38	17	43	47	231
Corinne, UT	36	12	30	180	70

[a]First cell.

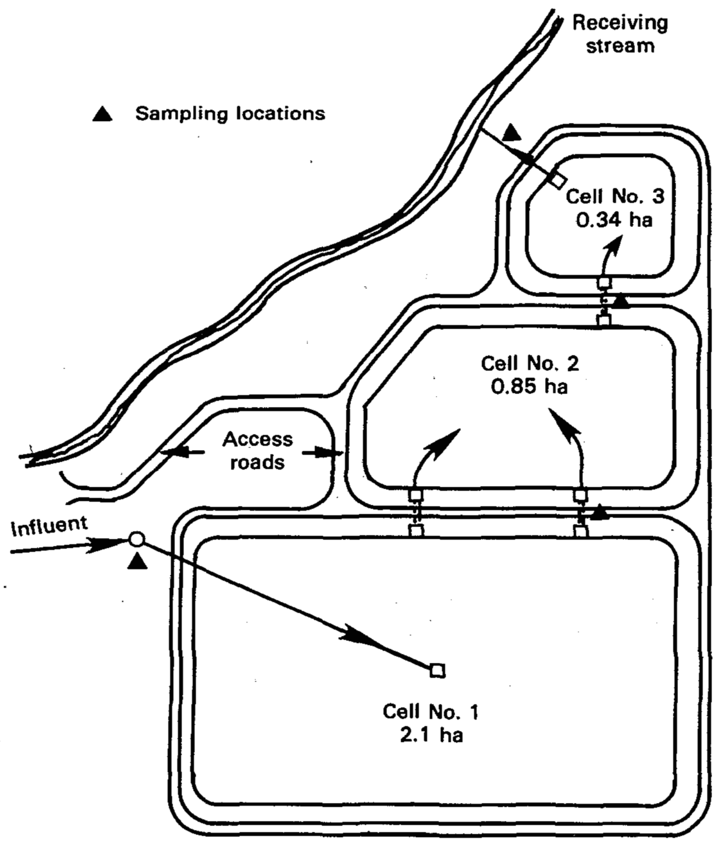
b. Kilmichael, Mississippi

The Kilmichael facultative waste stabilization pond system consists of three cells operated in series with a total surface area of 3.3 ha (8.1 ac). The effluent is not chlorinated. A schematic drawing of the facility is shown in Figure 2-5.

The design load as specified by the State of Mississippi standards for the first cell in the series was 67 kg BOD_5/d/ha (60 lb BOD_5/d/ac). The second cell was designed with a surface area equivalent to 40 percent of the surface area of the first cell. The third cell was designed with a surface area equivalent to 16 percent of the first cell. The system was designed for a hydraulic flow of 690 m^3/d (0.18 mgd). The average depth of the cells is approximately 2 m (6.6 ft). This provides for a theoretical hydraulic detention time of 79 days. The result of the study indicated that the actual average organic load on the first cell averaged 23 kg BOD_5/d/ha (21 lb BOD_5/d/ac) and that the average hydraulic inflow to the system was 280 m^3/d (0.07 mgd). The theoretical hydraulic detention time based upon the flow entering the plant was 214 days. Loading rates and detention times are summarized in Table 2-1.

FIGURE 2-5

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE FACULTATIVE POND SYSTEM AT KILMICHAEL, MISSISSIPPI



c. Eudora, Kansas

The Eudora facultative waste stabilization pond system consists of three cells operated in series with a total surface area of 7.8 ha (19.3 ac). A schematic diagram of the system is shown in Figure 2-6. The effluent is not chlorinated.

The facility was designed on an areal loading basis of 38 kg BOD_5/d/ha (34 lb BOD_5/d/ac) with a hydraulic flow of 1,510 m^3/d (0.4 mgd). At the design operating depth of 1.5 m (5 ft), the theoretical hydraulic detention time would be 47 days. The results of the study indicated that the actual mean organic load on the system was 17 kg BOD_5/d/ha (15 lb BOD_5/d/ac). The actual mean hydraulic flow to the system was 500 m^3/d (0.13 mgd), and theoretical hydraulic detention time in the system was 231 days. A summary of the loading rates and detention times are shown in Table 2-1.

d. Corinne, Utah

The Corinne facultative waste stabilization pond system consists of seven cells operated in series with a total surface area of 3.8 ha (9.5 ac). A schematic drawing of the system is shown in Figure 2-7. The effluent is not chlorinated.

The facility was designed on an areal loading basis of 36 kg BOD_5/d/ha (32 lb BOD_5/d/ac) with a hydraulic flow of 265 m^3/d (0.07 mgd).

With a design depth of 1.2 m (4 ft), the system has a theoretical hydraulic detention time of 180 days. The results of the study indicated that the actual mean organic load on the system was 12 kg BOD_5/d/ha (11 lb BOD_5/d/ac). The actual average hydraulic flow to the system was 690 m^3/d (0.18 mgd), and the theoretical hydraulic detention time in the system was 70 days. Loading rates and detention times are summarized in Table 2-1.

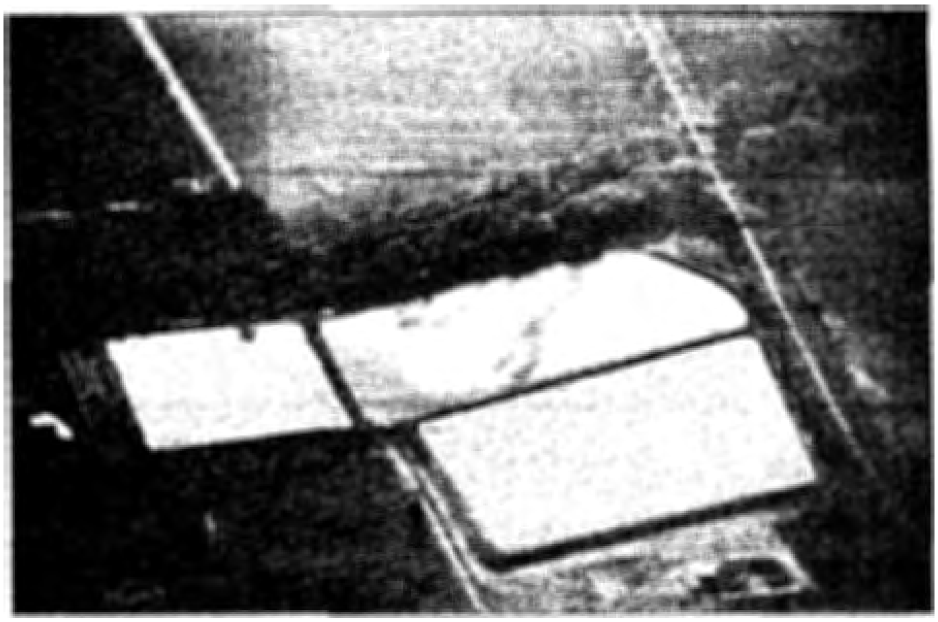
2.4.1.2 Performance

a. BOD_5 Removal

The monthly average effluent BOD_5 concentrations for the four previously described facultative pond systems are presented in Figure 2-8. In general, all of the systems were capable of providing a monthly average effluent BOD_5 concentration of less than 30 mg/l during the major portion of the year. Monthly average effluent BOD_5 concentrations ranged from 1.4 mg/l during September 1975 at the Corinne, UT, site, to 57 mg/l during March 1975 at the Peterborough, NH, site. Monthly average effluent BOD_5 concentrations tended to be higher during January, February, March, and April at all

FIGURE 2-6

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE FACULTATIVE POND SYSTEM AT EUDORA, KANSAS



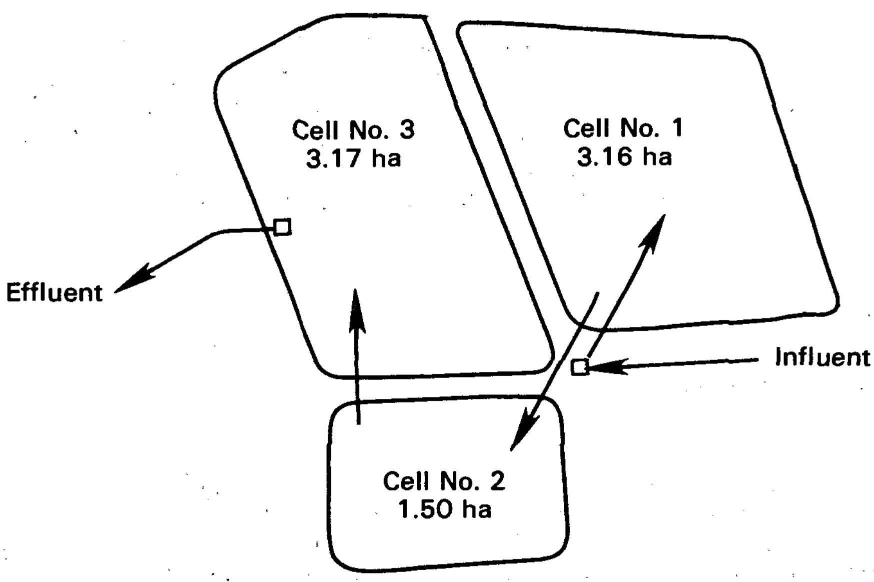
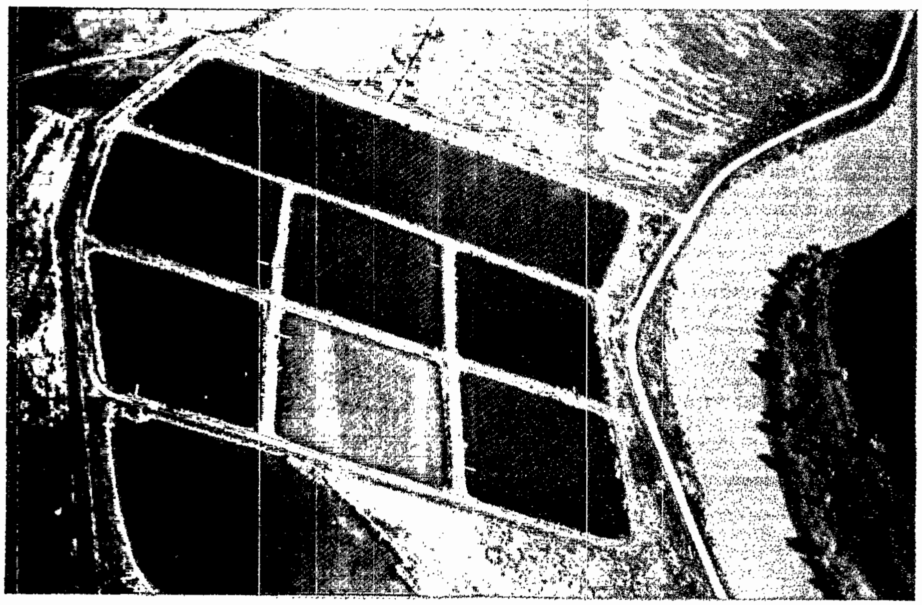


FIGURE 2-7

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE FACULTATIVE POND SYSTEM AT CORINNE, UTAH



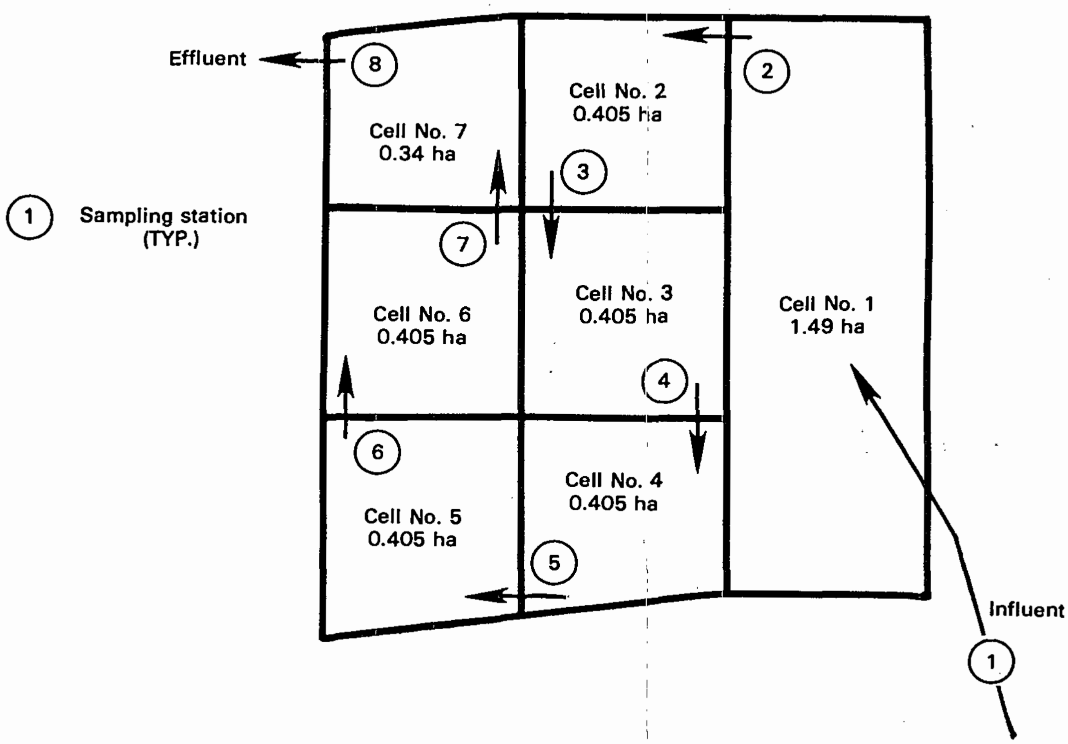
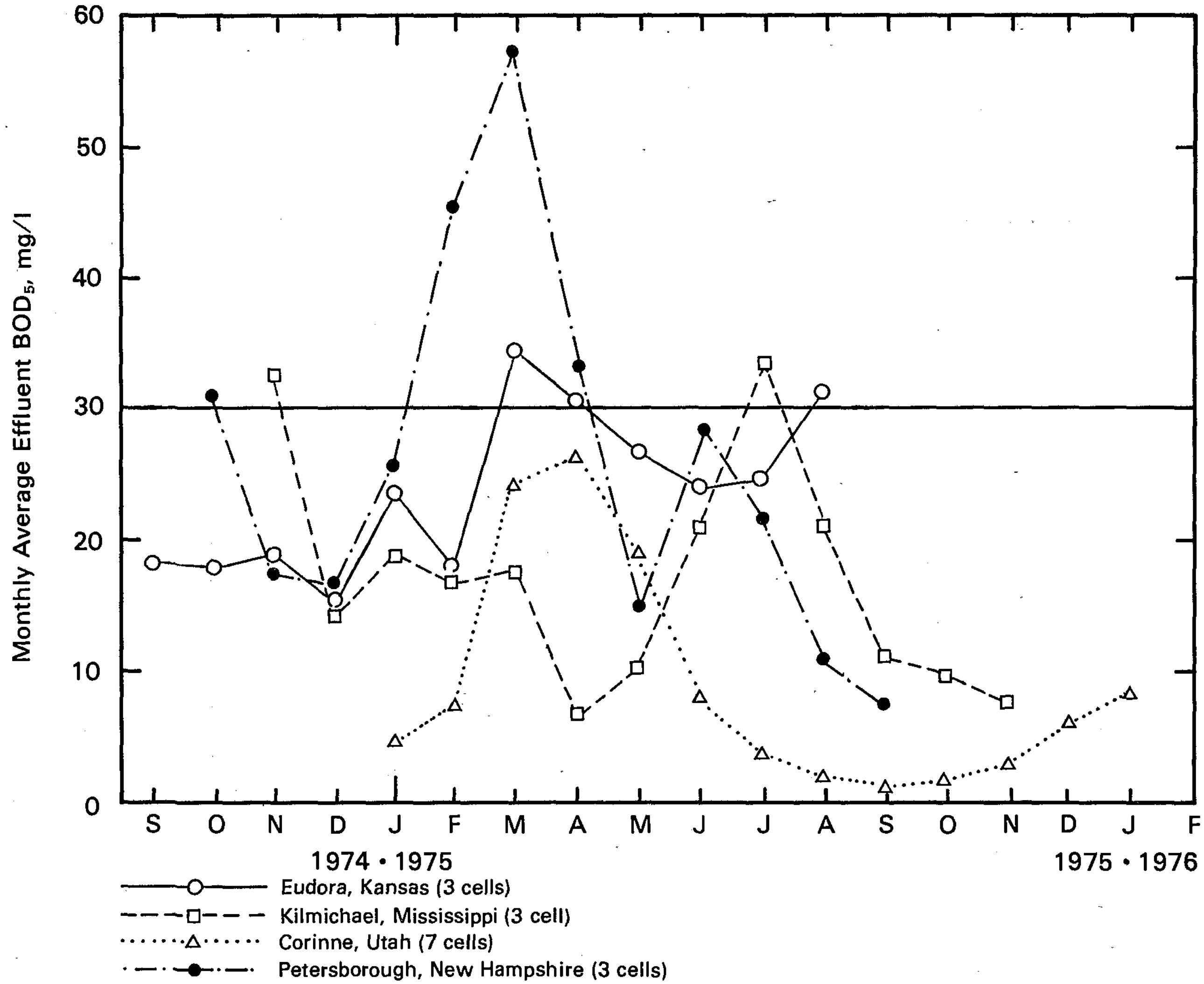


FIGURE 2-8

FACULTATIVE POND BOD_5 EFFLUENT CONCENTRATIONS



of the sites. This was especially evident at the Peterborough site when the cells were covered over by ice due to freezing temperatures. The ice cover caused the cells to become anaerobic. However, even when the ponds at the Corinne site were covered over with ice the monthly average effluent BOD_5 concentration did not exceed 30 mg/l.

None of the systems studied was significantly affected by the fall overturn; however, the spring overturn did cause significant increases in effluent BOD_5 concentrations at two of the sites. At the Corinne site two different spring overturns occurred. The first occurred in March 1975, with a peak daily BOD_5 concentration of 36 mg/l. The second occurred during April 1975, with a peak daily effluent BOD_5 concentration of 39 mg/l. At the Eudora site, the peak daily effluent BOD_5 concentration of 57 mg/l occurred during April 1975. The Kilmichael and Peterborough sites were not severely affected by the spring overturn period.

The monthly average effluent BOD_5 concentration of the Corinne pond system never exceeded 30 mg/l throughout the entire study. The Eudora pond system monthly average effluent BOD_5 concentration exceeded 30 mg/l on only two occasions during the study. The Peterborough pond system monthly average effluent BOD_5 concentration exceeded 30 mg/l in 4 of the 12 months studied.

Although these systems are subject to seasonal upsets, they are capable of producing an effluent sufficiently low in BOD_5 that discharge to a waterway is, in many cases, acceptable. It should be noted that three of the four systems were underloaded based on a comparison of design vs. actual organic and hydraulic loadings, as shown in Table 2-1. The Corinne system was grossly underloaded from an organic standpoint and grossly overloaded from a hydraulic standpoint.

b. Suspended Solids Removal

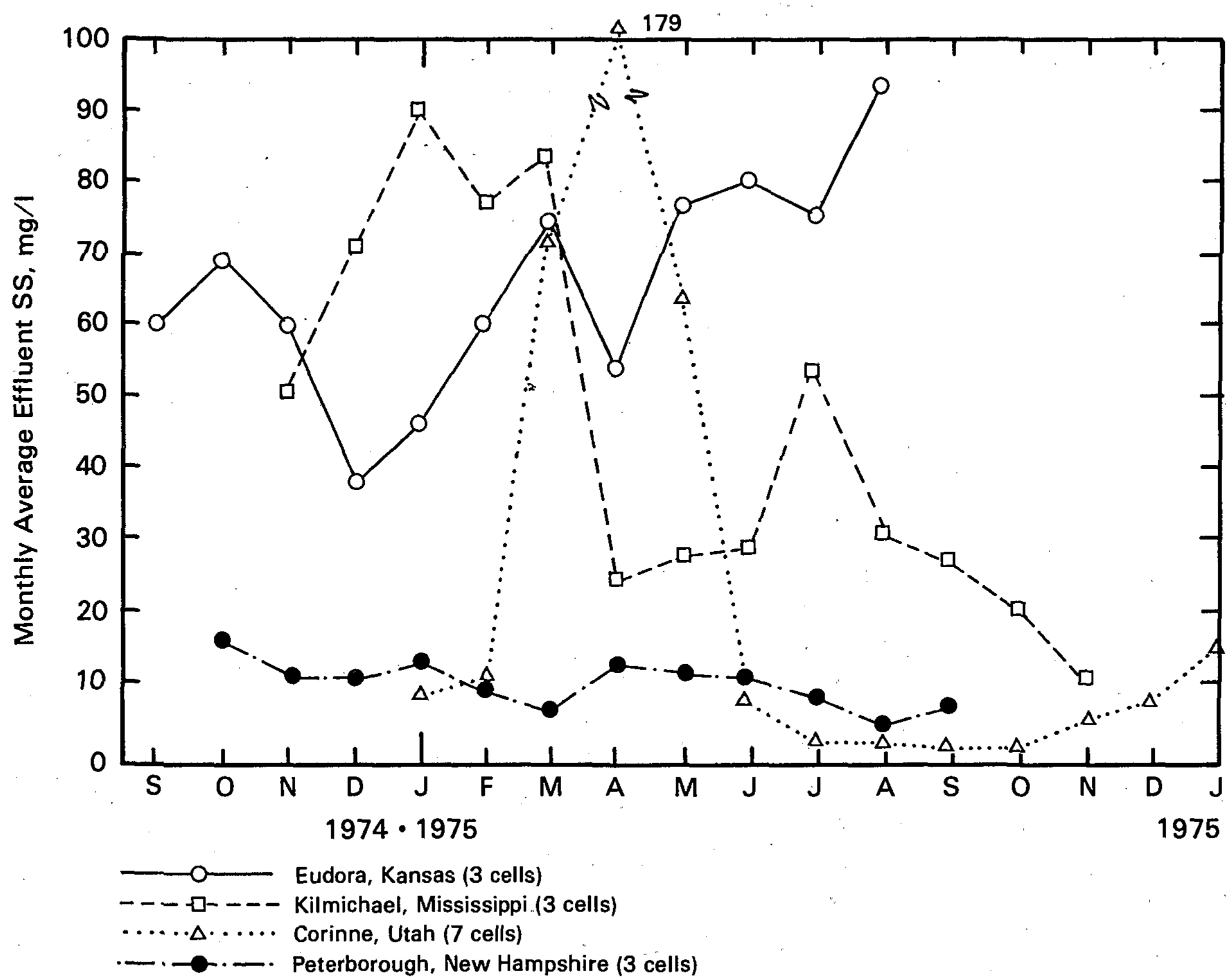
The monthly average effluent SS concentrations for each system are presented in Figure 2-9.

In general, the SS concentration in facultative pond effluents follows a seasonal pattern. Effluent SS concentration is high during summer months when algal growth is intensive and also during the spring overturn periods when settled solids are resuspended from bottom sediments due to mixing. The monthly average SS concentration ranged from 2.5 mg/l during September 1975 at Corinne to 179 mg/l during April 1975, also at Corinne. The high value of 179 mg/l occurred during the spring overturn period, which caused a resuspension of settled solids.

Eudora and Kilmichael illustrate the increase in effluent SS concentration due to algal growth during the warm summer months. However, Peterborough and Corinne were not significantly affected by algal growth during the summer months. In general, Corinne and Peterborough produced a monthly average effluent SS concentration of less than 20 mg/l. During 10 of the 13 months

FIGURE 2-9

FACULTATIVE POND SS EFFLUENT CONCENTRATIONS



studied, the monthly average effluent SS concentration at the Corinne site never exceeded 20 mg/l. However, the monthly average effluent SS concentration at Eudora was never less than 39 mg/l throughout the entire study.

The results of these studies indicate that facultative ponds can produce an effluent which has a low SS concentration; however, effluent SS concentrations will be high at various times throughout the year. In general, these SS are composed of algal cells which may not be particularly harmful to receiving streams. In areas where effluent SS standards are stringent, some type of polishing device or controlled discharge will be necessary to reduce SS concentrations to acceptable levels.

c. Fecal Coliform Removal

The monthly geometric mean effluent coliform concentrations for the four facultative pond systems are compared with a concentration of 200/100 ml in Figure 2-10.

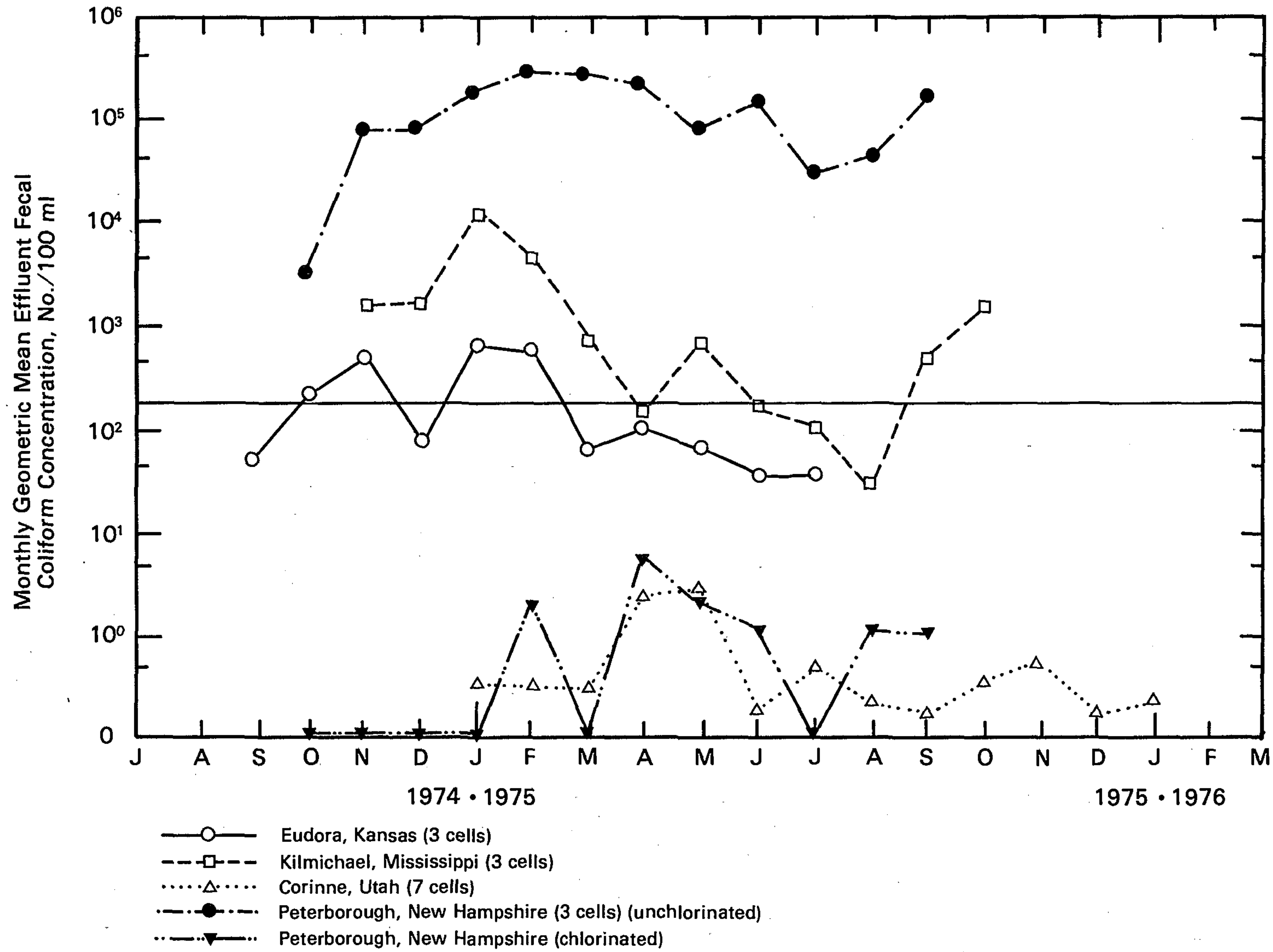
Only the Peterborough, NH, system provides chlorine disinfection. As illustrated in Figure 2-10, the chlorinated effluent at Peterborough never exceeded 20 fecal coliform organisms/100 ml. This clearly indicates that facultative pond effluent may be satisfactorily disinfected by the chlorination process.

For the three systems without disinfection processes, the geometric mean monthly effluent fecal coliform concentration ranged from 0.1 organisms/100 ml in June and September 1975, at the Corinne, UT, lagoon system to 13,527 organisms/100 ml in January 1975, at the Kilmichael, MS, lagoon system. In general, geometric mean effluent fecal coliform concentrations tend to be higher during the colder periods. The fecal coliform die-off during periods of ice cover can be expected to be significantly less than normal due to the reduced amount of sunlight reaching the organisms. The Eudora, KS, and the Kilmichael, MS, geometric mean monthly effluent fecal coliform concentrations consistently exceeded 200 organisms/100 ml during winter operation.

The fecal coliform concentration in the Corinne effluent never exceeded 10 organisms/100 ml even though this system did not include any form of disinfection. This system is composed of seven cells in series. Analysis of the fecal coliform concentrations between the seven cells indicated that fecal coliforms were essentially absent after the fourth cell in the series (17). The other two systems without disinfection only utilize three cells in series; however; fecal coliform die-off is primarily a function of actual hydraulic residence time rather than the absolute number of cells in series.

FIGURE 2-10

FACULTATIVE POND FECAL COLIFORM EFFLUENT CONCENTRATIONS



The results of these studies indicate that facultative pond effluent can be chlorinated to produce fecal coliform concentrations less than 10 organisms/100 ml. Two of the systems studied could not produce an effluent containing less than 200 fecal coliforms/100 ml. This was probably due to hydraulic short circuiting; however, the Corinne, UT, system study clearly indicated that facultative pond systems can significantly reduce fecal coliform concentrations by natural processes occurring in the pond.

2.4.1.3 Design Criteria for Organic Loading and Hydraulic Detention Time

Canter and Englande (18) reported that most states have design criteria for organic loading and/or hydraulic detention time for facultative waste stabilization ponds. These design criteria are established by these states in an effort to ensure that the quality of pond effluent would meet applicable state or Federal standards. Effluents from ponds constructed according to these design criteria repeatedly fail to meet the quality standards, thus indicating deficiencies in the current design criteria. Reported organic loading design criteria averaged 29 kg BOD_5/ha/d (26 lb BOD_5/ac/d) in the north region (above 42° latitude), 49 kg BOD_5/ha/d (44 lb BOD_5/ac/d) in the southern region (below 37° latitude), and 37 kg BOD_5/ha/d (33 lb BOD_5/ac/d) in the central region. Reported design criteria for detention time averaged 117 days in the north, 82 days in the central, and 31 days in the south region.

Design criteria for organic loading in New Hampshire was 39 kg BOD_5/ha/d (35 lb BOD_5/ac/d). The Peterborough treatment system was designed for a loading of 20 kg BOD_5/ha/d (18 lb BOD_5/ac/d) in 1968 to be increased as population increased to 40 kg BOD_5/ha/d (35 lb BOD_5/ac/d) in the year 2000. Actual loading during 1974-1975 averaged 15 kg BOD_5/ha/d (14 lb BOD_5/ac/d) with the highest monthly loading being 21 kg BOD_5/ha/d (19 lb BOD_5/ac/d). Although the organic loading was substantially below the state design limit, the effluent BOD_5 exceeded 30 mg/l during the months of October 1974 and February, March, and April 1975.

Mississippi's design criteria for organic loading was 67 kg BOD_5/ha/d (60 lb BOD_5/ac/d) in the first cell. Based on an overall loading rate, the Kilmichael treatment system was loaded at 43 kg BOD_5/ha/d (38 lb BOD_5/ac/d). Actual loading during 1974-1975 averaged 15 kg BOD_5/ha/d (13 lb BOD_5/ac/d) with a monthly maximum of 25 kg BOD_5/ha/d (22 lb BOD_5/ac/d), and yet the effluent BOD_5 exceeded 39 mg/l twice during the sample year (November and July).

The design load for the Eudora, KS, system was the same as the state design limit, 38 kg BOD_5/ha/d (34 lb BOD_5/ac/d). Actual loading during 1974-1975 averaged only 17 kg BOD_5/ha/d (15 lb BOD_5/ac/d) with a maximum of 31 kg BOD_5/ha/d (28 lb BOD_5/ac/d). The effluent BOD_5 exceeded 30 mg/l three months during the sample year (March, April, and August).

Utah has both an organic loading design limit, 45 kg BOD_5/ha/d (40 lb BOD_5/ac/d) on the primary cell and a winter detention time design criteria of 180 days. Design loading for the Corinne system was 36 kg BOD_5/ha/d (32 lb BOD_5/ac/d), and design detention time was 180 days. Although the organic loading averaged 30 kg BOD_5/ha/d (34 lb BOD_5/ac/d) on the primary cell, during two months of the sample year it exceeded 56 kg BOD_5/ha/d (50 BOD_5/ac/d). Average organic loading on the total system was 12 kg BOD_5/ha/d (11 lb BOD_5/ac/d), and the hydraulic detention time was estimated to be 88 days during the winter. Regardless of the deviations from the state design criteria, the average monthly BOD_5 never exceeded 30 mg/l.

A summary of the state design criteria for each location and actual design values for organic loading and hydraulic detention time are shown in Table 2-2. Also included is a list of the months the Federal effluent standard for BOD_5 was exceeded. Note that the actual organic loading for all four systems are nearly equal, yet as the monthly effluent BOD_5 averages shown in Figure 2-8 indicate, the Corinne system consistently produced a higher quality effluent. This may be a function of the larger number of cells in the Corinne system--seven as compared to three for the rest of the systems. Hydraulic short circuiting may be occurring in the three cell systems, resulting in a shorter actual detention time than exists in the Corinne system. Detention time may also be affected by the location of cell inlet and outlet structures. As shown in Figure 2-7, the outlet is at the furthest point possible from the inlet in the Corinne, UT, system. At the Eudora, KS, system shown in Figure 2-6, large "dead zones" undoubtedly occur in each cell due to the unnecessarily short distance between inlet and outlet. These dead zones result in decreased hydraulic detention and increased effective organic loading rate.

2.4.1.4 Nitrogen Removal

a. Theoretical Considerations

Differences between influent and effluent nitrogen concentration in facultative stabilization ponds occur principally through the following processes:

1. Gaseous ammonia stripping to the atmosphere
2. Ammonia assimilation in algal biomass
3. Nitrate assimilation in algal and other plant biomass
4. Biological nitrification-denitrification

TABLE 2-2

SUMMARY OF DESIGN AND PERFORMANCE DATA--SELECTED FACULTATIVE PONDS

Location	Organic Loading: Design Standard (kg BOD_5/ha/d)	Organic Loading: Design (kg BOD_5/ha/d)	Organic Loading: Actual (1974-1975) (kg BOD_5/ha/d)	Theoretical Hydraulic Detention Time: Design Standard (days)	Theoretical Hydraulic Detention Time: Design (days)	Theoretical Hydraulic Detention Time: Actual (days)	Months Effluent Exceeded 30 mg/l BOD_5
Peterborough, NH	39	20	16	None	57	107	Oct/Feb/Mar/Apr
Kilmichael, MS	56	67[a] 43[b]	27[a] 18[b]	None	79	214	Nov/Jul
Eudora, KS	38	38	19	None	47	231	Mar/Apr/Aug
Corinne, UT	45[a]	36[a]	34[a] 15[b]	180	180	70 88[c]	None

[a]Primary cell.
[b]Entire system.
[c]Estimated from dye study.

The following design equations are based on equations describing the loss of gaseous ammonia to the atmosphere; however, the design parameters also reflect the influence of all processes associated with ammonia nitrogen removal. The rate of gaseous ammonia losses to the atmosphere depends mainly on the pH value, surface to volume ratio, temperature, and the mixing conditions in the pond. Alkaline pH shifts the equilibrium equation $NH_3^o + H_2O \rightleftarrows NH_4^+ + OH^-$ toward gaseous ammonia production, while the mixing conditions affect the magnitude of the mass transfer coefficient. Temperature affects both the equilibrium constant and mass transfer coefficient.

Ammonia removal in ponds can be expressed by assuming a first order reaction (19)(20). A conceptual development of the mathematical model is presented elsewhere (21). The final design equations are given below:

<u>For temperatures of 1°C to 20°C</u>:

$$\frac{C_e}{C_o} = \frac{1}{1 + \frac{A}{Q}(0.0038 + 0.000134T)e^{(1.041 + 0.044T)(pH-6.6)}} \tag{2-11}$$

<u>For temperatures of 21°C to 25°C</u>:

$$\frac{C_e}{C_o} = \frac{1}{1 + \frac{A}{Q}(5.035 \times 10^{-3})e^{1.540\,(pH-6.6)}} \tag{2-12}$$

C_o = influent concentration of $(NH_4^+ + NH_3^o)$, mg/l as N

C_e = effluent concentration of $(NH_4^+ + NH_3^o)$, mg/l as N

A = surface area of the pond, m^2

Q = flow rate, m^3/d

T = temperature, °C

b. Pond Systems

The stabilization pond systems located in Peterborough, NH; Eudora, KS; and Corinne, UT, described earlier, are exposed to similar climatic conditions. During the winter the water temperatures range between 1°C and 5°C (between 34°F and 41°F), and ice cover is experienced during the winter. During the summertime the water temperature is approximately 20°C (68°F) and isgenerally less than 25°C (77°F). The fourth system at Kilmichael, MS, was excluded from the analysis because of the different aeration systems and much milder climate.

The characteristics of the wastewater entering these systems were significantly different, as shown in Table 2-3. The Eudora wastewater is a typical medium-strength domestic wastewater; the Corinne wastewater is slightly alkaline; and Peterborough has a neutral unbuffered wastewater. The pH value in the ponds has a marked effect on ammonia-N removal. In the Corinne ponds the pH values were above 9, while in the Peterborough system the pH value during the summer was 6.7 to 7.4, which is low compared with other ponds.

c. Ammonia-N Removal

The annual average percentage nitrogen removals, calculated as ammonium nitrogen, at the Eudora and Corinne facultative pond systems were about 95 percent, and at the Peterborough ponds it was about 46 percent. The major removal occurred in the primary cells. Table 2-4 summarizes these results.

During the months of June through September, ammonia-N removal in the Peterborough system reached 67 percent, while at Eudora ammonia-N removal was almost 99 percent. In the winter period of December to March in the primary cell of the Peterborough system, there was no ammonia-N removal, and in the total system there was approximately 19 percent ammonia-N removal. In the Eudora and Corinne systems, during this period, there was about 90 percent ammonia-N removal with 53-62 percent removal occurring in the primary cells.

2.4.2 Aerated Ponds

Results of intensive aerated pond performance studies of aerated ponds at Bixby, OK; Pawnee, IL; North Gulfport, MS; Lake Koshkonong, WI; and Windber, PA (22-26) are presented in this section. Data collection encompassed 12 months with four separate 30-consecutive-day, 24-hour composite sampling periods, once each season.

TABLE 2-3

INFLUENT WASTEWATER CHARACTERISTICS AT SELECTED FACULTATIVE PONDS

Parameter	Peterborough NH	Eudora KS	Corinne UT
BOD, mg/l	138	270	74
COD, mg/l	271	559	128
NH_4, mg/l	21.5	25.5	7.5
pH	7.0	7.7	8.4
Alkalinity, mg/l as $CaCO_3$	107	428	576

TABLE 2-4

ANNUAL AVERAGE AMMONIA-NITROGEN REMOVAL BY SELECTED FACULTATIVE PONDS

Parameter	Peterborough NH	Eudora KS	Corinne UT
NH_4-N, mg/l			
Influent	21.5	25.5	7.5
Cell #1 Effluent	16.5	9.2	1.4
Final Effluent	11.5	1.1	0.2
NH_4-N Removal, percent			
Cell #1	23.3	63.9	81.3
Total System	46.5	95.7	97.3
Theor. Detention Time, days			
Cell #1	44	92	29
Total System	107	231	70

Aerated ponds are medium-depth, manmade basins designed for the biological treatment of wastewater. A mechanical aeration device is used to supply supplemental oxygen to the system. Well-mixed aerated ponds are aerobic throughout their entire depth. The mechanical aeration device may cause turbulent mixing (i.e., surface aerator) or may produce laminar flow conditions (diffused air systems). The five aerated ponds were considered to be partial mix systems by the investigators, although data describing the mixing characteristics were not collected.

The North Gulfport system contains surface aerators; the average depth of the aerated cells is 1.9 m (6.3 ft). Diffused-air aeration systems are used in the other ponds, and the average depth is 3.0 m (10 ft). The Bixby and North Gulfport systems consist of two aerated cells in series and the others, three cells in series. The aerated cells of the North Gulfport system are followed by settling ponds and a chlorine contact pond. The operating conditions for these systems and the influent wastewater characteristics entering these systems were significantly different as shown in Tables 2-5 and 2-6.

2.4.2.1 Site Descriptions

a. Bixby, Oklahoma

A diagram of the Bixby, OK, system is shown in Figure 2-11 (22). The system consists of two aerated cells with a total surface area of 2.3 ha (5.8 ac) designed to treat 335 kg BOD_5/d (737 lb BOD_5/d) with a hydraulic loading rate of 1,550 m^3/d (0.4 mgd). The design organic loading rate on the first cell is shown in Table 2-6. There is no chlorination facility at the site. The design hydraulic retention time was 32 days.

b. Pawnee, Illinois

A diagram of the Pawnee, IL, system is shown in Figure 2-12 (23). The system consists of three aerated cells in series with a total surface area of 4.45 ha (11.0 ac). The design flow rate 1,890 m^3/d (0.5 mgd) with a design organic load of 386 kg BOD_5/d (850 lb BOD_5/d) and a design hydraulic retention time of 60 days. The design organic loading rate on the first cell is shown in Table 2-6. The facility is equipped with chlorination disinfection and a slow sand filter for polishing the effluent. The filter was removed following the study. Data reported below were collected prior to the filters and represent only pond performance.

c. North Gulfport, Mississippi

A diagram of the North Gulfport, MS, system is shown in Figure 2-13 (24). The system consists of two aerated cells in series with a total surface area

TABLE 2-5

INFLUENT WASTEWATER CHARACTERISTICS AT SELECTED AERATED PONDS

Parameter	Pawnee IL	Bixby OK	Lake Koshkonong WI	Windber PA	North Gulfport MS
BOD_5, mg/l	473	368	85	173	178
COD, mg/l	1,026	653	196	424	338
TKN, mg/l	51.4	45.0	15.3	24.3	26.5
NH_4-N, mg/l	26.32	29.58	10.04	22.85	15.7
pH	6.8-7.4	6.1-7.1	7.2-7.4	5.6-6.9	6.7-7.5
Alkalinity, mg/l as $CaCO_3$	242	154	397	67	144

TABLE 2-6

DESIGN AND ACTUAL LOADING RATES AND DETENTION TIMES FOR SELECTED AERATED PONDS

Location	Organic Loading Rate on First Cell				Total System Theor. Hydraulic Detention Time	
	Design	Actual	Design	Actual	Design	Actual
	kg BOD_5/ha/d		kg BOD_5/1,000 m^3/d		days	
Pawnee, Il	154[a]	150	5.8	5.6	60	144
Bixby, OK	284[b]	161	12.0	6.7	32	108
Lake Koshkonong, WI	509[b]	87	20.2	3.6	30	73
Windber, PA	497[b]	285	18.6	9.8	30	48
North Gulfport, MS	375[c]	486	19.3	25.1	26[d]	18

[a]Equaled or exceeded 7 of the 12 months monitored.
[b]Not exceeded.
[c]Exceeded 11 of the 12 months monitored.
[d]Aerated cells only.

FIGURE 2-11

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE AERATED POND SYSTEM AT BIXBY, OKLAHOMA

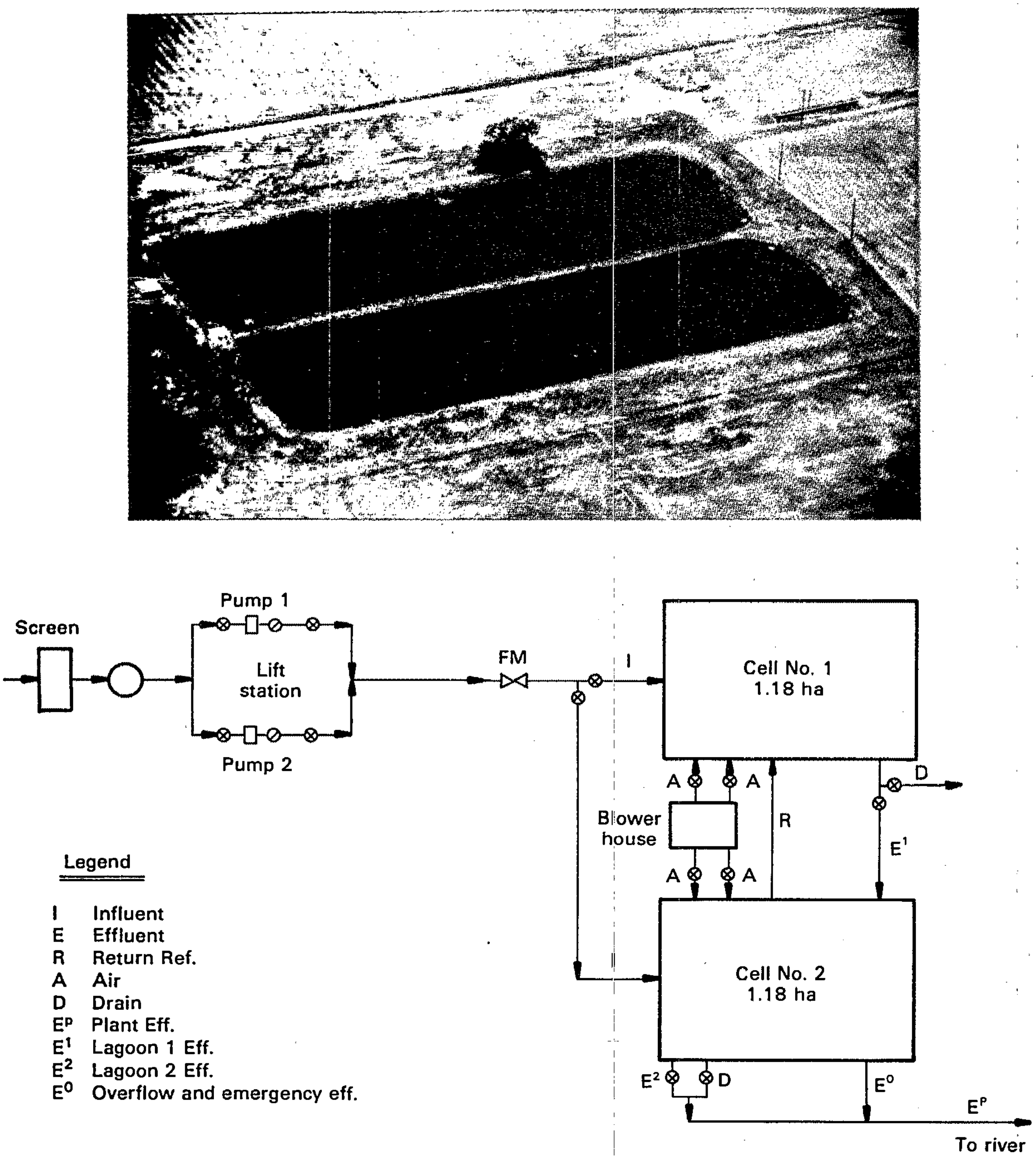


FIGURE 2-12

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE AERATED POND SYSTEM AT PAWNEE, ILLINOIS

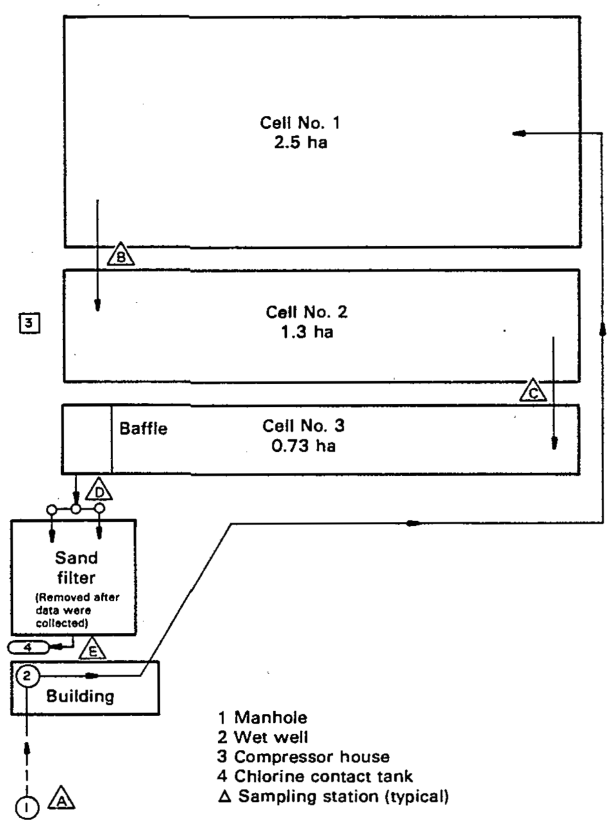
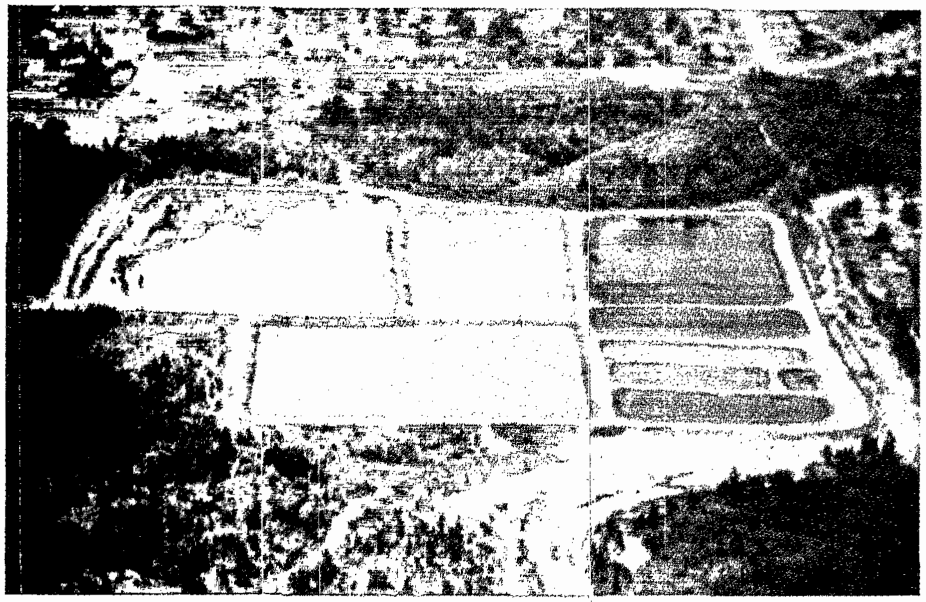
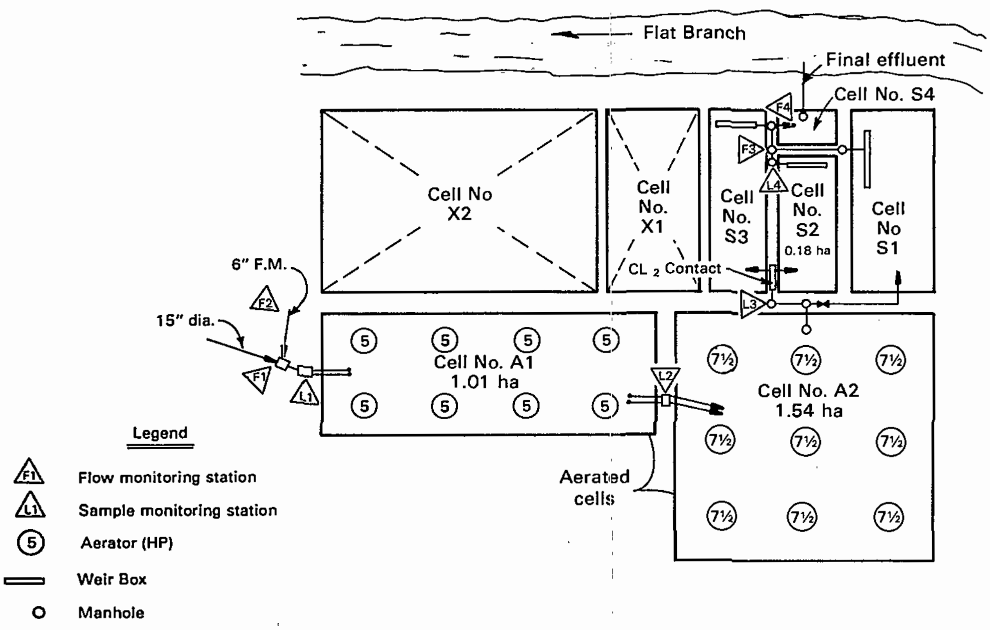


FIGURE 2-13

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE AERATED POND SYSTEM AT GULFPORT, MISSISSIPPI





of 2.5 ha (6.3 ac) followed by settling ponds with a theoretical detention time of five days. The system was designed to treat 1,890 m^3/d (0.5 mgd) with a total theoretical hydraulic retention time of 26 days. The design organic loading rate on the first cell in the series is shown in Table 2-6. The system is equipped with a chlorination facility.

d. Lake Koshkonong, Wisconsin

A diagram of the Lake Koshkonong, WI, system is shown in Figure 2-14 (25). The system consists of three aerated cells with a total surface area of 2.8 ha (6.9 ac) followed by chlorination. The design flow was 2270 m^3/d (0.6 mgd) with a design organic load of 463 kg BOD_5/d (1,020 lb BOD_5/d) and a design hydraulic detention time of 30 days. The design organic loading rate on the first cell is shown in Table 2-6.

e. Windber, Pennsylvania

The Windber, PA, system consists of three cells in series with a total surface area of 8.4 ha (20.7 ac) followed by chlorination (Figure 2-15) (26). The design flow rate was 7,576 m^3/d (2.0 mgd) with a design organic load of approximately 1,540 kg BOD_5/d (3,400 lb BOD_5/d). The design organic loading rate on the first cell is shown in Table 2-6. The design mean hydraulic residence time was 30 days for the three cells operating in series.

2.4.2.2 Performance

a. BOD_5 Removal

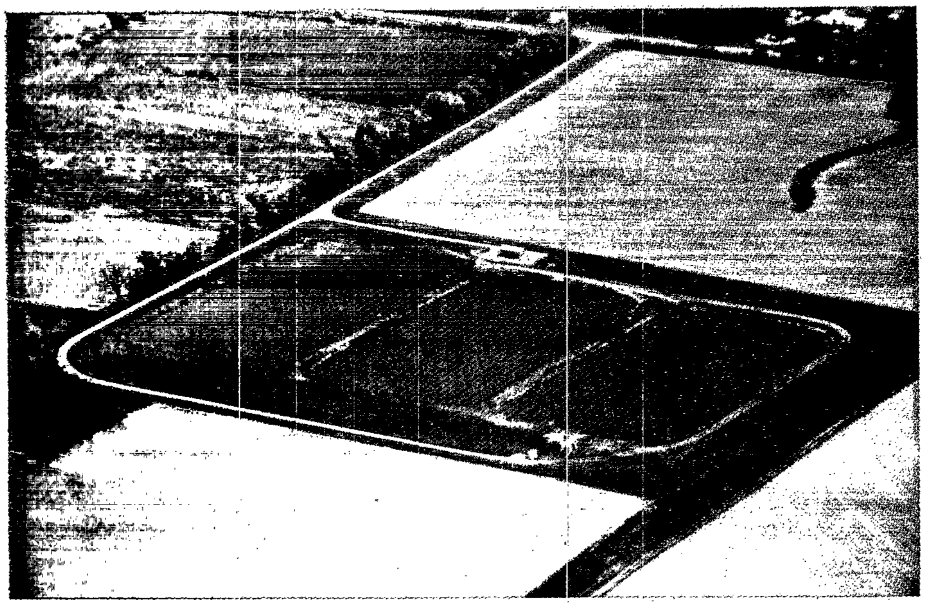
The monthly average effluent BOD_5 concentrations for the five previously described aerated pond systems are presented in Figure 2-16.

In general, all of the systems, except the Bixby, OK, system, were capable of producing monthly average effluent BOD_5 concentrations of less than 30 mg/l. Monthly average effluent BOD_5 concentrations appear to be independent of influent BOD_5 concentration fluctuations and not significantly affected by seasonal variations in temperature.

Monthly average influent BOD_5 concentrations at Bixby, OK, ranged from 212 mg/l to 504 mg/l with a mean of 388 mg/l during the study period reported. The design influent BOD_5 concentration was 240 mg/l, or only 62 percent of the actual influent concentration. The mean flow rate during the period of study was 520 m^3/d (0.12 mgd), which is less than one-third of

FIGURE 2-14

SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE AERATED POND SYSTEM AT LAKE KOSHKONONG, WISCONSIN



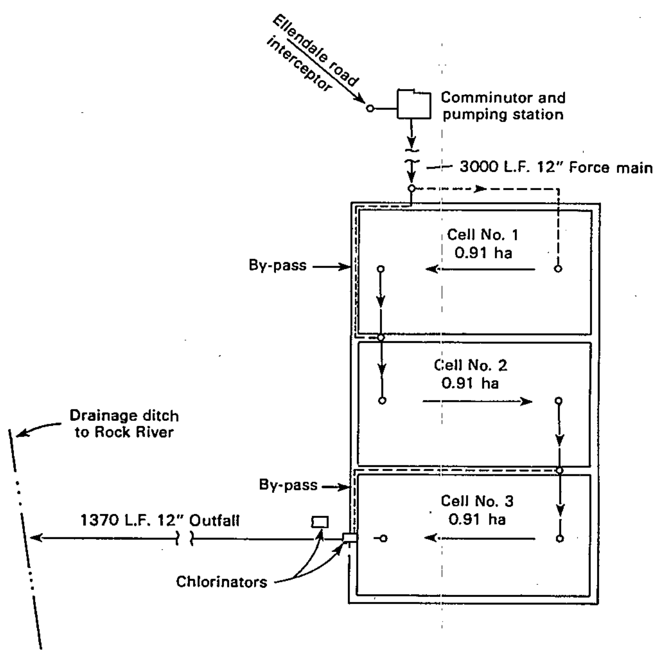
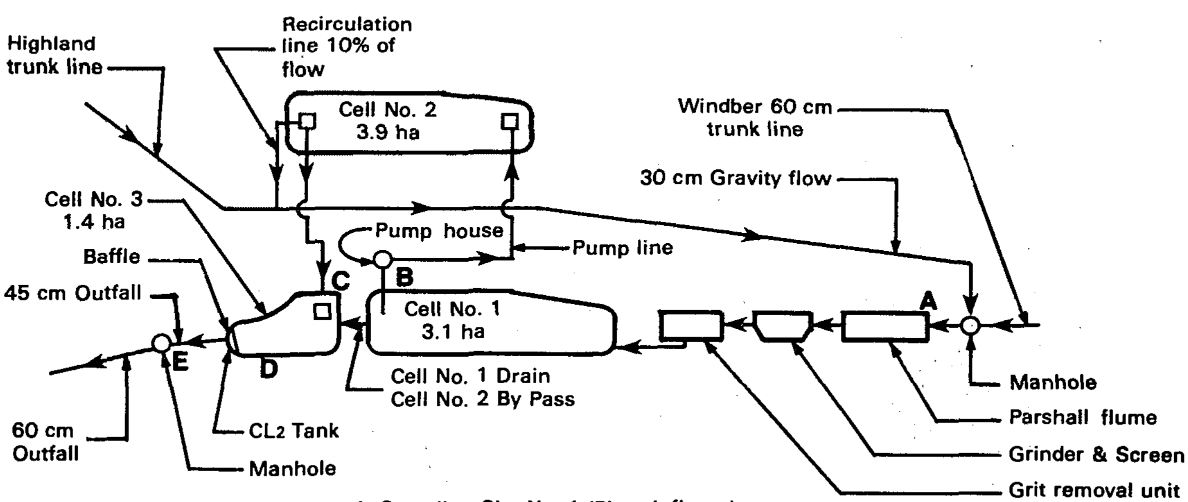


FIGURE 2-15

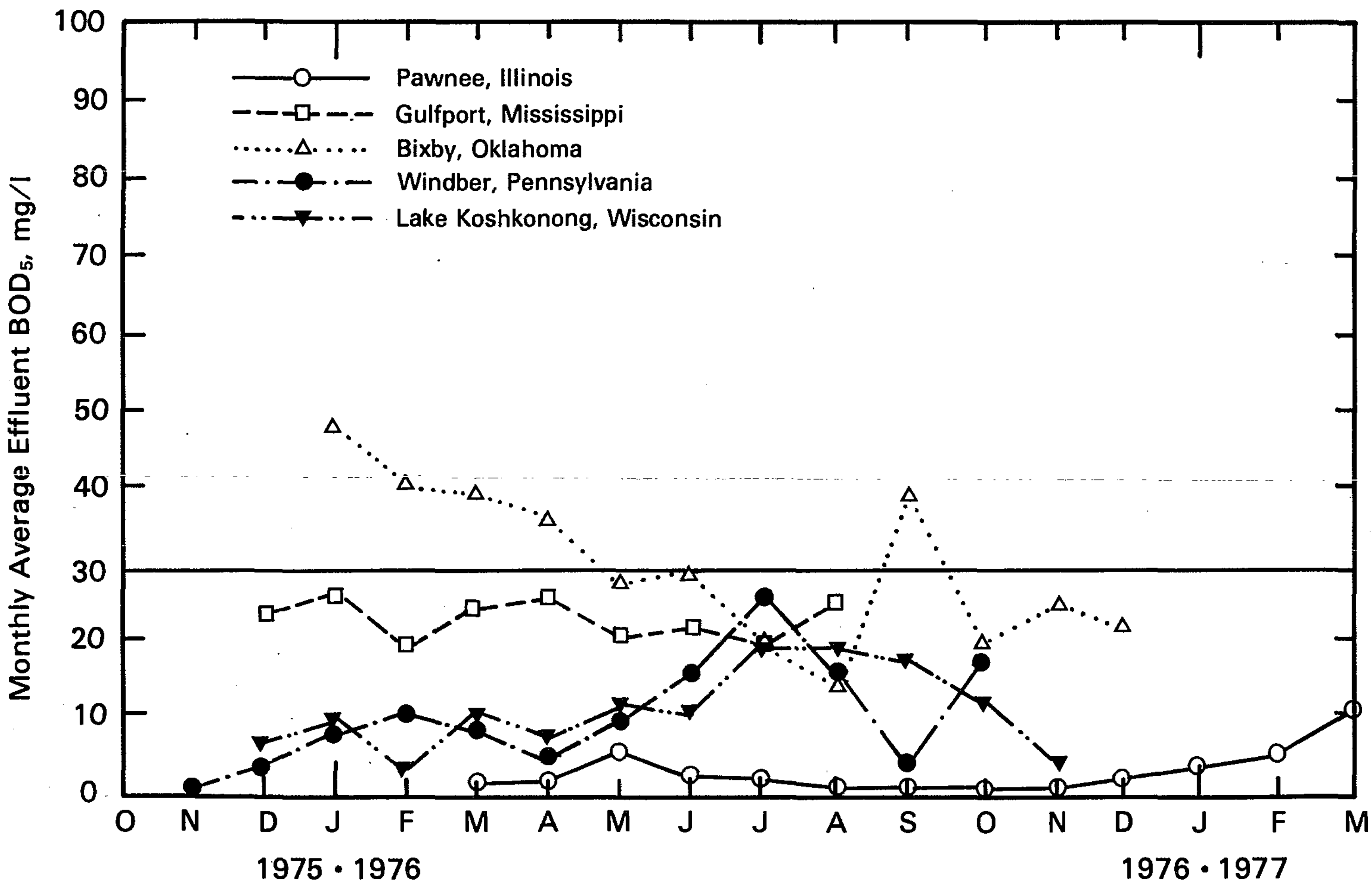
SCHEMATIC FLOW DIAGRAM AND AERIAL PHOTOGRAPH OF THE AERATED POND SYSTEM AT WINDBER, PENNSYLVANIA



A Sampling Site No. 1 (Plant Influent)
B Sampling Site No. 2 (Cell No. 1 Effluent)
C Cell No. 2 Effluent
D Cell No. 3 Effluent
E Plant Effluent (After Chlorination)

FIGURE 2-16

AERATED POND BOD_5 EFFLUENT CONCENTRATIONS



the design flow rate. The Bixby system was designed to treat 336 kg BOD_5/d (740 lb BOD_5/d), and apparently a load of only 203 kg BOD_5/d (446 lb BOD_5/d) was entering the lagoon. The only major difference between the Bixby and other aerated lagoons is the number of cells. Bixby has only two cells in series. Based upon the results of studies with facultative ponds which show improved performance with an increase in cell number, this difference in configuration could account for the relatively poor performance by the Bixby system. However, there are other possible explanations, i.e., operating procedures and short circuiting.

The results of these studies indicate that partial mix aerated ponds which are properly designed, operated, and maintained can consistently produce an effluent BOD_5 concentration of less than 30 mg/l. In addition, effluent quality is not seriously affected by seasonal climate variations.

b. Suspended Solids Removal

The monthly average effluent SS concentrations for each system are presented in Figure 2-17. With the exception of the Bixby system, the ponds produced relatively constant effluent SS concentrations throughout the entire year.

Mean monthly effluent SS concentrations ranged from 2 mg/l at Windber, PA, in November 1975, to 96 mg/l at Bixby, OK, in June 1976. The mean monthly effluent SS concentration for the Windber, PA, site never exceeded 30 mg/l throughout the entire study period and, at Pawnee, IL, and Lake Koshkonong, WI, only exceeded 30 mg/l during one of the months reported.

The results of these studies indicate that aerated pond effluent SS concentrations are variable. However, a well-designed, operated, and maintained aerated pond can produce final effluents with low SS concentrations.

c. Fecal Coliform Removal

Only two monthly geometric mean fecal coliform values were available for Bixby, OK. The monthly geometric mean effluent fecal coliform concentrations compared to a concentration of 200 organisms/100 ml for all five systems are illustrated in Figure 2-18.

All of the aerated pond systems except Bixby, OK, have chlorine disinfection. These data show that aerated pond effluent can be disinfected. In general, the Windber PA, and the Pawnee, IL, systems produced final effluent monthly geometric mean fecal coliform concentrations of less than 200 organisms/100 ml. The nonchlorinated Bixby, OK, effluent had a high fecal coliform concentration. The Gulfport, MS, system produced an effluent containing more than 200 fecal coliforms/100 ml most of the time, but the fecal coliforms were measured in effluent samples from a holding pond

FIGURE 2-17

AERATED POND SS EFFLUENT CONCENTRATIONS

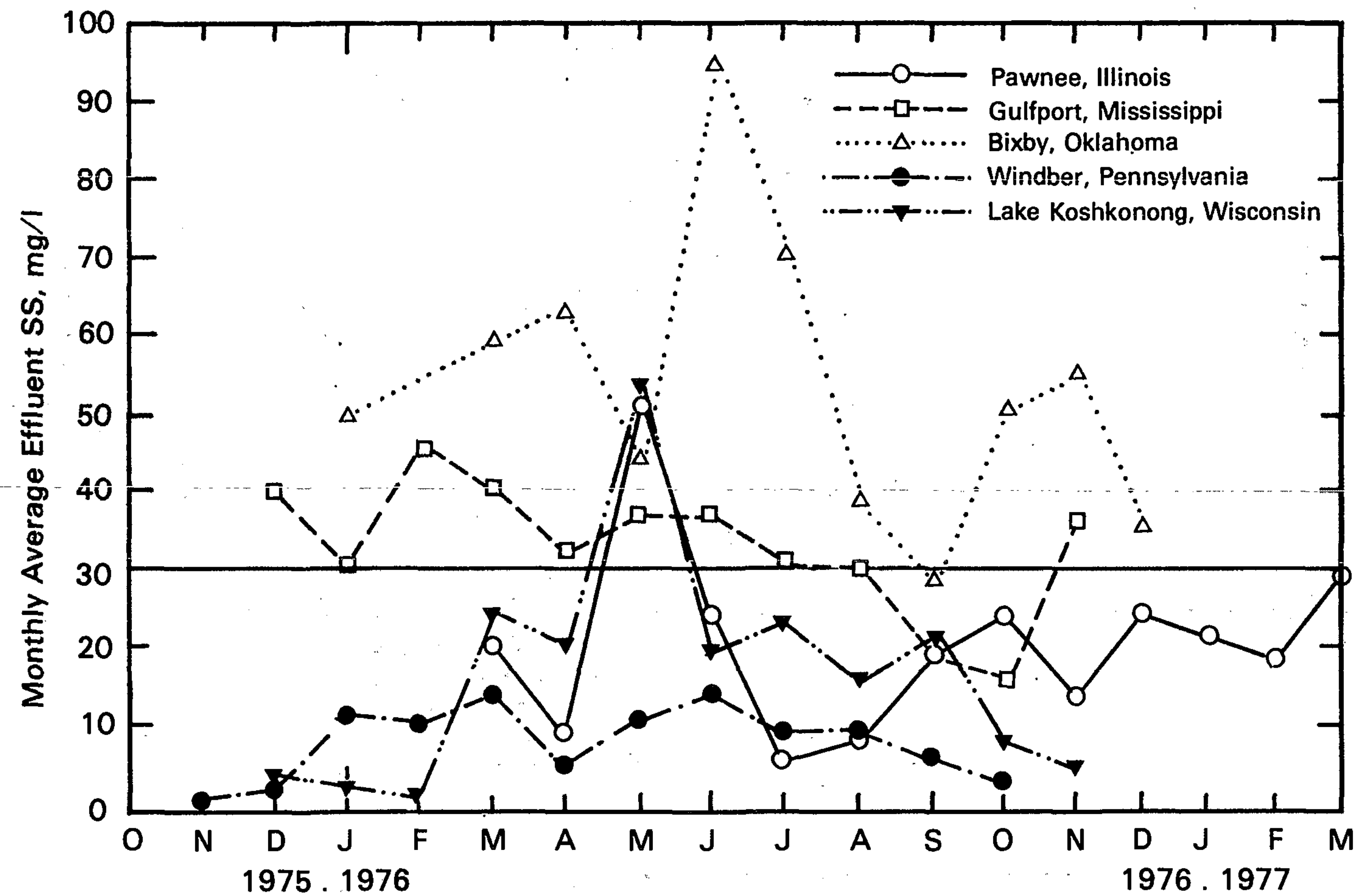
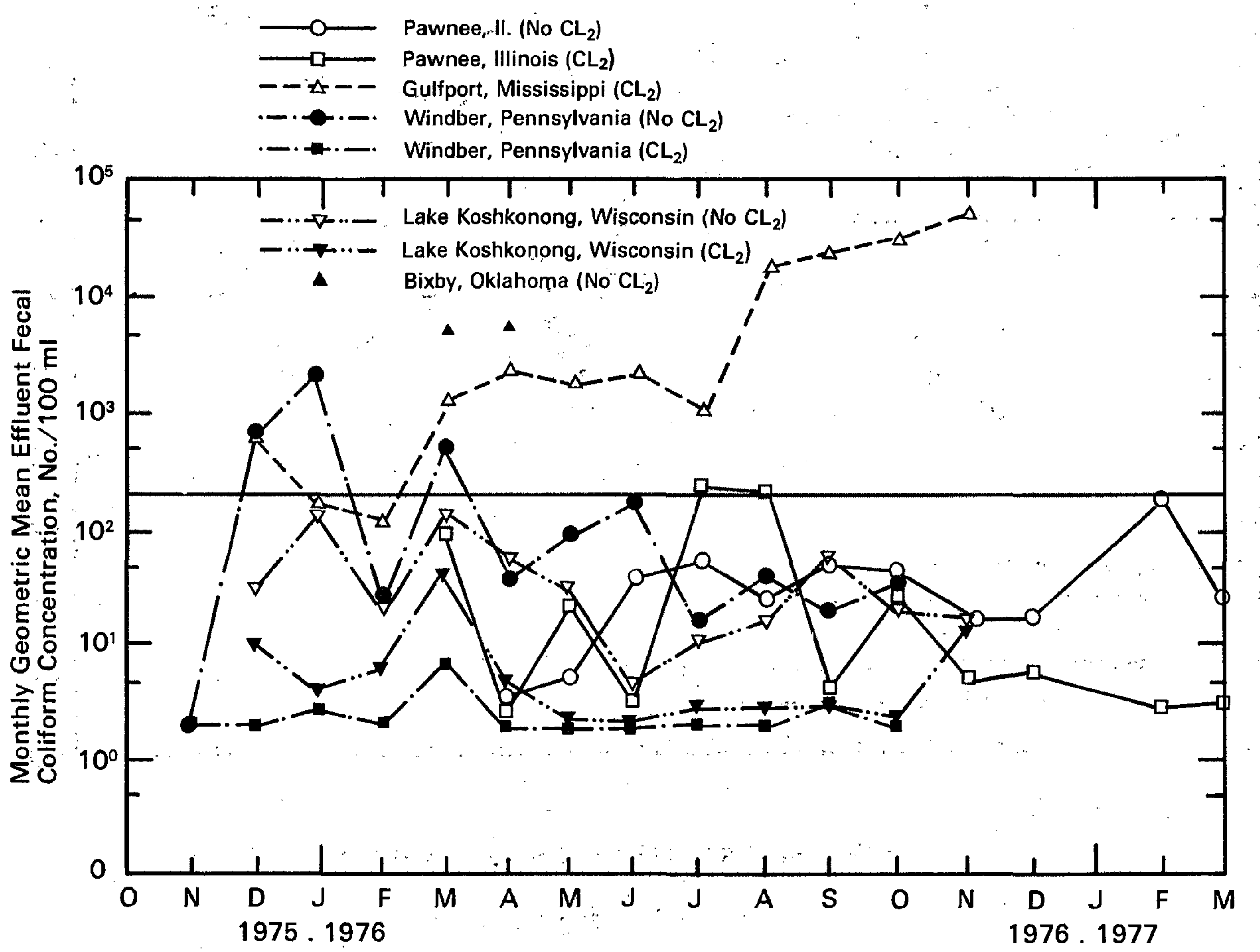


FIGURE 2-18

AERATED POND FECAL COLIFORM EFFLUENT CONCENTRATIONS



with a long detention time following the addition of the chlorine. Aftergrowth of the fecal coliform probably accounted for the high concentrations. Aerated pond systems are capable of producing effluents containing less than 200 fecal coliforms/100 ml without disinfection.

d. Summary

From the performance data currently available, it appears that (1) aerated ponds can produce an effluent BOD_5 concentration of less than 30 mg/l, (2) aerated pond SS concentrations are relatively stable throughout the year, and (3) an aerated pond effluent can be produced containing less than 200 fecal coliforms/100 ml.

2.4.2.3 Design Criteria

Most partial and complete mix aerated ponds have been designed using a complete mix hydraulic model and pseudo-first order removal rates. The Ten-State Standards (27), which is the basis for the majority of the states' standards, recommend that a complete mix formula be used. The U.S. Army Corps of Engineers has designed most of its aerated ponds using the complete mix formula. Results presented in the Appendix indicate that this formula does not give the best fit of the performance data for the five selected partial mix aerated pond systems discussed above.

A plug flow-first order kinetic model best described the performance of these five partial mix aerated ponds. Use of the plug flow model is illustrated in a design example in Chapter 3.

Performance data were not collected for complete mix aerated ponds, but experience has shown that when adequate mixing is applied, the complete mix hydraulic model and pseudo-first order kinetics can be used for design. A design example based on the complete mix model is presented in Chapter 3 (28).

The development and analyses of the complete mix, plug flow, and other models used to design aerated ponds are presented in the Appendix. Environmental conditions have a significant effect on the design of aerated ponds. Some of the effects of environmental factors were discussed earlier in this chapter, and others are discussed in Chapter 3 and in the Appendix.

2.4.2.4 Nitrogen Removal

a. Theoretical Considerations

Ammonia nitrogen exists in aqueous solutions as ammonia or ammonium ions. At a pH value of 8.0, approximately 95 percent of the nitrogen is in the form of ammonium ion. Therefore, in biological systems such as aerated ponds where the pH values are usually less than 8.0, the majority of the ammonia nitrogen is in the form of ammonium ion.

Total Kjeldahl nitrogen (TKN) is composed of the ammonia, ammonium, and organic nitrogen. Organic nitrogen is a potential source of ammonia and ammonium nitrogen because of the deamination reactions during the metabolism of organic matter in wastewater.

TKN reduction in aerated ponds can occur through several processes:

1. Gaseous ammonia stripping to the atmosphere

2. Ammonium assimilation in biomass

3. Biological nitrification

4. Biological denitrification

5. Sedimentation of insoluble organic nitrogen

6. Nitrate assimilation

Table 2-7 contains a summary of selected equations developed by Middlebrooks and Pano (29) to predict ammonia nitrogen and TKN removal in diffused-air aerated ponds. All of the equations have a common data base; however, the data were used differently to develop several of the equations. The "System" column in Table 2-7 describes the cell or series of cells that were used to develop the equation. The description "Cells 1, 2, and 3" indicates that the influent concentrations of TKN or ammonia nitrogen were used in conjunction with the effluents from the first cell, second cell, and third cell in series to estimate the removal or detention time necessary to achieve the measured removal. The description "Total System" indicates that only the influent and final effluent from the system were used to develop the equations. The description "All Data" indicates that all possible combinations of the system were used. In this combination, the results developed under "Cells "1, 2, and 3," as well as the results developed from the intermediate cells, are incorporated. For example, the ammonia nitrogen concentration in the effluent from the first cell of the pond system would be considered the influent to the second cell, and this influent concentration and the effluent from the second cell would then be used in the formulas to calculate the detention time or removal rates. These combinations of data were analyzed statistically, and the equations presented in Table 2-7 were selected based upon the best statistical fit of these data for the various combinations that were tried. Therefore, although the combinations of data are not directly comparable, the comparison presented in Table 2-7 does take into account the best statistical fit of the data.

All of the relationships for TKN removal are statistically significant at levels higher than one percent. Because of the small difference in detention

TABLE 2-7

COMPARISON OF VARIOUS EQUATIONS DEVELOPED TO PREDICT AMMONIA NITROGEN AND TKN REMOVAL IN DIFFUSED-AIR AERATED PONDS (48)

Equation Used to Estimate Detention Time	Correlation Coefficient	Hydraulic Detention Time (Days)	Comparison with Max Detention Time (% Dif.)	System
<u>TKN Removal</u>				
% TKN Removal = 130-2,324 (Hydraulic Loading Rate)	0.998	142	0	Total System-- Mean Annual Data
% TKN Removal = 137-6,511 (1/Detention Time)	0.993	114	19	Total System-- Mean Annual Data
ln C_e/C_0 = -0.0129 (Detention Time)	0.911	125	12	Cell 1, 2, and 3-- Mean Monthly Data
TKN Removal Rate = 0.809 (Loading Rate)	0.983	132	7	Total System-- Mean Monthly Data
TKN Removal Rate = 0.0946 (BOD_5 Loading Rate)	0.967	113	20	Total System-- Mean Monthly Data
TKN Fraction Removed = 0.0062 (Detention Time)	0.959	129	9	Cells 1, 2, and 3-- Mean Monthly Data
<u>Ammonia-N Removal</u>				
% NH_4-N Removal = 139-2,498 (Hydraulic Loading Rate)	0.996	129	2	Total System-- Mean Annual Data
% NH_4-N Removal = 147-7,032 (1/Detention Time)	0.995	105	20	Total System-- Mean Annual Data
ln C_e/C_0 = -0.0205 (Detention Time)	0.798	79	40	All Data-- Mean Monthly Data
NH_4-N Removal Rate = 0.869 (Loading Rate)	0.968	92	30	Total System-- Mean Monthly Data
NH_4-N Removal Rate = 0.0606 (BOD_5 Loading Rate)	0.932	132	0	Total System-- Mean Monthly Data
NH_4-N Fraction Removed = 0.0066 (Detention Time)	0.936	121	8	Cells 1, 2, and 3-- Mean Monthly Data

Hydraulic Loading Rate = $m^3/m^2/d$; Detention Time = days; Loading Rate = $g/m^3/d$; BOD_5 = $g/m^3/d$; Removal Rate = $g/m^3/d$; C_e = Effluent TKN or Ammonia-N concentration, mg/l; C_0 = Influent TKN or Ammonia-N concentration, mg/l.

times calculated using all of the expressions, there is a good basis to apply any of the relationships in design of ponds to estimate TKN removal. Using the mean annual data for diffused-air aerated ponds yields a more conservative design when employing the hydraulic loading rate relationship. However, the values obtained using the reciprocal of the detention time relationship yields a value slightly lower than that recommended by the majority of the other expressions. In view of the error that might be introduced by taking annual means, the results based upon the mean annual data are in excellent agreement with the results obtained using the mean monthly data. Using any of the above expressions will result in a good estimate of the TKN removal that is likely to occur in diffused-air aerated ponds.

The relationships developed to predict ammonia nitrogen removal yielded highly significant (1 percent level) relationships for all of the equations presented in Table 2-7. However, the agreement between the calculated detention times for ammonia nitrogen removal differed significantly from that observed for the TKN data. This variation is not surprising in view of the many mechanisms involved in ammonia nitrogen production and removal in wastewater ponds, but this variation in results does complicate the use of the equations to estimate ammonia nitrogen removal in aerated ponds. Statistically, a justification exists to use either of the expressions in Table 2-7 to calculate the detention time required to achieve a given percentage reduction in ammonia nitrogen.

2.5 Disinfection

2.5.1 Introduction

Since chlorine, at present, is less expensive and offers more flexibility than other means of disinfection, chlorination is the most practical method of disinfection. Basic principles of chlorination are presented elsewhere (7)(29)(39).

2.5.2 Effects of Chlorinating Pond Effluents

White (31) suggested that chlorine demand increases with high concentrations of algae commonly found in pond effluents. A chlorine dose of 20-30 mg/l was required to satisfy chlorine demand and to produce enough residual to effectively disinfect algae-laden wastewater within 30-45 minutes. Kott (32) reported increases in demand as a result of algae, but found that a chlorine dose of 8 mg/l was sufficient to produce adequate disinfection within 30 minutes. If contact times are kept relatively short, no serious chlorine demand by algae cells is encountered (32). For pond effluents, a chlorine demand of only 2.6 to 3.0 mg/l was exerted after 20 minutes of contact (33).

At low chlorine doses, very little increase in chlorine demand is attributable to algae, but at higher doses, the destruction of algal cells greatly increases demand (34). This occurs because dissolved organic compounds released from destroyed algal cells are oxidized by chlorine and thus increase chlorine demand (35).

Another concern regarding the chlorination of pond effluents are the effects on BOD and COD. Conflicting results have been reported (32-37), indicating that either an increase or decrease in BOD and COD occurred with increased chlorine concentrations. A conclusion would be that the effect of chlorination on BOD and COD is a function of wastewater characteristics, chlorine application methods and contact time, and other undefined parameters.

The formation of toxic chloramines is also of concern in chlorinating pond effluents. These compounds are found in waters high in ammonium nitrogen concentration and are extremely toxic to aquatic life found in receiving water. For example, a chloramine concentration of 0.06 mg/l is lethal to trout (38).

Not all of the side effects of chlorination pond effluents are detrimental. Kott (39) observed reductions of SS as a result of chlorination. Reductions of volatile suspended solids (VSS) by as much as 52 percent and improved water clarity (reduced turbidity) by 32 percent were observed following chlorination (33). Chlorine may enhance the flocculation of algae masses by causing algal cells to clump together (35).

Four systems of identically designed chlorine mixing and contact tanks, each capable of treating 190 m^3/d (50,000 gpd), were used to study the chlorination of pond effluents (40). Three of the four chlorination systems were used for directly treating the pond effluent. The effluent treated in the fourth system was filtered through an intermittent sand filter to remove algae prior to chlorination. The filtered effluent was also used as the solution water for all four chlorination systems.

Following recommendations of others (41-45), the chlorination systems were constructed to provide rapid initial mixing followed by chlorine contact in plug flow reactors. A serpentine flow configuration having a length to width ratio of 25:1, coupled with inlet and outlet baffles, was used to enhance plug flow hydraulic performance. The maximum theoretical detention time for each tank was 60 minutes, while the maximum actual detention time averaged about 50 minutes.

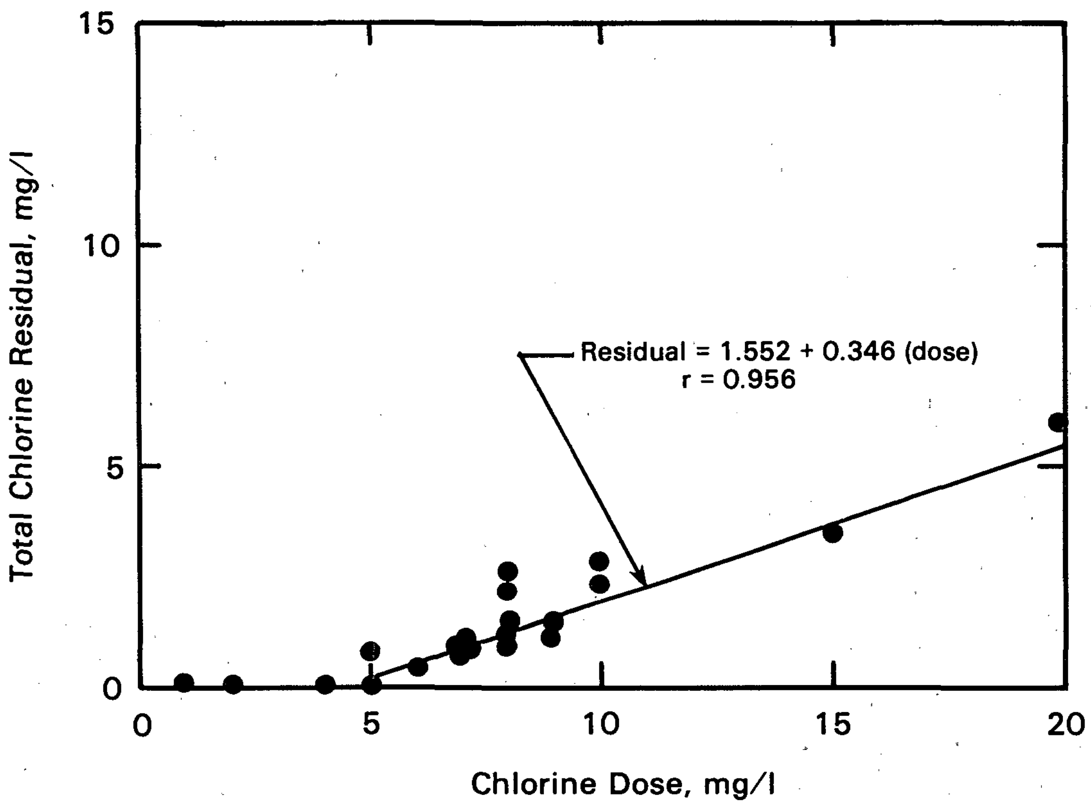
The pond effluent was chlorinated at doses ranging from 0.25 to 30.0 mg/l under a variety of contact times, temperatures, and seasonal effluent conditions from August 1975 to August 1976. Some of the major findings of this study are summarized below.

1. Sulfide, produced as a result of anaerobic conditions in the ponds during winter months when the ponds are frozen over, exerts a significant

chlorine demand (Figure 2-19). For sulfide concentrations of 1.0 to 1.8 mg/l, a chlorine dose of 6 to 7 mg/l was required to produce the same residual as a chlorine dose of about 1 mg/l for conditions with no sulfide.

FIGURE 2-19

CHLORINE DOSE vs. RESIDUAL FOR INITIAL SULFIDE CONCENTRATIONS OF 1.0-1.8 mg/l

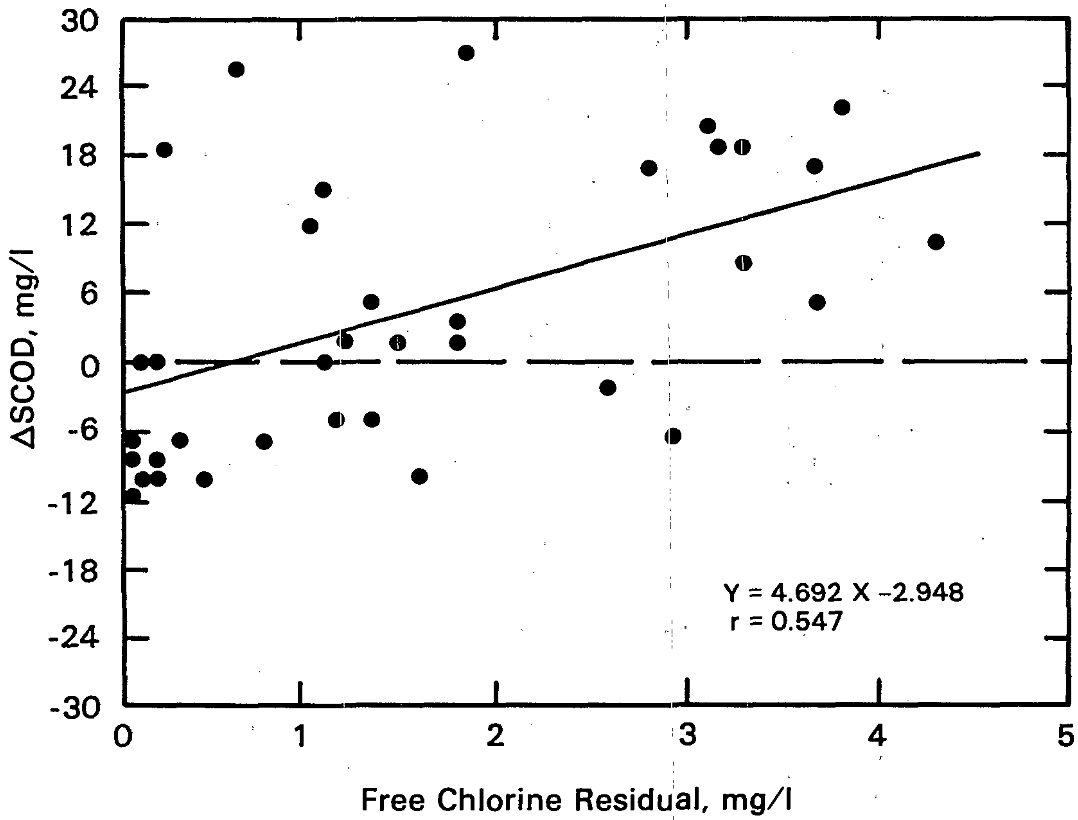


2. For all concentrations of ammonia encountered, adequate disinfection could be obtained with combined chlorine residual in 50 minutes or less of contact time. Therefore, breakpoint chlorination, and the subsequent production of free chlorine residual, was rarely, if ever, necessary in disinfecting pond effluent.

3. Total COD concentration in a pond effluent was virtually unaffected by chlorination. Soluble COD increased with increasing concentrations of free chlorine only. This increase was attributed to the oxidation of SS by free chlorine. Increases in soluble COD vs. free chlorine residual are shown in Figure 2-20.

FIGURE 2-20

CHANGES IN SOLUBLE COD vs. FREE CHLORINE RESIDUAL - UNFILTERED POND EFFLUENT



4. Some reduction in SS, due to the breakdown and oxidation of suspended particulates, and resulting increases in turbidity were attributed to chlorination. However, this reduction was of limited importance when compared with reductions of SS resulting from settling. SS were reduced by 10 to 50 percent by settling in the contact tanks.

5. Filtered pond effluent exerted a lower chlorine demand than unfiltered pond effluent, due to the removal of algae (Figure 2-21). The rate of exertion of chlorine demand was directly related to chlorine dose and total chemical oxygen demand.

6. A summary of total coliform removal efficiencies as a function of a total chlorine residual for filtered and unfiltered effluent is illustrated in Figure 2-22. The rate of disinfection was a function of the chlorine dose

FIGURE 2-21

CHLORINE DOSE vs. TOTAL RESIDUAL - FILTERED AND UNFILTERED POND EFFLUENT

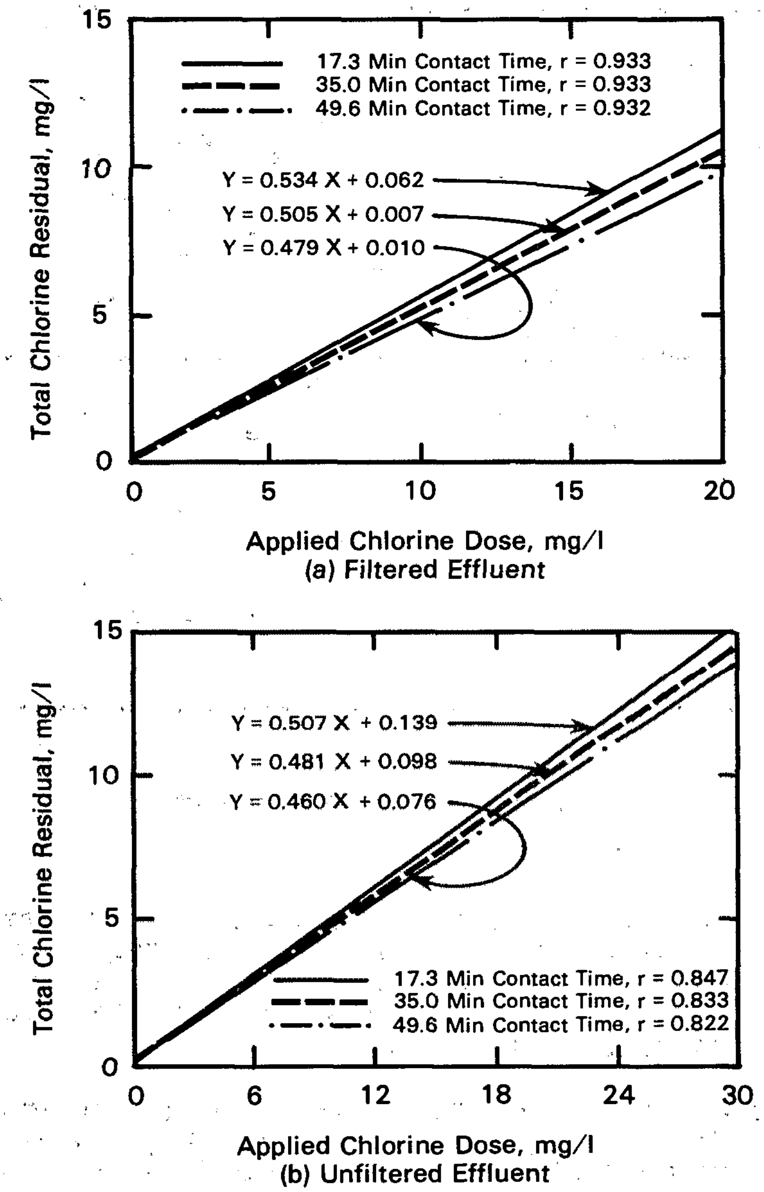
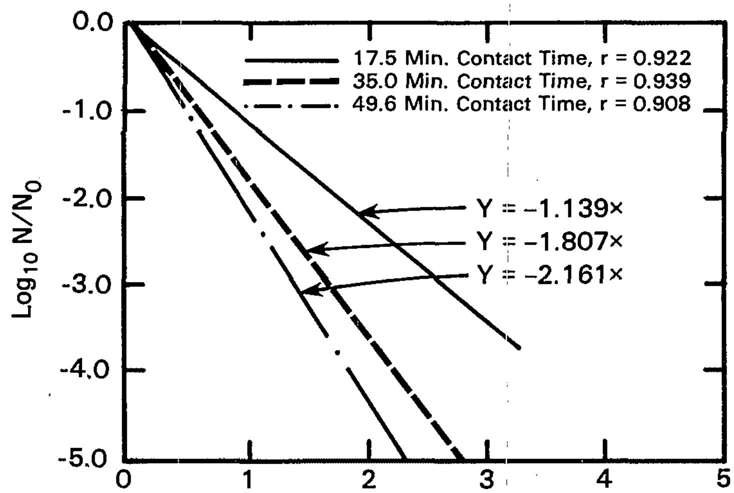
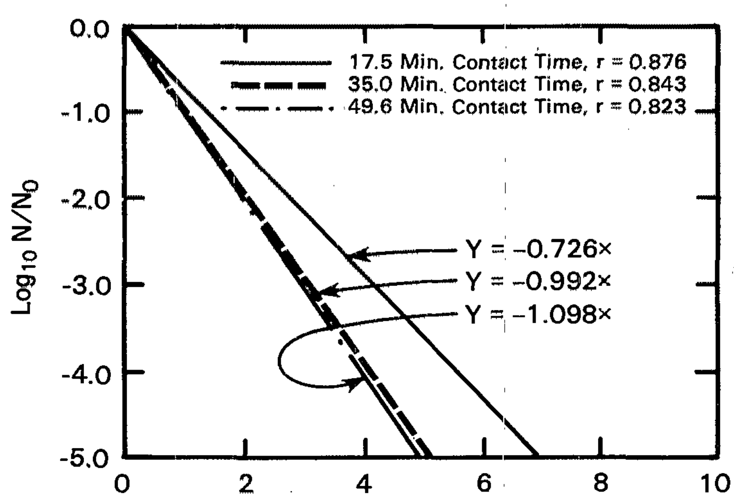


FIGURE 2-22

TOTAL COLIFORM REMOVAL EFFICIENCIES - FILTERED AND UNFILTERED POND EFFLUENT



Total Chlorine Residual, mg/l
(a) Filtered Effluent



Total Chlorine Residual, mg/l
(b) Unfiltered Effluent

and bacterial concentration. Generally, the chlorine demand was about 50 percent of the applied chlorine dose except during periods of sulfide production.

7. Disinfection efficiency was temperature dependent. At colder temperatures, the reduction in the rate of disinfection was partially offset by reductions in the exertion of chlorine demand; however, the net effect was a reduction in the chlorine residual necessary to achieve adequate disinfection with increasing temperature for a specific contact period.

8. In almost all cases, adequate disinfection was obtained with combined chlorine residuals of between 0.5 and 1.0 mg/l after a contact period of approximately 50 minutes. This indicated that disinfection can be achieved without discharging excessive concentrations of toxic chlorine residuals into receiving waters.

2.5.3 Predicting Required Residuals

Using the data from the study summarized in section 2.5.2, Johnson et al. (40) developed a model to predict the chlorine residual required to obtain a specified bacterial kill. The model was used to construct a series of design curves for selecting chlorine doses and contact times for achieving desired levels of disinfection. An example may best illustrate how these design curves are applied.

Assume that a particular lagoon effluent is characterized as having a fecal coliform (FC) concentration of 10,000/100 ml, 0 mg/l sulfide, 20 mg/l TCOD, and a temperature of 5°C. If it is necessary to reduce the FC counts to MPN of 100/100 ml, or a 99 percent bacterial reduction, and an existing chlorine contact chamber has an average residence time of 30 minutes, the required chlorine residual is obtained from Figure 2-23. A 99 percent bacterial reduction corresponds to log (N_0/N) equal to 2.0. For a contact period of 30 minutes, a combined chlorine residual of 1.3 mg/l is required. This is indicated by Point 1 in Figure 2-23.

Going to Figure 2-24, it is determined that if a chlorine dose produces a residual of 1.30 mg/l at 5°C, the same dose would produce a residual of 0.95 mg/l at 20°C. This is because of the faster rate of reaction between TCOD and chlorine at the higher temperature. This is indicated by Point 2 in Figure 2-24. For an equivalent chlorine residual of 0.95 mg/l at 20°C and 20 mg/l TCOD, it is determined from Figure 2-25 that the same chlorine dose would produce a residual of 0.80 mg/l if the TCOD were 60 mg/l. This is because higher concentrations of TCOD increase the rate of chlorine demand. Point 3 on Figure 2-25 corresponds to this residual. The chlorine dose required to produce an equivalent residual of 0.80 mg/l at 20°C and 60 mg/l TCOD is determined from Figure 2-26. For a chlorine contact period of 30 minutes, a chlorine dose of 2.15 mg/l is necessary to produce the desired combined residual as indicated by Point 4 on Figure 2-26. This dose will produce a reduction in FC from 10,000/100 ml to 100/100 ml within 30 min at 5°C and with 20 mg/l TCOD.

FIGURE 2-23

COMBINED CHLORINE RESIDUAL AT 5°C FOR COLIFORM = 10^4/100 ml

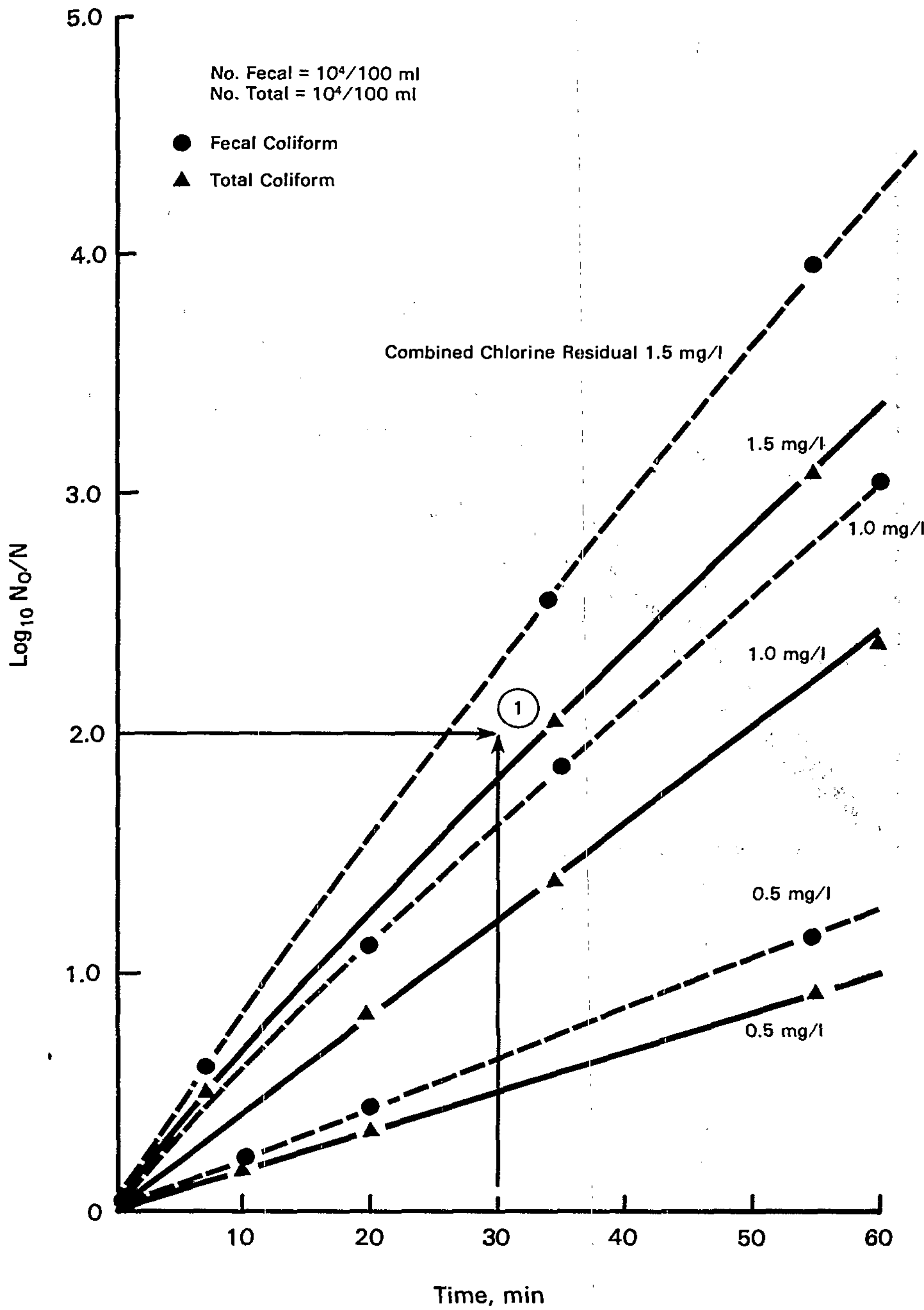


FIGURE 2-24

CONVERSION OF COMBINED CHLORINE RESIDUAL AT TEMP. 1
TO EQUIVALENT RESIDUAL AT 20°C

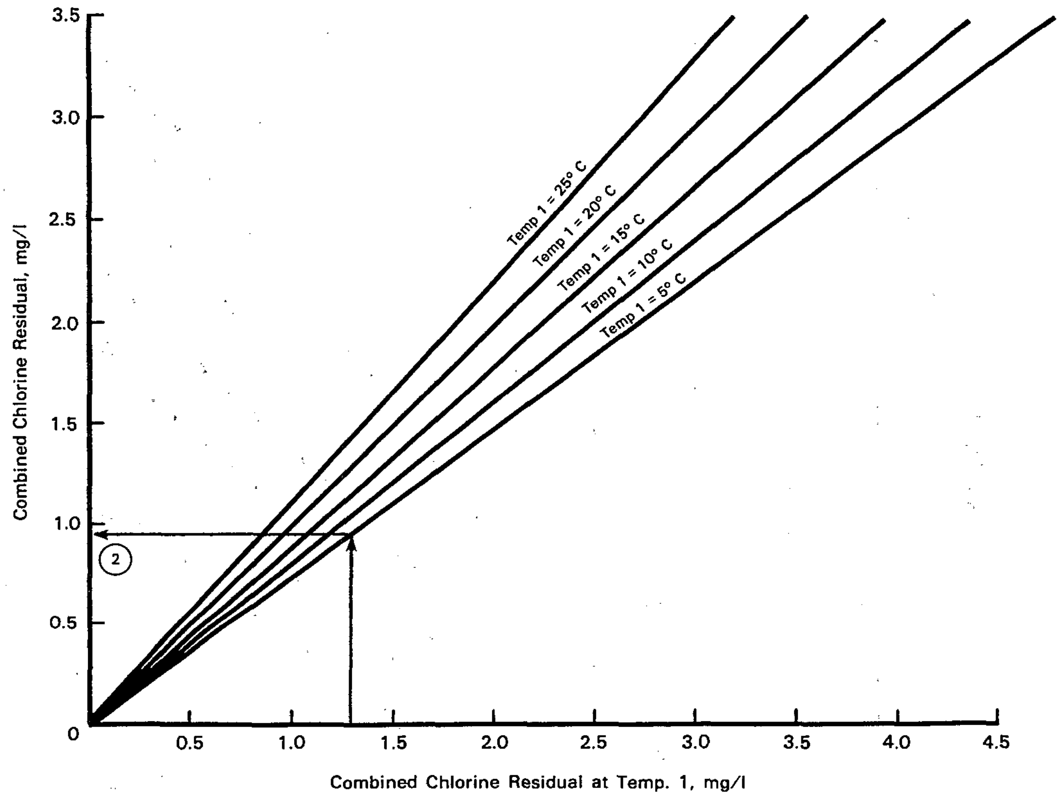


FIGURE 2-25

CONVERSION OF COMBINED CHLORINE RESIDUAL AT TCOD 1 AND 20°C
TO EQUIVALENT RESIDUAL AT TCOD = 60 mg/l AND 20°C

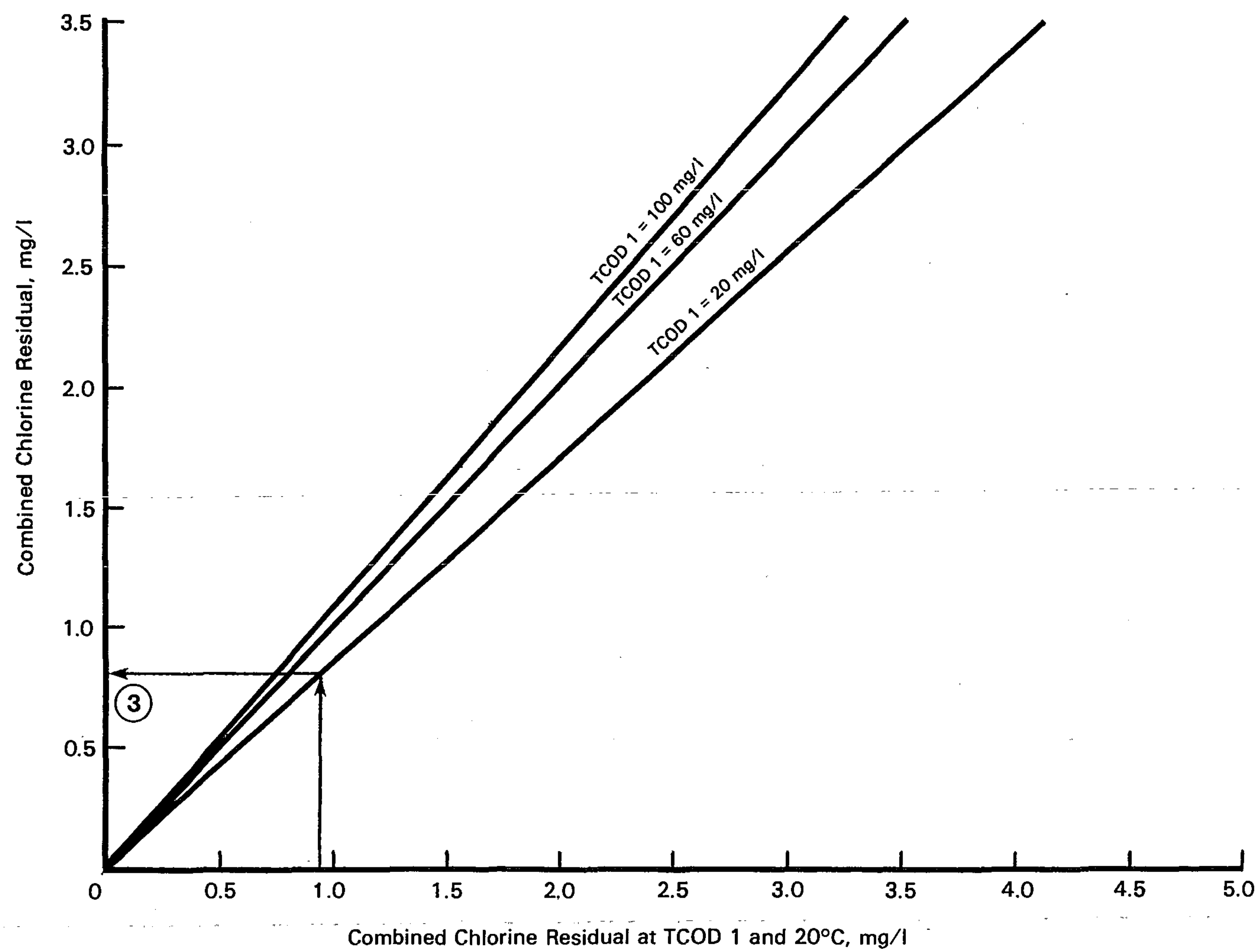
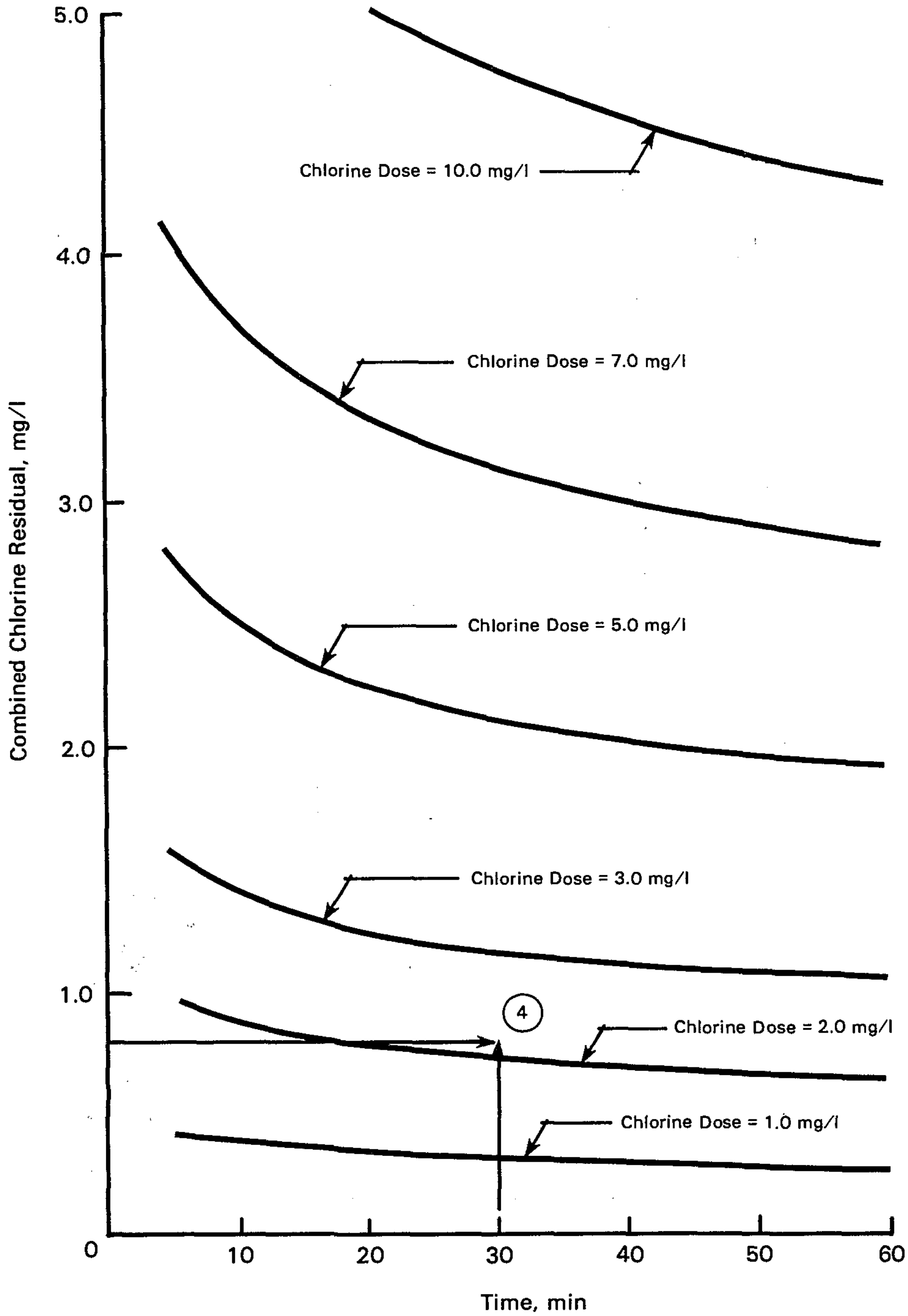


FIGURE 2-26

DETERMINATION OF CHLORINE DOSE REQUIRED FOR EQUIVALENT COMBINED RESIDUAL AT TCOD = 60 mg/l AND 20°C



If, in the previous example, the initial sulfide concentration were 1.0 mg/l instead of 0 mg/l, it would be necessary to go directly from Figure 2-23 to Figure 2-27. Here, chlorine residual of 1.30 mg/l at the TCOD of 20 mg/l and a temperature of 5°C is converted to an equivalent chlorine residual of 1.10 mg/l for a TCOD of 60 mg/l and 5°C. This is represented by Point 5 on Figure 2-27. Going to Figure 2-28, which corresponds to an initial sulfide concentration of 1.0 mg/l, it is determined that a chlorine dose of 6.6 mg/l is necessary to produce an equivalent chlorine residual of 1.1 mg/l after a contact period of 30 min. Point 6 on Figure 2-28 corresponds to this dose. The sulfide remaining after chlorination is determined to be 0.4 mg/l from Figure 2-29 as indicated by Point 7.

2.5.4

The primary objective of good chlorine contact tank design is to design for hydraulic performance which will allow for a minimum usage of chlorine with a maximum exposure of microorganisms to the chlorine. An evaluation of a number of chlorine contact tanks indicates that mixing, detention time, and chlorine dosage are the critical factors to consider in providing adequate disinfection. Good design not only optimizes disinfection efficiency, but should also minimize the concentration of undesirable compounds being discharged to the environment and reduce the accumulation of solids in the tank by keeping the flow-through velocity high enough to prevent solids from settling (46). A discussion of chlorine contact chamber hydraulic characteristics is presented elsewhere (9)(30). Table 2-8 presents a summary of chlorination design criteria.

2.6 Odor Control

2.6.1 Introduction

Odors are usually created at wastewater ponds because they are overloaded, excessive surface scum has been allowed to accumulate, or aquatic and pond slope weeds are completely uncontrolled. All three of these causes can be eliminated by adequate design, including design features for effective operation and maintenance.

2.6.2 Overloading

Process design considerations are discussed in Chapter 3. Careful design which incorporates the requirements set forth in that chapter should eliminate pond overloading. These requirements include (1) selection of loading criteria applicable to the influent loads and the operational limitations created by local land use and climatic conditions, and (2) design of a layout which assures the effective utilization of all pond volume. Dead areas which do not maintain adequate circulation or flow must be eliminated.

FIGURE 2-27

CONVERSION OF COMBINED CHLORINE RESIDUAL AT TCOD 1 AND 5°C TO EQUIVALENT RESIDUAL AT TCOD = 60 mg/l AND 5°C

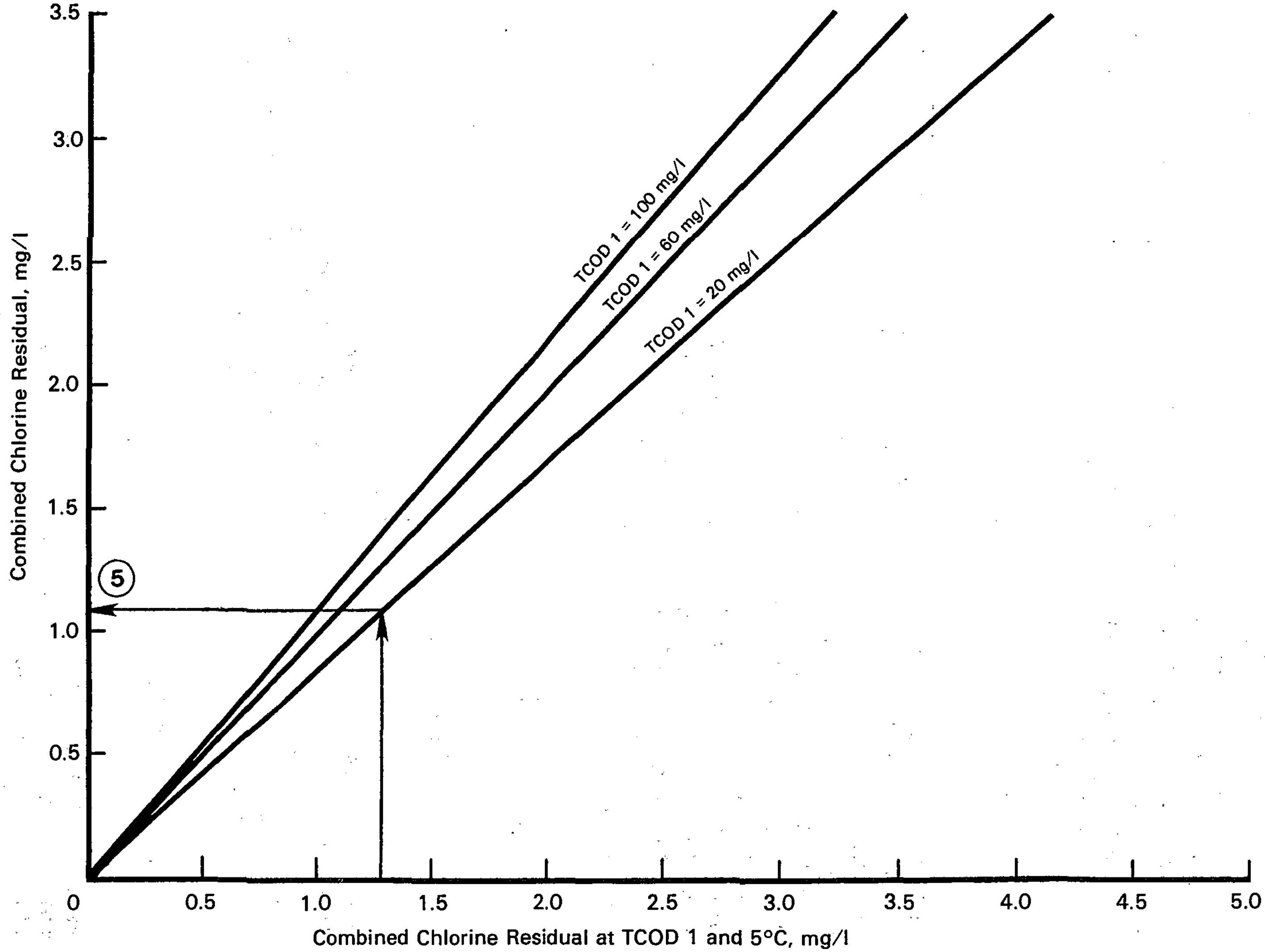


FIGURE 2-28

DETERMINATION OF CHLORINE DOSE REQUIRED WHEN S = 1.0 mg/l, TCOD = 60 mg/l, AND TEMP. = 5°C

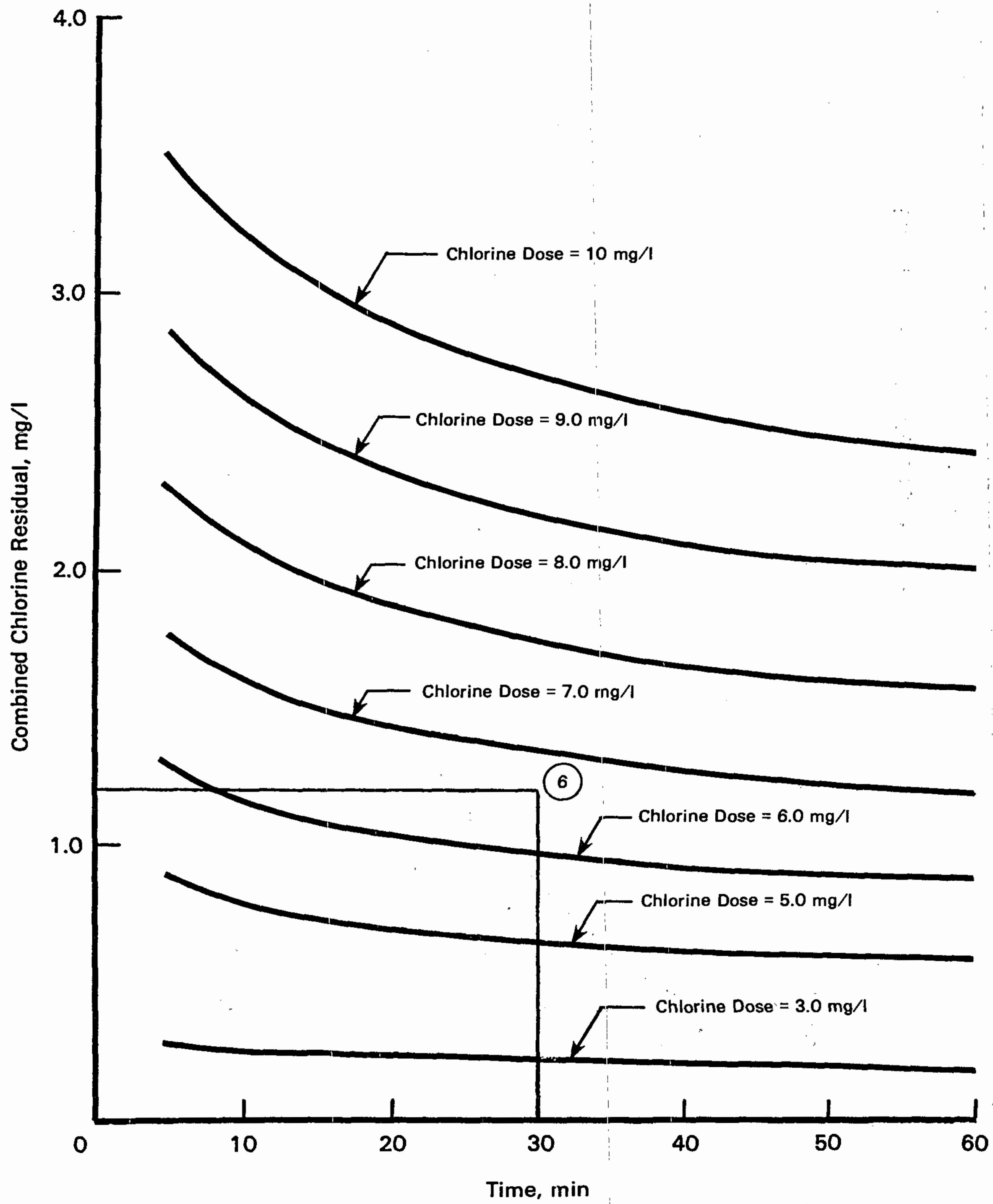


FIGURE 2-29

SULFIDE REDUCTION AS A FUNCTION OF CHLORINE DOSE

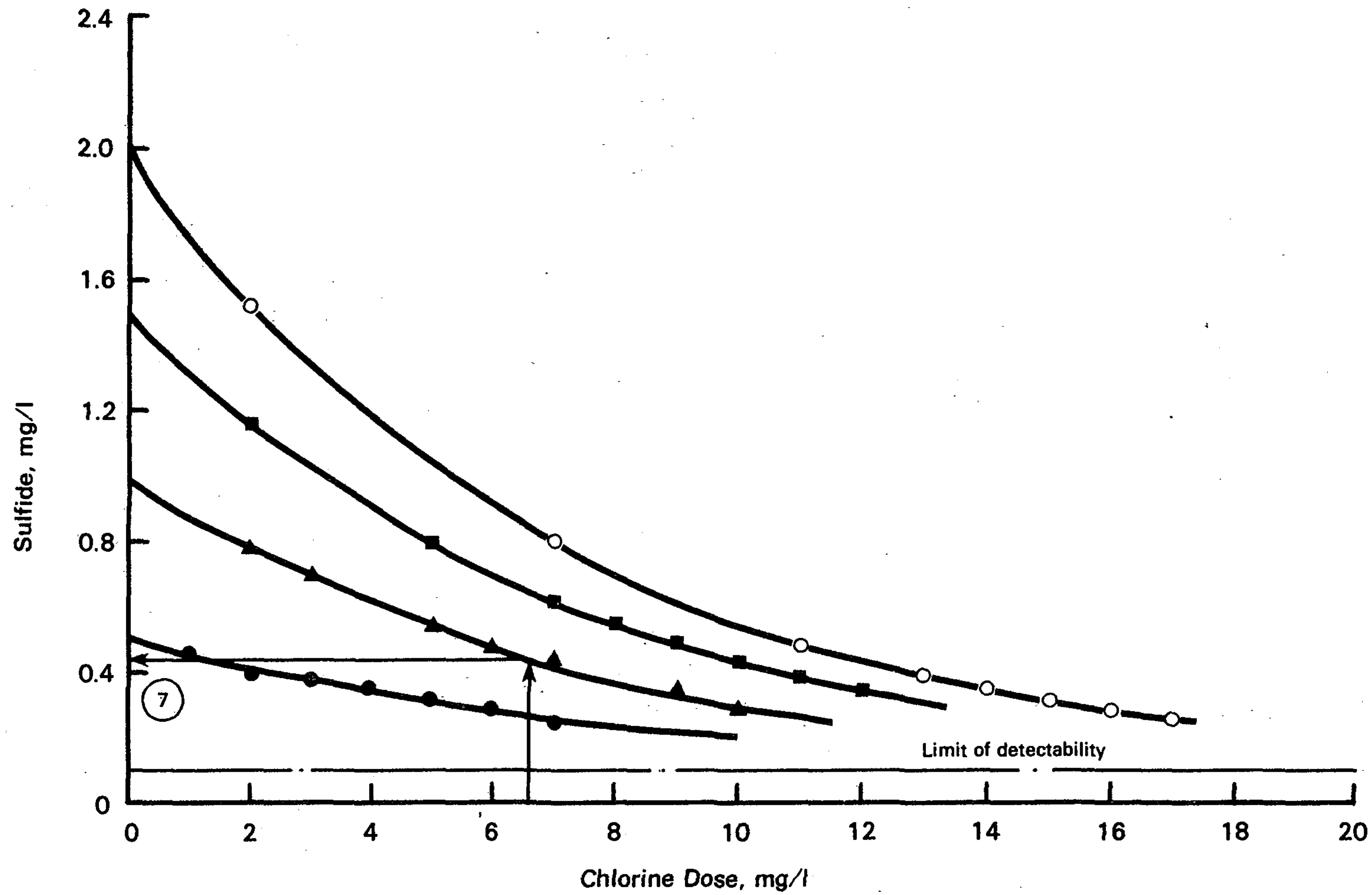


TABLE 2-8

SUMMARY OF CHLORINATION DESIGN CRITERIA

Mixing

I. Rapid initial mixing should be accomplished within 5 sec and before liquid enters contact tank. Design hydraulic residence time $\geq$ 30 sec for tanks using mechanical mixers.

II. Methods available
1. Hydraulic jump in open channels.
2. Mechanical mixers located immediately below point of chlorine application.
3. Turbulent flow in a restricted reactor.
4. Pipe flowing full. Least efficient and should not be used in pipes with diameter > 76 cm (> 30 in).

Contact Chamber

I. Hydraulic residence time
1. $\geq$ 60 minutes at average flow rate.
2. $\geq$ 30 minutes at peak hourly flow rate.

II. Hydraulic performance
1. Model value obtained in dye studies $\geq$ 0.6, $t_p/T \geq 0.6$.
2. Efficiency of disinfection increases as t_p/T increases.
3. Design for maximum economical t_p/T.

III. Length to width ratio
1. L/W $\geq$ 25:1.
2. Cross-baffles used to eliminate short circuiting caused by wind.

IV. Solids removal
1. Baffles arranged to remove floating solids.
2. Provide drain to remove solids and liquid for maintenance.
3. Provide duplicate contact chambers.
4. Width between channels should be adequate for easy access to clean and maintain chamber.

V. Storage of chlorine
1. Provide a minimum of one filled chlorine cylinder for each one in service.
2. Maintain storage area at a temperature $\geq$ 13°C ($\geq$ 55°F).
3. Never locate cylinders in direct sunlight or apply direct heat.

---

tp = Time for tracer at outlet of contact tank to reach peak concentration.
T = Volume of contact tank/flow rate = theoretical detention time.
L = Length of contact tank.
W = Width of contact tank.

TABLE 2-8 (continued)

4. Limit maximum withdrawal rate from 45- and 68-kg (100- and 150-lb) cylinders to 18 kg/d (40 lb/d).
5. Limit maximum withdrawal rate from 909-kg (2,000-lb) cylinders to 182 kg/d (400 lb/d).
6. Provide scales to weigh cylinders.
7. Provide cylinder handling equipment.
8. Install automatic switchover sytem.

VI. Piping and valves

1. Use Chlorine Institute approved piping and valves.
2. Supply piping between cylinder and chlorinator should be Sc. 80 black seamless steel pipe with 2,000-lb forged steel fitting. Unions should be ammonia type with lead gaskets.
3. *Chlorine solution lines should be Sc. 80 PVC, rubberlined* steel, Saran-lined steel, or fiber cast pipe approved for moist chlorine use. Valves should be PVC, rubber lined, or PVC lined.
4. Injector line between chlorinator and injector should be Sc. 80 PVC or fiber cast approved for moist chlorine use.

VII. Chlorinators

1. Should be sized to provide dosage $\geq$ 10 mg/l.
2. Maximum feed rate should be determined from knowledge of local conditions.
3. Direct feed gas chlorinators should be used only in small installations. Check state regulations. Prohibited in certain states.
4. Vacuum feed gas chlorinators are most widely used and are much safer.
5. Hypochlorite solutions should be considered in installations where safety is major concern.

VIII. Safety equipment and training

1. Install an exhaust fan near floor level with switch actuated when door is opened.
2. Exhaust fan should be capable of one air exchange per minute.
3. Gas mask located outside chlorination room.
4. Emergency chlorine container repair kits.
5. Chlorine leak detector.
6. Alarms should be installed to alert operator when deficiencies or hazardous conditions exist.
7. Operator should receive detailed hands-on training with all emergency equipment.

IX. Diffusers

1. Minimum velocity through diffuser holes $\geq$ 3-4 m/sec ($\geq$ 10-12 ft/sec).
2. Diffusers should be installed for convenient removal, cleaning, and replacement.

Whenever possible, influent loadings should be shared with other cells by means of forced recirculation. Mechanical aeration should be used to supplement natural photosynthesis whenever conservative loading rates cannot be applied. The intermittent operation of an influent cell mechanical aeration unit can often be adjusted to compensate for conditions which could not be anticipated during design. Its availability can then be an excellent tool to eliminate odors. Temporary relief from odors can be obtained by applying sodium nitrate to the pond influent on spreading over the surface. Details on the application of sodium nitrate, other chemicals, and other methods of odor control are presented in Reference (47).

2.6.3 Scum Accumulation

Scum often accumulates on pond surfaces from debris which enters from the influent sewer, dead or decaying algae which remains buoyant, and debris which enters from surrounding areas. Such surface scum often decomposes without sinking if the surface is quiescent or the scum attaches itself to riprap, floating debris, or aquatic growth. Small clumps of scum, spread over a fairly large surface, will not usually create sufficient odor levels to be an offsite nuisance.

One effective way of eliminating stabilization pond scum buildup is by providing a means of mechanical agitation. Another means of eliminating scum accumulation is through the use of recirculation. The Sunnyvale, CA, secondary oxidation ponds have been receiving a major portion of that plant's stabilized sludge over the past several years (48). The pond recirculating system is designed to maintain mixing throughout, and the pond has never experienced any significant scum accumulation in any of its supply and return channels or cells.

2.6.4 Scum Attachment

One of the most significant effects of aquatic plants is their ability to support scum accumulations. If a pond were heavily loaded, the resulting scum would certainly have no chance of dissipating, and odors would result. Good housekeeping, which means control of aquatic weeds and berm weeds, is essential to odor control. Raw wastewater ponds where scum accumulation is expected should not have riprap which allows scum to accumulate in cracks and crevices. A concrete or asphalt apron can be used to protect the embankment where scum accumulation is expected.

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CHAPTER 3

DESIGN PROCEDURES

3.1 Preliminary Treatment

In general the only mechanical or monitoring and control equipment required for wastewater pond systems are flow measurement devices, sampling systems, and pumps. The flow diagrams presented in Chapter 2 illustrate the variety of preliminary treatment options in use. Design criteria and examples for preliminary treatment components are presented in several other publications (1-6) as well as in equipment manufacturer's catalogs. Flow measurement can be accomplished with relatively simple devices such as Palmer-Bowlus flumes, V-notch weirs, and Parshall flumes used in conjunction with a recording meter. Frequently, flow measurements and 24-hour compositing samplers are combined in a common manhole, pipe, or other housing arrangement. If pumping facilities are necessary, the wet well is sometimes used as a point to recycle effluent or to add chemicals for odor control. Pretreatment facilities should be kept to a minimum at pond systems.

3.2 Facultative Ponds

Facultative pond design is based upon BOD removal; however, the majority of the suspended solids will be removed in the primary cell of a pond system. Sludge fermentation feedback of organic compounds to the water in a pond system is significant and has an effect on the performance. During the spring and fall, overturn of the pond contents can result in significant quantities of solids being resuspended. The rate of sludge accumulation is affected by the liquid temperature, and additional volume is added for sludge accumulation in cold climates. Although SS have a profound influence on the performance of pond systems, most design equations simplify the incorporation of the influence of SS by using an overall reaction rate constant. Effluent SS generally consist of suspended organism biomass and do not include suspended waste organic matter.

Several empirical and rational models for the design of facultative wastewater ponds have been proposed. These relationships include the ideal plug flow and complete mix models, as well as models proposed by

Fritz et al., Gloyna, Larson, Marais, McGarry and Pescod, Oswald et al., and Thirumurthi (7-14). Of these models, several produce satisfactory results; however, use may be restricted because of the difficulty in evaluating specific coefficients or by the model complexity. Equations from these sources are presented in the Appendix.

Because of the many approaches to the design of facultative ponds, an attempt will not be made to select the "best" procedure. An evaluation of several design formulas with the operational data presented in Chapter 2 failed to show that any of the methods are superior to the others in terms of predicting the performance of pond systems (see Appendix). The design methods most often referenced are summarized in Table 3-1. Each of these will be used to design a facultative pond for the domestic wastewater described in Table 3-2. Following the calculations of the size of the pond system by each method, a summary and comparison of the results will be presented.

3.2.1 Areal Loading Rate Procedure

The BOD_5 areal loading rate recommended for an average winter air temperature of less than 0°C is 11-22 kg/ha/d (10-20 lb/ac/d) (Table 3-1). The more extreme the environment, the lower the loading rate. In this hypothetical case, an intermediate BOD_5 loading rate of 17 kg/ha/d and a minimum hydraulic detention time of 180 days was selected. The minimum hydraulic detention time of 180 days was selected because of the severe climatic conditions and long periods of ice cover when effluent can not be discharged. The detention time must be selected in consideration of the climatic conditions and values may range from no minimum value specified up to 180 days as used in this example. The organic loading rate on the first cell in the system will be limited to 40 kg BOD_5/ha/d to avoid overloading and anaerobic conditions and odors.

TABLE 3-1

FACULTATIVE POND DESIGN EQUATIONS

	AREAL LOADING RATE (15)	GLOYNA EQUATION (8)
Design Equation or Parameter	For Avg. Winter Air Temperature of above 15°C (60°F) BOD_5 Loading = 45-90 kg/ha/d (40-80 lb/ac/d) For Avg. Winter Air Temperature of 0-15°C (32-60°F) BOD_5 Loading = 22-45 kg/ha/d (20-40 lb/ac/d) For Avg. Winter Air Temperature of below 0°C (32°F) BOD_5 Loading = 11-22 kg/ha/d (10-20 lb/ac/d) BOD_5 Loading in first cell is usually limited to 40 kg/ha/d or less and the hydraulic detention time is 120 to 180 days in climates where the average air temperature is below 0°C. In mild climates (air temp. > 15°C) loadings on the primary cell can be 100 kg/ha/d	$\frac{V}{Q} = t = 0.035 L_a \theta^{(35-T)} ff'$ V = pond volume, m^3 Q = influent flow rate, l/d t = hydraulic residence time, d L_a = ultimate influent BOD or COD, mg/l θ^a = temperature coefficient = 1.085 T = pond water temperature, °C f = algal toxicity factor f' = sulfide oxygen demand factor BOD removal efficiency = 80-90% f = 1.0 for domestic wastes f' = 1.0 for SO_4 < 500 mg/l Depth = 1 m for calculation of surface loading Depth varies with climate Depth = 1 m for ideal conditions, i.e., uniform temp., tropical to sub-tropical, min. settleable solids. Depth = 1.25 m for same condition as above but with modest amounts of settleable solids. Depth = 1.5 m for locations with significant seasonal variation in temperatures. Depth = 1.5-2 m for severe climates.
Temperature Adjustment of Parameters	Given above	Included in equation

TABLE 3-1

CONTINUED

<table>
<tr><th></th><th>MARAIS & SHAW EQ.
(COMPLETE MIX MODEL)
(10)</th><th>PLUG FLOW
MODEL
(16) (17)</th><th>WEHNER-WILHELM EQ. &
THIRUMURTHI APPLICATION
(13) (14) (18)</th></tr>
<tr><td>Design Equation
or
Parameter</td>
<td>$\frac{C_n}{C_o} = \left[\frac{1}{1 + k_c t_n}\right]^n$

C_n = effluent BOD_5 concentration, mg/l
C_o = influent BOD_5 concentration, mg/l
k_c = complete mix 1st order reaction rate, $days^{-1}$
t_n = hydraulic residence time in each pond, days
n = number of ponds in series

$(C_e)_{max} = \frac{700}{0.6d + 8}$

$(C_e)_{max}$ = maximum pond BOD_5 conc. consistent with aerobic conditions, mg/l
d = depth of pond, ft
Max. efficiency in a series of ponds is obtained when t_n in each pond is equal.</td>
<td>$\frac{C_e}{C_o} = e^{-k_p t}$

C_e = effluent BOD_5 concentration, mg/l
C_o = influent BOD_5 concentration, mg/l
e = base of natural logarithms, 2.7183

k_p = plug flow first order reaction rate, day^{-1}
t = hydraulic residence time, days
k_p varies with the BOD_5 loading rate rate as shown below:

<table>
<tr><th>BOD_5 Loading Rate
kg/ha/d</th><th>$k_{P_{20}}$
day^{-1}</th></tr>
<tr><td>22</td><td>0.045</td></tr>
<tr><td>45</td><td>0.071</td></tr>
<tr><td>67</td><td>0.083</td></tr>
<tr><td>90</td><td>0.096</td></tr>
<tr><td>112</td><td>0.129</td></tr>
</table></td>
<td>$\frac{C_e}{C_o} = \frac{4\,ae^{1/2D}}{(1+a)^2 e^{a/2D} - (1-a)^2 e^{-a/2D}}$

C_o = influent BOD_5 concentration, mg/l
C_e = effluent BOD_5 concentration, mg/l
e = base of natural logarithms, 2.7183

a = $\sqrt{1 + k\ t\ D}$
k = 1st order reaction rate, day^{-1}
t = hydraulic residence time, d
D = dimensionless dispersion number

$D = \frac{H}{vL} = \frac{Ht}{L^2}$

H = axial dispersion coef., area per time
v = fluid velocity, length per time
L = length of travel path of a typical particle, length</td></tr>
<tr><td>Temperature
Adjustment of
Parameters</td>
<td>$k_{c_T} = k_{c_{35}} (1.085)^{T-35}$

k_{c_T} = reaction rate at min. operating water temperature
k_{c35} = reaction rate at 35°C = 1.2 day^{-1}
T = minimum operating water temperature, °C</td>
<td>$k_{P_T} = k_{P_{20}} (1.09)^{T-20}$

k_{P_T} = reaction rate at min. operating water temperature
$k_{P_{20}}$ = reaction rate at 20°C
T = minimum operating water temperature, °C</td>
<td>$k_T = k_{20} (1.09)^{T-20}$
k_T = reaction rate at minimum operating water temperature
k_{20} = reaction rate at 20°C = 0.15 day^{-1}

T = minimum operating water temperature, °C</td></tr>
</table>

TABLE 3-2

ASSUMED CHARACTERISTICS OF WASTEWATER AND ENVIRONMENTAL CONDITIONS FOR FACULTATIVE POND DESIGN

Q = Design flow rate = 1893 m^3/d (0.5 mgd)

C_o = Influent BOD_5 = 200 mg/l

C_e = Effluent BOD_5 = 30 mg/l

T = Water temperature at critical period of year = 0.5°C

T_a = Average winter air temperature = <0°C

Light Intensity = adequate

Evaporation = rainfall

Suspended Solids = 250 mg/l

$SO_4^{=}$ = <500 mg/l

Design Conditions: See Table 3-2.

Requirements: Size a facultative wastewater treatment pond to treat the wastewater described in Table 3-2, and specify the following parameters for the system.

1) Detention time in total system and first cell, t and t_1

2) Volume in total system and first cell, V and V_1

3) Surface area in total system and first cell, A and A_1

4) Depth, d

5) Length, L

6) Width, W

Solution:

$$BOD_5 \text{ Loading} = 200 \text{ mg/l } (1893 \text{ m}^3/\text{d})\left(\frac{1000 \text{ liters}}{\text{m}^3}\right)\left(\frac{\text{kg}}{1 \times 10^6 \text{ mg}}\right)$$

$$BOD_5 \text{ Loading} = 379 \text{ kg/d}$$

$$\text{Surface area required in first cell} = A_1 = \frac{379 \text{ kg/d}}{40 \text{ kg/ha/d}}$$

$$A_1 = 9.5 \text{ ha} = 95{,}000 \text{ m}^2 \text{ (23.4 ac)}$$

$$\text{Total surface area required} = A = (379 \text{ kg/d})/(17\text{kg/ha/d})$$

$$A = 22.3 \text{ ha} = 223{,}000 \text{ m}^2 \text{ (55.1 ac)}$$

The surface area of 95,000 m^2 (23.4 ac) required in the primary cells is larger than normally provided in one cell; therefore, the system will be divided into two parallel systems with a surface area of 47,500 m^2 (11.7 ac) in each primary cell. The remaining surface area requirement will be divided into two parallel systems with three equal size cells in series in each parallel system.

$$\text{Surface area in secondary cell} = A_2 = (223{,}000 - 95{,}000)/ \\ \text{(2 parallel systems)(3 cells in series)}$$

$$A_2 = A_3 = A_4 = 21{,}330 \text{ m}^2$$

Using a length to width ratio of 3:1 and an embankment slope of 4:1, the dimensions of the cells at the water surface and at maximum depth can be calculated as follows:

$A_1 = L_1 \times W_1 = L(L/3) = L^2/3 = 47{,}500\ m^2$

$L_1 = 378$ m (1240 ft)

$W_1 = 378/3 = 126$ m (413 ft)

$A_2 = L^2/3 = 21{,}330\ m^2$

$L_2 = 253$ m (830 ft)

$W_2 = 84$ m (277 ft)

Depth selection is usually controlled by state standards, and depths specified will range from 1.0 to 2.1 m (3 to 7 ft) in the primary pond to 2.5 to 3.0 m (8 to 10 ft) in secondary ponds.

Let the depth of primary pond = 2 m (6.6 ft)

Depth includes 0.3 m (1 ft) for ice cover and 0.3 m (1 ft) for sludge storage.

The "effective" depth in the primary cell = 1.4 m (4.6 ft).

Depth selection in the remaining cells is also controlled by state standards, and most states will allow greater depths in the secondary cells.

Let the depth of the other cells = 3 m (10 ft)

Depth includes 0.3 m (1 ft) for ice cover and 0.3 m (1 ft) for sludge storage.

The "effective" depth in the secondary cells = 2.4 m (7.9 ft).

The volume of the cells can be calculated using the following formula for the volume of a rectangular basin with sloped sides and rounded corners.

$$V = \left[(L \times W) + (L-2sd)(W-2sd) + 4\,(L-sd)(W-sd)\right]\frac{d}{6}$$

where,

V = volume, m^3

L = length of pond at water surface, m

W = width of pond at water surface, m

s = horizontal slope factor, i.e., 3:1 slope, $s = 3$

d = depth of pond, m

The total volume of the primary cells includes the sludge and ice storage or a total depth of 2 m. The "effective" volume is based on a depth of 1.4 m and is the volume used to calculate the theoretical hydraulic detention time.

$$\text{Total Volume in One Primary Cell (TVPC)} = \Big[(378)(126) + (378-2 \times 4 \times 2) + (126-2 \times 4 \times 2) + 4\,(378-4 \times 2)(126-4 \times 2)\Big]\frac{2}{6}$$

$$\text{TVPC} = 87{,}363\ \text{m}^3\ (3.09 \times 10^6\ \text{ft}^3)$$

$$\text{Effective Volume} = \Big[(378)(126) + (378-2\text{x}4\text{x}1.4)(126-2\text{x}4\text{x}1.4) + 4(378-4 \times 1.4)(126-4 \times 1.4)\Big]\frac{1.4}{6}$$

$$\text{Effective Volume} = 62{,}786\ \text{m}^3\ (2.22 \times 10^6\ \text{ft}^3)$$

Theoretical hydraulic detention time in primary cell = t_1

$$t_1 = 62{,}786/(1893/2) = 66\ \text{days}$$

The total volume of the secondary cells includes the ice and sludge storage for a total depth of 3 m (10 ft). The "effective" volume of the secondary cells is based on a depth of 2.4 m (7.9 ft), and the theoretical hydraulic detention time is calculated based on the volume of the cell at a depth of 2.4 m.

$$\text{Total Volume in One Secondary Cell (TVSC)} = \Big[(253)(84) + (253-2 \times 4 \times 3)(84-2 \times 4 \times 3) + 4\,(253-4 \times 3)(84-4 \times 3)\Big]\frac{3}{6}$$

$$\text{TVSC} = 52{,}200\ \text{m}^3\ (1.84 \times 10^6\ \text{ft}^3)$$

$$\text{Effective Volume in One Secondary Cell} = \Big[(253)(84) + (253-2 \times 4 \times 2.4)(84-2 \times 4 \times 2.4) + 4\,(253-4 \times 2.4)(84-4 \times 2.4)\Big]\frac{2.4}{6}$$

$$\text{Effective Volume} = 43{,}535\ \text{m}^3\ (1.54 \times 10^6\ \text{ft}^3)$$

Theoretical hydraulic detention time in secondary cell = t_2

$t_2 = 43{,}535/(1893/2) = 46$ d

t = effective total theoretical hydraulic detention time

$t = t_1 + t_2 + t_3 + t_4$

$t = 66 + 46 + 46 + 46 = 204$ d

The hydraulic residence time calculated for the selected loading rates exceeds the minimum acceptable residence time of 180 days. The system can be designed for a hydraulic residence time of 180 days without discharge during the winter months and discharge during the summer. Another option is to operate the system as a controlled discharge pond system. Results such as these will occur frequently when the design is based on conservative loading rates used in areas with severe climates.

The size of the pond with a 180-day hydraulic detention time is calculated as follows.

Volume of one primary cell = same as above because of loading limit of 40 kg BOD_5/ha/d.

$V_1 = 62{,}786\ m^3\ (2.22 \times 10^6\ ft^3)$

$t_1 = 62{,}786\ m^3\ (1893\ m^3/d/2) = 66$ d

$t_2 = 180 - 66 = 114$ d

V_2 = volume in one secondary cell =

$114\ d\ (1893\ m^3/d)/(2)(3) = 35{,}967\ m^3\ (1.27 \times 10^6\ ft^3)$

There are numerous options as to how the ponds may be arranged besides the four cells in series selected above. The simplest, but not the best option, would be a two pond system without baffles. The hydraulic characteristics of the two pond system could be improved by installing baffles to direct the flow patterns. In severe climates the use of baffles must be conducted with care because of the potential for ice damage. The two parallel systems with each processing one-half of the flow is an excellent choice because of the flexibility provided by such a flow configuration. The four ponds in series will provide a hydraulic residence time that would approach the theoretical value.

Many state standards require that a minimum of 0.6 m (2 ft) of water depth be maintained in ponds; therefore, the volume required to store 180 days of wastewater flow is based on the volume above the 0.6-m water depth. The volume allowed for sludge and ice storage in the system will satisfy this requirement.

The dimensions of the two primary cells will be the same as calculated above, or 378 m x 126 m at the water surface at maximum depth. The dimensions of the six secondary cells can be calculated using the formula used above to calculate the volume of a rectangular basin with sloped sides and rounded corners. Side slopes are 4:1 and the length to width ratio is 3:1.

$$V_2 = 35{,}967 = \left[(L \times L/3) + (L - 2 \times 4 \times 2.4)(L/3 - 2 \times 4 \times 2.4) + 4\,(L - 4 \times 2.4)(L/3 - 4 \times 2.4)\right]\frac{2.4}{6}$$

$$L^2/3 + (L - 19.2)(L/3 - 19.2) + 4\,(L - 9.6)(L/3 - 9.6) = 89{,}918$$

$$L^2/3 + L^2/3 - 38.4L + 368.64 + 4(L^2/3 - 19.2L + 92.16) = 89{,}918$$

$$2\,L^2 - 115.2L + 737.28 = 89{,}918$$

Solve quadratic equation by completing the square.

$$L^2 - 57.6L + 829.44 = 44{,}591 + 829.44$$

$$(L - 28.8)^2 = 45{,}420$$

$$L - 28.8 = 213.1$$

$L = 241.9$ m, Use 242 m (794 ft)

$W = 241.9/3 = 80.6$ m, Use 81 m (266 ft)

Surface area of water surface in one secondary cell = 242 (81) = 19,602 m^2

Surface area in all secondary cells = 19,602 (6) = 117,612 m^2

3.2.2 Gloyna Equation

The Gloyna equation and design parameters are summarized in Table 3-1.

Design Conditions: See Table 3-2.

Requirements: Size a facultative wastewater treatment pond to treat the wastewater described in Table 3-2, and specify the following parameters for the system.

1) Detention time in total system and first cell, t and t_1
2) Volume in total system and first cell, V and V_1
3) Surface area in total system and first cell, A and A_1

4) Depth, d

5) Length, L

6) Width, W

Solution:

Gloyna suggests that the ultimate BOD be used in the equation, and this is logical because in treatment units using extended detention periods, the ultimate demand is important. The COD is the logical measure of oxygen demand if industrial wastes or sulfate on other sulfur compounds are present. Ultimate BOD values usually are not available, and it is necessary to use the COD or a multiplier to estimate the ultimate BOD. A multiplier of 1.2 will be used to estimate the ultimate BOD (8).

$$\frac{V}{Q} = t = 0.035L_a \quad \theta^{(35-T)} \quad ff'$$

$L_a = 1.2\ (200\ mg/l) = 240\ mg/l$

$\theta = 1.085$

$T = 0.5°C$

$f = 1.0$

$f' = 1.0$

$t = 0.035\ (240)\ \ 1.085^{(35-0.5)}$

$t = 140\ d$

$V = t\ (Q) = 140\ (1893\ m^3/d) = 265{,}000\ m^3\ (9.36 \times 10^6\ ft^3)$

Climate is severe; therefore, the depth should be 2 m (6.6 ft). The depth of 2 m includes approximately 0.3 m (1 ft) for ice cover and 0.3 m (1 ft) for sludge storage.

The effective depth of 1 m is to be used in the calculations of surface loadings and surface areas.

Surface area = A = $265{,}000\ m^3/1\ m$ = 26.5 ha (65.5 ac)

Surface area loading rate = 379 kg/d/26.5 ha

Surface area loading rate = 14.3 kg BOD_5/ha/d

The length and width of individual cells can be calculated as shown for the previous example.

More detail on the arrangement of ponds as perceived by Gloyna is available in several publications (8) (19) (20).

Gloyna and Tischler (19) have reported that facultative ponds are effective only in a water temperature range of 5 to 35°C; therefore, it is unlikely that they would recommend the use of the Gloyna equation at the design water temperature of 0.5°C used in this example. However, the results agree well with the areal loading design presented above. A comparison of these two methods as well as others presented in the following paragraph is presented at the end of the section on facultative ponds.

3.2.3 Marais-Shaw Equation

The Marais-Shaw equation is based on a complete mix model and first order kinetics. The equation and conditions necessary for its application are shown in Table 3-1.

Design Conditions: See Table 3-2.

Requirements: Size a facultative wastewater treatment pond to treat the wastewater described in Table 3-2, and specify the following parameters for the system.

1) Detention time in total system and first cell, t and t_1
2) Volume in total system and first cell, V and V_1
3) Surface area in total system and first cell, A and A_1
4) Depth, d
5) Length, L
6) Width, W

Solution:

Marais and Shaw (21) proposed that the maximum BOD_5 concentration in the primary cells, $(C_e)_{max}$, be 55 mg/l to avoid anaerobic conditions and odors.

The permissible depth of the pond, d in feet, was found to be related to $(C_e)_{max}$ as follows: (Equation must be used with English units because of the empirical constants that cannot be converted to metric units.)

$$(C_e)_{max} = \frac{700}{0.6\ d + 8}$$

$$55 = \frac{700}{0.6\ d + 8}$$

$$d = 7.9 \text{ ft } (2.4 \text{ m})$$

Use a d = 8 ft (2.4 m)

Detention time in the primary cell is calculated as follows:

$$\frac{C_n}{C_o} = \left[\frac{1}{1 + k_c\ t_n}\right]^n$$

$$t_n = \frac{(C_o/C_n)^{1/n} - 1}{k_c}$$

$$k_{cT} = k_{c35}\ (1.085)^{T-35}$$

$$k_{c35} = 1.2 \text{ day}^{-1}$$

$$k_{cT} = 1.2\ (1.085)^{0.5-35} = 1.2\ (0.0599)$$

$$k_{cT} = 0.072 \text{ day}^{-1}$$

$$t_1 = \frac{(200/55) - 1}{0.072}$$

$$t_1 = 36.6 \text{ d}$$

$$V_1 = 36.6\ (1{,}893 \text{ m}^3/\text{day}) = 69{,}300 \text{ m}^3$$

Calculate the surface area of the primary cell.

$$A_1 = \frac{69{,}300 \text{ m}^3}{2.4 \text{ m}}$$

$$A_1 = 2.9 \text{ ha } (7.2 \text{ ac})$$

Determine the number of ponds in series that will be required to produce an effluent containing 30 mg/l of BOD_5.

$$\frac{C_n}{C_o} = \left[\frac{1}{1 + k_c\ t_1}\right]^n$$

$$30/200 = \left[\frac{1}{1 + 0.072\ (36.6)}\right]^n$$

$$n = 1.5$$

Two ponds of equal volume in series with a depth of 2.4 m (8 ft) and a surface area of 2.8 ha (7.0 ac) will be required (Mara (22) has shown that the most efficient series operation consists of equal volumes).

Calculate the surface loading rate being applied to the primary cell.

$$\text{Surface loading rate} = \frac{(200\ \text{mg/l})(1893\ \text{m}^3/\text{d})\left[\dfrac{1000\ \text{liters}}{\text{m}^3}\right]\left[\dfrac{\text{kg}}{1 \times 10^6\ \text{mg}}\right]}{2.8\ \text{ha}}$$

Surface loading rate = 135 kg of BOD_5/ha/d

The surface loading rate applied to the primary cell is much higher than the value of 40 kg BOD_5/ha/d normally recommended as the maximum for a severe climate such as the environmental conditions specified for this design. Because the method was developed in a warm climate, it is likely that the method cannot be applied to northern areas. This design approach does not make any allowance for ice cover and/or sludge storage to determine an "effective" depth. If the values used in the previous examples were applied here, the total depth would approach 3 m.

The dimensions of ponds designed by this method are also calculated as shown in the first design example (Section 3.2.1).

3.2.4 Plug Flow Model

The plug flow equation and design parameters are summarized in Table 3-1.

Design Conditions: See Table 3-2.

Requirements: Size a facultative wastewater treatment pond to treat the wastewater described in Table 3-2, and specify the following parameters for the system.

1) Detention time in total system and first cell, t and t_1

2) Volume in total system and first cell, V and V_1

3) Surface area in total system and first cell, A and A_1

4) Depth, d

5) Length, L

6) Width, W

Solution:

The difficult part of using any of the design methods is selecting the reaction rate. The plug flow model is no exception. As shown in Table 3-1, the value of the reaction rate varies with the organic loading rate. The size of the pond system can be based on an average value for the total system, or the removal in each stage of the system and the organic loading rate to the succeeding cell can be estimated and the reaction rate varied for each cell. Theoretically, the latter approach should be used; however, in most cases an overall $k_{p_{20}}$ of 0.1 day^{-1} is used to size the system. Both approaches will be illustrated.

Variable kp

In severe climates the organic loading rate (kg BOD_5/ha/d) in the primary cell is limited to 40 kg/ha/d; therefore, a $k_{p_{20}}$ value of 0.071 day^{-1} will be used to estimate the removal of BOD_5 in the first cell. The size of the primary cell will be the same as the value calculated in the design using organic loading rate criteria.

A_1 = (379 kg/d)/(40 kg/ha/d) = 9.5 ha (23.4 ac)

A total depth of 2 m (6.6 ft) is selected based on the criteria described earlier. The depth includes 0.3 m (1 ft) for ice cover and 0.3 m (1 ft) for sludge storage. The "effective" depth in the primary cell is 1.4 m (4.6 ft).

Total Volume in One Primary Cell = 87,363 m^3 (3.09×10^6 ft^3)

Effective Volume in One Primary Cell = 62,786 m^3 (2.22×10^6 ft^3)

See Section 3.2.1 for the calculations of the above volumes.

t_1 = 62,786 m^3/(1,893 m^3/d/2) = 66 d

Calculate the effluent quality of the primary cell.

$$k_{p_T} = k_{p_{20}} (1.09)^{T-20}$$

$$k_{p_T} = 0.071 (1.09)^{0.5-20}$$

$$k_{p_T} = 0.013 \text{ day}^{-1}$$

$$\frac{C_1}{C_0} = e^{-k_{p_T} t}$$

$$\frac{C_1}{200} = e^{-0.013(66)}$$

$$C_1 = 200 (0.424)$$

$$C_1 = 85 \text{ mg/l}$$

The hydraulic detention time required to remove the remaining BOD_5 is calculated as follows. The organic loading rate on the cells following the primary cell is much lower than that applied to the primary cell. It is necessary to select a lower reaction rate of $k_{p_{20}} = 0.045 \text{ day}^{-1}$.

Calculate the t_2 necessary to produce an effluent with a BOD_5 concentration of 30 mg/l.

$$k_{p_T} = 0.045 (1.09)^{0.5-20}$$

$$k_{p_T} = 0.0084$$

$$\frac{30}{85} = e^{-0.0084 t_2}$$

$$-0.0084 t_2 = -1.041$$

$$t_2 = 124 \text{ d}$$

$$V_2 = Qt_2 = (1,893 \text{ m}^3/\text{d})(124\text{d})$$

$$V_2 = 234,700 \text{ m}^3 (8.29 \times 10^6 \text{ ft}^3)$$

Most state standards permit the use of a greater depth in cells following the primary cell. Depths as great as 2.5 m (8 ft) are allowed. A depth of 2.5 m will be used in the secondary cells with an "effective" depth of 1.9 m.

$$d_2 = 1.9 \text{ m}$$

$$A_2 = V_2/d_2 = 234{,}700 \text{ m}^3/1.9 \text{ m}$$

$$A_2 = 123{,}500 \text{ m}^2 \text{ (30.5 ac)}$$

The total detention time in the system is 190 days; therefore, the minimum acceptable value of 180 days will control. The size of the pond system would be calculated the same way as that shown in Section 3.2.1.

Constant k

An average value of $k_{P_{20}}$ of 0.1 day^{-1} will be used to size the entire system. The size of the primary cell is controlled by the limit of 40 kg of BOD_5/ha/d; therefore, the primary cell will be the same size as that used in the design using the organic areal loading rate criteria.

$$A_1 = \text{(379 kg/d)/(40 kg/ha/d)} = 9.5 \text{ ha (23.4 ac)}$$

A depth of 2 m (6.6 ft) is selected based on criteria described earlier. The depth includes 0.3 m (1 ft) for ice cover and 0.3 m (1 ft) for sludge storage. The "effective" depth in the primary cell is 1.4 m (4.6 ft).

$$\text{Effective Volume} = 125{,}600 \text{ m}^3$$

$$t_1 = 125{,}600 \text{ m}^3/1{,}893 \text{ m}^3/\text{d} = 66 \text{ d}$$

Calculate the effluent quality of the primary cell.

$$k_{P_T} = k_{P_{20}} \ (1.09)^{T-20}$$

$$k_{P_T} = 0.1 \ (1.09)^{0.5-20}$$

$$k_{P_T} = 0.019 \text{ day}^{-1}$$

$$\frac{C_1}{C_o} = e^{-k_{P_T} t}$$

$$\frac{C_1}{200} = e^{-0.019 \ (66)}$$

$$C_1 = 57 \text{ mg/l}$$

To satisfy the effluent standard of 30 mg/l of BOD_5, a secondary pond will be required. The detention time necessary to produce an effluent BOD_5 of 30 mg/l is calculated as follows:

$$\frac{C_1}{C_o} = e^{-k_{P_T} t}$$

$$\frac{30}{57} = e^{-0.019t}$$

$$t = 34 \text{ d}$$

The effluent from the secondary pond meets the standard of 30 mg/l of BOD_5; therefore, additional ponds would not be required. Using a two-pond system is not recommended, but if such a system were selected, baffling would be necessary to improve the hydraulic characteristics so that the actual hydraulic residence time approached the theoretical.

3.2.5 Wehner-Wilhelm Equation

The Wehner-Wilhelm (18) equation for arbitrary flow was proposed by Thirumurthi (14) as a method to design facultative pond systems. The equation and design parameters are summarized in Table 3-1. Thirumurthi developed the chart shown in Figure 3-1 to facilitate the use of the equation. The term kt is plotted versus the percent BOD_5 remaining in the effluent for dispersion factors varying from zero for ideal plug flow to infinity for a complete mix reactor. Dispersion factors for ponds range from 0.1 to 2 with most values not exceeding 1.0. Using the arbitrary flow equation is complicated in that two "constants" must be selected, the reaction rate (k) and the dispersion factor (D). The influence of the dispersion factor can be illustrated by using several values to estimate the detention time required to reduce the BOD_5 from 200 mg/l to 30 mg/l as specified in the other examples. A value of 0.15 day^{-1} was recommended for k_{20}.

$$k_T = k_{20} (1.09)^{T-20}$$

$$k_T = 0.15 (1.09)^{0.5-20}$$

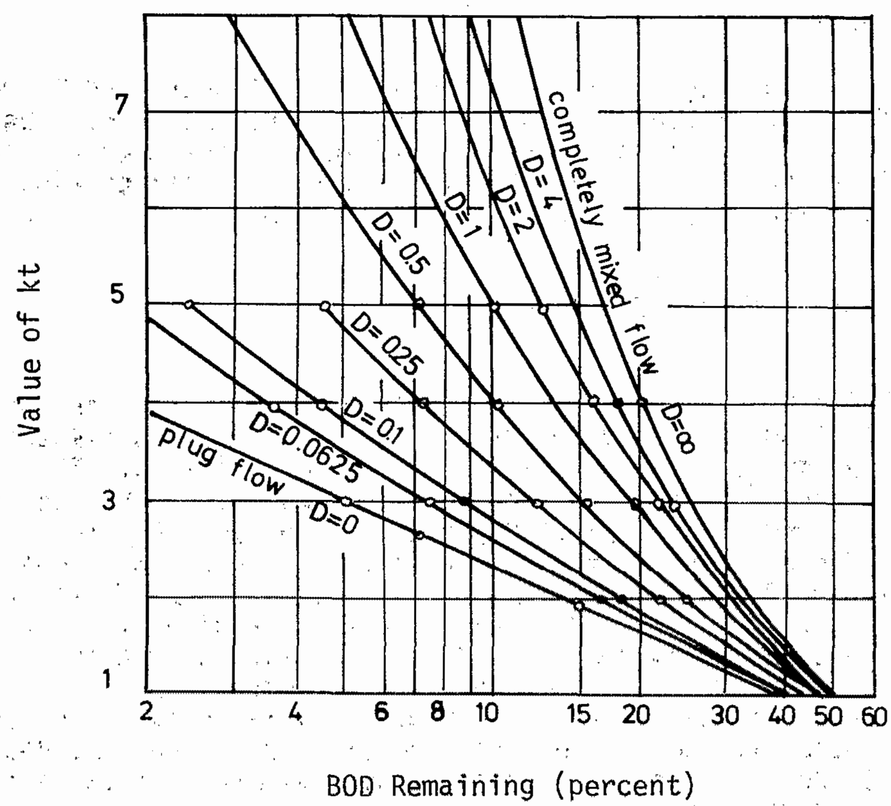
$$k_T = 0.028 \text{ day}^{-1}$$

$$\frac{C_e}{C_o} = \text{Percent BOD}_5 \text{ remaining} = (30/200)\ 100 = 15\%$$

Values of $k_T t$, t, V, and A for dispersion factors normally occurring in pond systems are summarized in Table 3.3. A depth of 2 m (6.6 ft) was selected based on the criteria outlined in the areal loading method of design. The "effective" depth is 1.4 m (4.6 ft).

FIGURE 3-1

WEHNER AND WILHELM EQUATION CHART (14)



A typical calculation for a D of 0.1 and a k_T of 0.028 day^{-1} is shown below. It is necessary to solve the equation by trial and error.

$$\frac{C_e}{C_o} = \frac{4\ ae^{1/2D}}{(1+a)^2\ e^{a/2D} - (1-a)^2\ e^{-a/2D}}$$

$$a = \sqrt{1 + 4\ k_T tD}$$

$$a = \sqrt{1 + 4\ (0.028)(0.1)\ t} = \sqrt{1 + 0.0112\ t}$$

First iteration:

Assume t = 50 d

$$a = \sqrt{1 + 0.0112(50)} = 1.25$$

$$\frac{C_e}{C_o} = \frac{30}{200} = \frac{4(1.25)\ e^{1/(2)(0.1)}}{(1+1.25)^2\ e^{1.25/(2)(0.1)} - (1-1.25)^2\ e^{-1.25/(2)(0.1)}}$$

$$0.15 \neq \frac{742.07}{(5.0625)(518.01) - (0.0625)(0.00193)} = 0.283$$

Agreement is not satisfactory; therefore, must perform iterations until two sides of equation agree.

nth iteration:

Assume t = 80 d

$$a = \sqrt{1 + 0.0112(80)} = 1.377$$

$$\frac{C_e}{C_o} = \frac{30}{200} = \frac{4(1.377)\ e^{1/(2)(0.1)}}{(1+1.377)^2\ e^{1.377/(2)(0.1)} - (1-1.377)^2\ e^{-1.377/(2)(0.1)}}$$

$$0.15 = \frac{817.46}{(5.65)(977.50) - (0.142)(0.00102)} = 0.148$$

Agreement is adequate and the design detention time is 80 days.

TABLE 3-3

VARIATIONS IN DESIGN PRODUCED BY VARYING THE DISPERSION FACTOR

D	0.1	0.25	0.5	1.0
$k_T t$	2.3	2.6	3.0	3.7
t,days	80	93	107	132
V, m^3	151,400	176,000	202,600	249,900
A, m^2	108,200	125,700	144,700	178,500

The selection of a value for D can dramatically affect the detention time required to produce a given quality effluent. The selection of a design value for k_{20} can have an equal effect.

The dimensions of ponds based on this design method are calculated as previously shown.

3.2.6 Discussion of Design Methods

All of the designs are based on parameters reported in the literature and no attempt was made to select parameters that would produce consistent results in all of the methods. Each design was made independently using the values of constants recommended by the author of a method.

A summary of the results from the various design methods is shown in Table 3-4. Numerous and varying requirements are imposed on the designs by the conditions under which the methods were developed. These limitations on the design methods make it difficult to make direct comparisons; however, an examination of the hydraulic detention times and total volume requirements calculated by all of the methods shows considerable consistency if the Marais and Shaw method is excluded and a value of 1.0 is selected for the dispersion factor in the Wehner-Wilhelm method.

All of the design equations have limitations and several have been mentioned in the design examples. To determine the limitations of a particular method, the original reference should be consulted. The major limitation of all the methods is the selection of a reaction rate constant or other factors in the equations. Even with this limitation, if the pond hydraulic system is designed and constructed such that the theoretical hydraulic detention time is approached, reasonable success can be assured with all of the design methods. Short circuiting is the

TABLE 3-4

SUMMARY OF RESULTS FROM DESIGN METHODS

DESIGN METHOD	DETENTION TIME, d		VOLUME, m^3		SURFACE AREA, ha		Depth m	No. Cells in Series	SURFACE LOADING RATE kg BOD_5/ha/d	
	Primary Pond	Total System	Primary Pond	Total System	Primary Pond	Total System			Primary Pond	Total System
AREAL LOADING RATE	66[a]	180	125,600[a]	386,800	9.5	22.3	2 (1.4)[c]	4	40	17
GLOYNA	–	140	125,600[a]	265,000	–	26.5	2 (1)[c]	–	–	14
MARAIS & SHAW	37[b]	74	69,300[b]	138,600	2.8	5.6	2.4	2	135	68
PLUG FLOW	66[a]	180	125,600[a]	386,800	9.5	22.3	2 (1.4)[c]	4	40	25
WEHNER & WILHELM	66[a]	80-132	125,600[a]	151,400 to 249,900	9.5	10.8 to 17.9	2 (1.4)[c]	4	–	30-48

[a]Controlled by state standards and is equal to value calculated for an areal loading rate of 40 kg/ha/d and an effective depth of 1.4 m.

[b]Also would be controlled by state standard for areal loading rate; however, the method includes a provision for calculating a value and this calculated value is shown.

[c]Effective depth.

greatest deterrent to successful pond performance, barring any toxic effects. The importance of the hydraulic design of a pond system cannot be overemphasized.

The surface loading rate approach to design requires a minimum of input data, and is based on operational experiences in various geographical areas of the country. This is probably the most conservative of the design methods, but attention to the hydraulic design is as important as the selection of the BOD_5 loading rate.

The Gloyna method is applicable only for 80 to 90 percent BOD removal efficiency, and it is assumed that solar energy for photosynthesis is above the saturation level. Provisions for removals outside this range are not made; however, an adjustment for light can be made by multiplying the pond volume by the ratio of sunlight in the particular area to the average found in the Southwest. Mara (23) has discussed the limitations of the Gloyna equation, and if a detailed critique is needed, the reference should be consulted.

The Marais and Shaw method of design is based on complete mix hydraulics and first order reaction kinetics. Complete mix hydraulics are not approached in facultative ponds, but the greatest weakness in the approach may lie in the calculation of the volume of a primary cell that will not turn anaerobic. Mara (22) (23) has also discussed the limitations of the Marais and Shaw approach and these references should be consulted.

Plug flow hydraulics and first order reaction kinetics have been found to describe the performance of many facultative pond systems (14) (16) (17). As shown in the Appendix, a plug flow model was found to best describe the performance of the four facultative pond systems summarized in Chapter 2. Because of the arrangement of most facultative pond systems into a series of three or more ponds, logically it would be expected that the hydraulic regime could be approximated by a plug flow model.

The plug flow design reaction rate used in the above example was based on values in the literature (14) (16). The reaction rate (slope of the line of best fit) calculated from Figure A-9 in the Appendix, adjusted for the temperature using the expression in Table 3-1 for the plug flow model, yields a hydraulic detention time of over 400 days. This large difference in detention times (190 versus 400 days) is probably attributable to the low hydraulic and organic loading rates applied to the four systems. These low loading rates tend to result in a lower value for the reaction rate (16). This discrepancy in reaction rates further illustrates the difficulty and importance of selecting the design parameters.

Use of the Wehner-Wilhelm equation requires knowledge of both the reaction rate and the dispersion factor which further complicates the design procedure. If knowledge of the hydraulic characteristics of a proposed pond configuration exists or can be determined, the

Wehner-Wilhelm equation will yield satisfactory results. However, because of the difficulty of selecting both parameters, design with one of the simpler equations is likely to be as good as one using the Wehner-Wilhelm equation.

In summary, all of the design methods discussed can provide a valid design, if the proper design parameters are selected and the hydraulic characteristics of the system are controlled.

3.3 Complete Mix Aerated Ponds

Complete mix aerated ponds are designed and operated as flow-through ponds with or without solids recycle. Most systems are operated without solids recycle; however, many systems are built with the option to recycle effluent and solids. Even though the recycle option may not be exercised, it is desirable to include it in the design to provide flexibility in the operation of the system. If the solids are returned to the pond, the process becomes a modified activated sludge process.

Solids in the complete mix aerated pond are kept suspended at all times. The effluent from the aeration tank will contain from one-third to one-half the concentration of the influent BOD in the form of solids (3). These solids must be removed by settling before discharging the effluent. Settling is an integral part of the aerated pond system. Either a settling basin or a quiescent portion of one of the cells separated by baffles may be used for solids removal.

Six factors are considered in the design of an aerated pond: 1) BOD removal, 2) effluent characteristics, 3) oxygen requirements, 4) mixing requirements, 5) temperature effects, and 6) solids separation (3). BOD removal and the effluent characteristics are generally estimated using a complete mix hydraulic model and first order reaction kinetics. A combination of Monod-type kinetics, first order kinetics, and a complete mix model has been proposed, but there is limited experience with the method (3) (24). The complete mix hydraulic model and first order reaction kinetics will be used in the following example. Oxygen requirements will be estimated using equations based upon mass balances; however, in a complete mix system the power input necessary to keep the solids suspended is much greater than that required to transfer adequate oxygen. Temperature effects are incorporated into the BOD removal equations. Solids removal will be accomplished by installing a settling pond. If a higher quality effluent is required, the solids removal devices described in Chapter 5 should be evaluated and one selected to produce an acceptable effluent quality.

3.3.1 Complete Mix Model

The complete mix model using first order kinetics and operating in a series with n equal volume ponds is shown below.

$$\frac{C_n}{C_o} = \frac{1}{\left[1 + \frac{k_c t}{n}\right]^n}$$

where,

C_n = effluent BOD_5 concentration in cell n, mg/l

C_o = influent BOD_5 concentration, mg/l

k_c = complete mix first order reaction rate constant, day^{-1} (assumed to be constant in all n cells) = 2.5 $days^{-1}$ at 20°C

t = total hydraulic residence time in pond system, days

n = number of ponds in series.

If other than a series of equal volume ponds are to be employed, it is necessary to use the following general equation.

$$\frac{C_n}{C_o} = \left(\frac{1}{1 + k_{c_1} t_1}\right)\left(\frac{1}{1 + k_{c_2} t_2}\right) \cdots \left(\frac{1}{1 + k_{c_n} t_n}\right)$$

where,

$k_{c_1}, k_{c_2}, k_{c_n}$ = complete mix first order reaction rate constant in each of n ponds. Because of the lack of better information, all are generally assumed to be equal.

t_1, t_2, t_n = hydraulic residence time in each pond, days

3.3.2 Selection of k_c

The selection of a k_c value is the critical decision in the design of any pond system. A design value should be determined for the individual wastewater in bench or pilot scale tests. If this is impractical, the experiences of others should be evaluated. As an initial estimate, the value of 2.5 $days^{-1}$ may be used for a complete mix aerated pond system.

3.3.3 Influence of Number of Ponds

When using the complete mix model, the number of ponds in series has a pronounced effect on the size of the aerated ponds required to achieve a specific degree of treatment. The decrease in total volume of reactor required to achieve a given efficiency by increasing the number of ponds in series can best be illustrated by an example. Rearranging the complete mix model into the following form makes it more convenient to calculate the total detention time.

$$t = \frac{n}{k_c}\left[\left(\frac{C_o}{C_n}\right)^{1/n} - 1\right]$$

where,

t = total hydraulic residence time in pond systems, days

n = number of ponds in series

k_c = complete mix first order reaction rate constant, 2.5 days^{-1} at 20°C

C_o = influent BOD_5 concentration, mg/l

C_n = effluent BOD_5 concentration in cell n, mg/l

When n = 1,

$$t = \frac{1}{2.5}\left[\left(\frac{200}{30}\right)^{1/1} - 1\right] = 2.27 \text{ d}$$

When n = 2,

$$t = \frac{2}{2.5}\left[\left(\frac{200}{30}\right)^{1/2} - 1\right] = 1.27 \text{ d}$$

When n = 3,

$$t = \frac{3}{2.5}\left[\left(\frac{200}{30}\right)^{1/3} - 1\right] = 1.06 \text{ d}$$

When n = 4,

$$t = \frac{4}{2.5} \left[\left(\frac{200}{30}\right)^{1/4} - 1 \right] = 0.97 \text{ d}$$

When n = 5,

$$t = \frac{5}{2.5} \left[\left(\frac{200}{30}\right)^{1/5} - 1 \right] = 0.92 \text{ d}$$

Continuing to increase n will result in the detention time being equal to the detention time in a plug flow reactor with a k_c value of 2.5 days^{-1}.

Figure 3-2 is a plot of the complete mix equation for one to four ponds in series. The figure can be used to estimate the performance or required detention time when the reaction rate (k_c) has been selected. When the influent (C_o) and effluent (C_n) BOD_5 are known, the value of C_o/C_n is calculated and located on the horizontal axis of the plot. A vertical line is extended from this point to intersect with the line for the number of ponds in series. From this point a horizontal line is drawn to intersect with the k_ct axis. This value of k_ct is then divided by the k_c value to yield the total detention time required in n ponds in series. An example is shown on Figure 3-2.

3.3.4 Unequal Volume and k_c

Mara (22) has shown that a number of equal volume reactors in series is more efficient than unequal volumes; however, there may be cases where it is necessary to construct ponds of unequal volume. When this is necessary, the following example for a three pond series with half the volume in the first pond and one-fourth in the second and third ponds will illustrate how to calculate the needed detention time or efficiency. The reaction rate constant is assumed to be 2.5 days^{-1} in the first pond and 1.5 days^{-1} in the second and third ponds to illustrate the procedure for varying reaction rates in the event they are available.

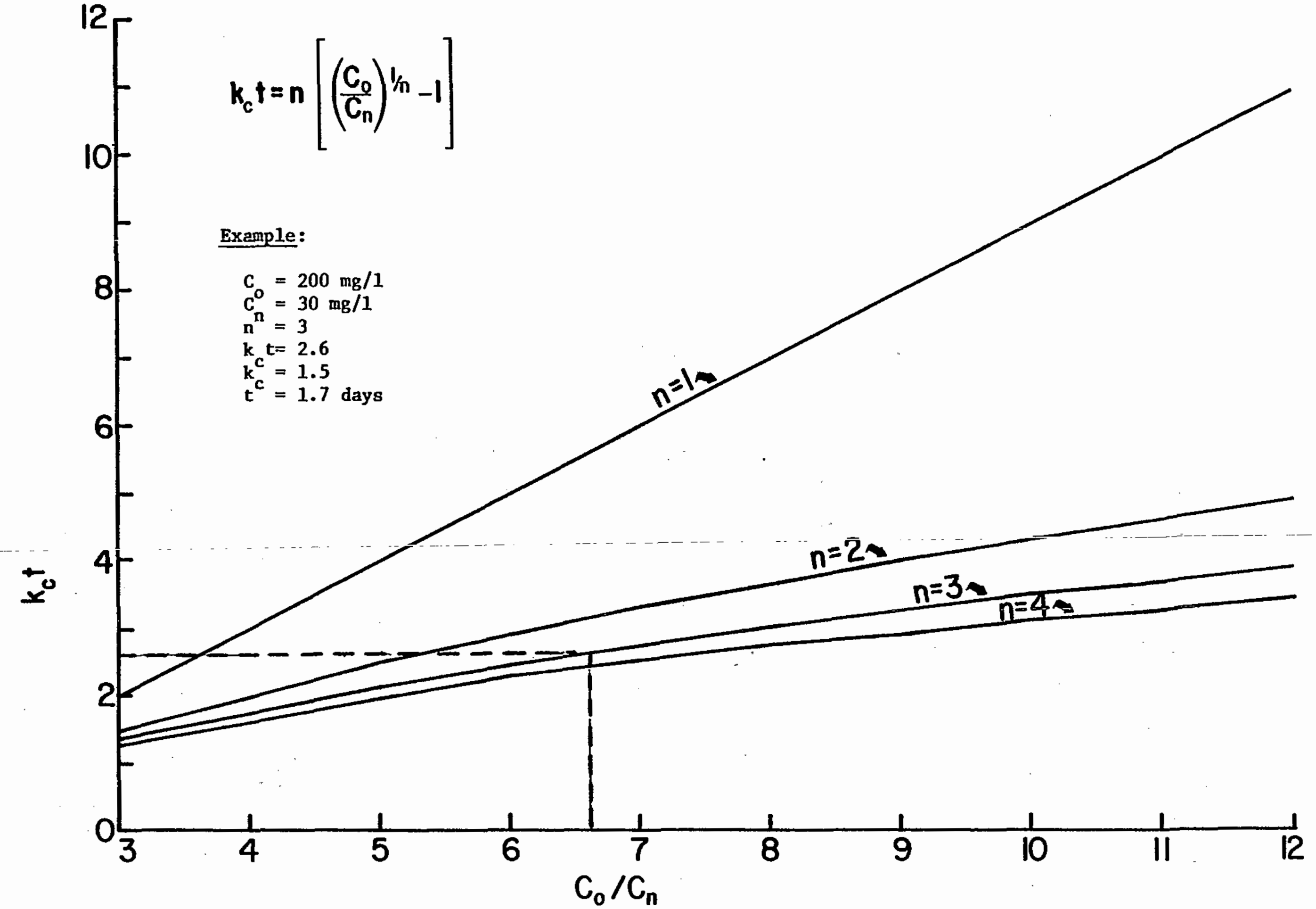
$$\frac{C_3}{C_o} = \left[\frac{1}{1 + k_{c_1} t_1}\right] \left[\frac{1}{1 + k_{c_2} t_2}\right] \left[\frac{1}{1 + k_{c_3} t_3}\right]$$

where,

C_3 = 30 mg/l

FIGURE 3-2

$k_c t$ VERSUS C_o/C_n FOR COMPLETE MIX MODEL



C_o = 200 mg/l

t_1 = 1/2 t

t_2 = t_3 = 1/4 t

t = total hydraulic residence time in system, days

$$\frac{30}{200} = \left[\frac{1}{1 + 2.5(t/2)}\right]\left[\frac{1}{1 + 1.5(t/4)}\right]\left[\frac{1}{1 + 1.5(t/4)}\right]$$

$$0.15 = \frac{1}{(1 + 1.25\ t)(1 + 0.375\ t)(1 + 0.375\ t)}$$

$$0.15 = \frac{1}{1 + 2\ t + 1.079\ t^2 + 0.176\ t^3}$$

$$0.0264\ t^3 + 0.162\ t^2 + 0.3\ t - 0.85 = 0$$

$$t^3 + 6.14\ t^2 + 11.36\ t - 32.2 = 0$$

The cubic equation can be solved by synthetic substitution as shown below or solved on a pocket calculator.

$$\begin{array}{l|l} 2.0 & 1 + 6.14 + 11.36 - 32.2 \\ & \quad + 2.00 + 16.28 + 55.3 \\ \hline & 1 + 8.14 + 27.64 + 23.1 \end{array}$$

If the sum of the last two terms are equal to zero, the assumed value (2.0 in this example) is a root of the equation. In the above iteration the assumed value is too large; therefore, another value must be assumed.

$$\begin{array}{l|l} 1.5 & 1 + 6.14 + 11.36 - 32.2 \\ & \quad + 1.50 + 11.46 + 34.2 \\ \hline & 1 + 7.64 + 22.82 + \ \ 2.0 \end{array}$$

The second estimated value is very close to a root and is probably accurate enough, but to complete the solution another trial will be completed.

$$\begin{array}{l|l} 1.45 & 1 + 6.14 + 11.36 - 32.2 \\ & \quad + 1.45 + 11.00 + 32.4 \\ \hline & 1 + 7.59 + 22.36 + \ \ 0.2 \end{array}$$

For the conditions described above, a total hydraulic detention time of 1.45 days (t) would be required. The first pond would have a detention time of 0.72 days (t/2) and the second and third ponds would have a detention time of 0.36 days (t/4).

3.3.5 Temperature Effects

The influence of temperature on the reaction rate is expressed as follows:

$$\frac{k_{c_T}}{k_{c_{20}}} = \theta^{T_w - 20}$$

where,

k_{c_T} = reaction rate at design temperature, $days^{-1}$

$k_{c_{20}}$ = reaction rate at 20°C, $days^{-1}$

θ = temperature factor, dimensionless = 1.085

T_w = temperature of pond water, °C

The impact of mixing and the ambient air temperature on the pond water temperature can be estimated by trial and error using the following equation developed by Mancini and Barnhart (25) and the complete mix model presented above.

$$T_w = \frac{AfT_a + QT_i}{Af + Q}$$

where,

T_w = pond water temperature, °C

T_a = ambient air temperature, °C

T_i = influent wastewater temperature, °C

A = surface area of pond, m^2

f = proportionality factor = 0.5

Q = wastewater flow rate, m^3/d

An estimate of the surface area is made using the complete mix model corrected for temperature, and then the water temperature is calculated using the Mancini and Barnhart equation.

After several iterations, when the water temperature used to correct the reaction rate coefficient agrees with the value calculated with the Mancini and Barnhart equation, the selection of the detention time in the aeration ponds is complete.

3.3.6 Mixing and Aeration

Aeration is used to mix the pond contents and to transfer oxygen to the liquid. In complete mix aerated ponds the mixing requirements control the power input to the system. There is no rational method available to predict the power input necessary to keep the solids suspended. The best approach is to consult equipment manufacturers' charts and tables to determine the power input needed to satisfy mixing requirements. Malina et al. (26) indicate that a minimum power level of 5.9 kW/1000 m^3 (30 hp/Mgal) of aeration tank volume is required to completely mix an aerated pond cell. Others indicate that approximately 2.96 kW/1000 m^3 (15 hp/Mgal) is adequate to maintain solids in suspension (27). These values can be used as a guide to make preliminary estimates of power requirements, but the final sizing of aeration equipment should be based on guaranteed performance by an equipment manufacturer.

There are several rational equations available to estimate the oxygen requirements for pond systems, and these equations can be found in many text books (3) (4) (20) (24). In most cases, the use of the BOD_5 entering the pond as a basis to estimate the biological oxygen requirements, is as effective as other approaches and has the advantage of being simple to calculate.

After determining the total horsepower requirement for a pond, the individual aeration units should be located in the pond so that there is an overlap of the diameter of influence providing complete mixing. Several small aerators are better than one or two large units. Large units create localized mixing; therefore, several small units would likely be more efficient and economical. Maintenance and repair of small units would have less of an impact on performance, and such an arrangement would provide more operational flexibility.

3.3.7 Design Example

The complete mix model with four equal volume ponds in series will be used. As discussed above, equal volume ponds in series are more efficient than unequal volumes, and increases in the number of ponds beyond four in a series does little to reduce the required hydraulic detention times. In addition to the benefits of reducing the required detention time, four ponds in series improves the hydraulic characteristics of the pond system. The following environmental and wastewater characteristics are given:

Q = 1893 m^3/d (0.5 mgd)

C_o = influent BOD_5 = 200 mg/l

C_n = effluent BOD_5 from the nth pond in an n-pond series lagoon system = 30 mg/l

k_{c_T} = $k_{c_{20}}(1.085)$

k_{c_T} = reaction rate at design temperature, $days^{-1}$

$k_{c_{20}}$ = reaction rate at 20°C = 2.5 $days^{-1}$

T_w = pond water temperature, °C

T_{a_1} = ambient air temperature in winter = 5°C

T_{a_2} = ambient air temperature in summer = 30°C

T_i = influent wastewater temperature = 15°C

f = proportionality factor = 0.5

Elevation = 100 m

Maintain a minimum dissolved oxygen concentration in ponds = 2.0 mg/l

Requirements: Size a complete mix aerated wastewater pond system to treat the wastewater and determine the following parameters for the system.

1) Total detention time, t

2) Volume, total and for each cell, V, V_1, etc.

3) Surface area, total and for each cell, A, A_1, etc.

4) Depth, d

5) Length of each cell, L

6) Width of each cell, W

7) Aeration requirements

Solution:

First iteration

$$t = \frac{n}{k_{c_T}}\left[\left(\frac{C_o}{C_n}\right)^{1/n} -1\right]$$

$$t = \frac{4}{k_{c_T}}\left[\left(\frac{200}{30}\right)^{1/4} -1\right]$$

Assume that t_w = 5°C during the winter.

$$k_{c_T} = 2.5\ (1.085)^{5-20}$$

$$k_{c_T} = 0.74\ \text{day}^{-1}$$

$$t = \frac{4}{0.74}\left[\left(\frac{200}{30}\right)^{1/4} -1\right] = 3.3\ \text{d}$$

$$t_1,\ t_2,\ t_3,\ \&\ t_4 = 3.3/4 = 0.825\ \text{d}$$

$$V_1 = 0.825\ (1893\ \text{m}^3/\text{day}) = 1562\ \text{m}^3\ (5.5 \times 10^4\ \text{ft}^3)$$

The pond depth is limited by the ability of the aeration equipment to maintain the pond contents at the desired level of mixing. Acceptable depths for aerated ponds range from 1.5 to 4.5 m (5 to 15 ft). A 3-m (10-ft) depth is used in this example.

$$A_1 = 1562\ \text{m}^3/3\ \text{m} = 520\ \text{m}^2\ (5597\ \text{ft}^2)$$

Check the pond water temperature using the surface area of 520 m^2 (5579 ft^2) and the other known parameters in the Mancini and Barnhart equation.

$$T_w = \frac{AfT_a + QT_i}{Af + Q}$$

$$T_w = \frac{520(0.5)(-5) + 1893(15)}{520(0.5) + 1893} = \frac{-1300 + 28395}{260 + 1893}$$

$$T_w = 12.6°\text{C}\ (55°\text{F})$$

Second iteration

As the winter pond water temperature increases, the surface area, detention time, and volume of the pond will decrease; therefore, the next estimate of T_w should be approximately equal to the calculated value of T_w of 12.6°C (55°F).

Assume T_w = 13°C in the winter.

$$k_{c_T} = 2.5\ (1.085)^{13-20}$$

$$k_{c_T} = 1.41\ \text{day}^{-1}$$

$$t = \frac{4}{1.41}\left[\left(\frac{200}{30}\right)^{1/4} - 1\right] = 1.72\ \text{d}$$

$$t_1 = 1.72/4 = 0.43\ \text{d}$$

$$V_1 = 0.43\ (1893) = 814\ \text{m}^3\ (2.87 \times 10^4\ \text{ft}^3)$$

$$A_1 = 814/3 = 271\ \text{m}^2\ (2900\ \text{ft}^2)$$

$$T_w = \frac{271(0.5)(-5) + 1893(15)}{271(0.5) + 1893} = \frac{-677.5 + 28395}{2028.5}$$

$$T_w = 13.7°\text{C}\ (57°\text{F})$$

The assumed value of 13°C is approximately equal to the results of the second iteration of 13.7°C. The limits of the method do not justify attempting to balance the temperatures closer than to the nearest degree.

The temperature will decrease from pond 1 to pond 2 and from pond 2 to pond 3, etc., but the decrease is not so great that an average T_w value cannot be used.

Final Size of Ponds

$$t = 1.72\ \text{d}$$

$$t_1, t_2, t_3\ \&\ t_4 = 0.43\ \text{d}$$

$$V = 3256\ \text{m}^3\ (11.5 \times 10^4\ \text{ft}^3)$$

$$V_1, V_2, V_3\ \&\ V_4 = 814\ \text{m}^3\ (2.87 \times 10^4\ \text{ft}^3)$$

$$A = 1084\ \text{m}^2\ (11.67 \times 10^3\ \text{ft}^2)$$

A_1, A_2, A_3 & A_4 = 271 m^2 (2.92 x 10^3 ft^2)

Dimensions of Ponds

Using the formula for the volume of a pond with sloped side walls for a square pond and the required volume in each pond, the dimensions of the ponds can be calculated. Use a slope of 2:1.

$$V = \left[(L \times W) + (L-2\,sd)(W-2\,sd) + 4\,(L-sd)(W-sd) \right] \frac{d}{6}$$

where,

V = volume = 814 m^3 (2.87 x 10^4 ft^3)

L = length of pond at water surface, m

W = width of pond at water surface, m

s = horizontal slope factor, i.e., 2:1 slope, s = 2

d = depth of pond = 3 m (10 ft)

$$814 = \left[(LW) + (L-2x2x3)(W-2x2x3) + 4\,(L-2x3)(W-2x3) \right] \frac{3}{6}$$

In a square pond L = W.

$$814\left(\frac{6}{3}\right) = L^2 + (L-12)(L-12) + 4\,(L-6)(L-6)$$

$$6L^2 - 72L + 288 = 1628$$

$$L^2 - 12L + 48 = 271.3$$

Solve quadratic equation by completing the square.

$$L^2 - 12L + 36 = 223.3 + 36$$

$$(L-6)^2 = 259.3$$

$$(L-6) = 16.1$$

$$L = 22.1 \text{ m at the water surface (72.5 ft)}$$

A minimum of 0.6 m (2 ft) of freeboard must be provided; therefore, the dimensions of a single pond will be 24.5 m x 24.5 m (80.4 ft x 80.4 ft) at the top of the inside of the dike with a water depth of 3 m (10 ft).

There are no rational design equations to predict the required mixing to keep the solids suspended in an aerated pond. Using the mass of BOD_5 entering the system as a basis to estimate the biological oxygen requirements is simple and as effective as other approaches.

Catalogs from equipment manufacturers must be consulted to ensure that adequate mixing is provided. Additionally, all types of equipment must be evaluated to ensure that the most economical and efficient system is selected. A municipal wastewater treatment system designed to provide complete mixing of the pond contents requires approximately 10 times as much power as a system designed to meet the oxygen requirements only. Therefore, an economic analysis along with sound engineering judgment is required to select the proper aeration equipment.

The following relationship is used to estimate aeration requirements:

$$N = \frac{N_a}{\alpha \left[\frac{C_{sw} - C_L}{C_s}\right] (1.025)^{T_w - 20}}$$

where,

N = equivalent oxygen transfer to tapwater at standard conditions, kg/hr

N_a = oxygen required to treat the wastewater, kg/hr

$\alpha = \dfrac{\text{oxygen transfer in wastewater}}{\text{oxygen transfer in tapwater}} \approx 0.9$

C_L = minimum DO concentration maintained in the waste, assume 2.0 mg/l

C_s = oxygen saturation value of tapwater at 20°C and one atmosphere pressure = 9.17 mg/l

T_w = wastewater temperature, °C

C_{sw} = $\beta(C_{ss})P$ = oxygen saturation value of the waste, mg/l

$\beta = \dfrac{\text{wastewater oxygen saturation value}}{\text{tapwater oxygen saturation value}} = 0.9$

C_{ss} = tapwater oxygen saturation value at temperature, T_w

P = ratio of barometric pressure at plant site to barometric pressure at sea level, assume 1.0 for the elevation of 100 m

The maximum oxygen transfer will be required in the summer months. The Mancini and Barnhart equation can be used to estimate the pond wastewater temperature during the summer.

$$T_w = \frac{271(0.5)(30) + 1893(15)}{271(0.5) + 1893}$$

$$T_w = \frac{4065 + 28395}{2028.5} = 16°C\ (61°F)$$

$$C_{ss} = 9.85\ mg/l\ at\ 16°C$$

BOD_5 in the wastewater $= C_o \times Q$

$$= (200\ g/m^3)\ (1893\ m^3/d)(kg/1000\ g)(day/24\ hr)$$

$$= 16\ kg/hr$$

Assume that the oxygen demand of the solids at peak flows will be 1.5 times the mean oxygen demand of 16 kg O_2/hr. Therefore,

$$N_a = 1.5 \times 16\ kg\ O_2/hr = 24\ kg\ O_2/hr$$

$$C_{sw} = 0.9(9.85\ mg/l)\ 1.0 = 8.87\ mg/l$$

$$N = \frac{24\ kg\ O_2/hr}{0.9\left[\frac{8.87 - 2.0}{9.17}\right](1.025)^{16-20}} = 39.3\ kg\ O_2/hr$$

Manufacturers' catalogs suggest 1.9 kg O_2/kWh (1.4 kg/hr/hp) for estimating power requirements. Therefore, the total power required to satisfy the oxygen demand in the pond system is:

$$\frac{39.3\ kg\ O_2/hr}{1.9\ kg\ O_2/kWh} = 20.7\ kW\ (27.8\ hp)$$

The power required to meet requirements for liquid mixing is (27):

minimum power = 1.5 kW/1000 m^3 of volume

power required = 1.5 kW/1000 m^3 (814 m^3) = 1.2 kW/cell

power total = (4 cells) (1.2 kW/cell) = 4.8 kW (6.4 hp)

The power required to meet requirements for solids suspension is (24):

minimum power = 15 kW/1000 m^3 of volume

power required = 15 kW/1000 m^3 (814 m^3/cell) = 12.2 kW/cell (16.4 hp/cell)

power required = (4 cells)(12.2 kW/cell) = 48.8 kW(65 hp)

The power required to maintain solids suspension exceeds the power required both to meet oxygen demand and for mixing of the pond liquid; therefore, the power requirement for solids suspension will be used to select aerators.

Assuming 90 percent efficiency for aerator gearing, the total motor power for solids suspension is:

$$\frac{48.8 \text{ kW}}{0.90} = 54.2 \text{ kW } (72.7 \text{ hp})$$

This value represents an approximate power requirement and is used to select aeration equipment. The power actually applied to the pond contents may be more or less than this value being determined by the zone of complete mixing requirements in each cell. Using the data presented in a catalog by Aqua-Aerobic Systems, Inc. (27), six 2.2-kW (3-hp) aerators for each cell would provide zones of complete mix having 13-m (42-ft) diameters and would not require a draft tube or anti-erosion assembly in a pond with a depth of 3 m (10 ft). This selection in aerators and aerator placement leaves small areas where solids suspension might occur. However, additional power and, hence, additional operating cost would not significantly improve efficiency. Figure 3-3 depicts aerator placement, zones of complete mixing for solids suspension in one of the four ponds. A settling pond with a hydraulic detention time of two days is provided after the fourth pond.

Summary

$V = 3256 \text{ m}^3$

$A = 1084 \text{ m}^3$

$t = 1.72 \text{ d}$

O_2 required for treatment = 39.3 kg O_2/hr (20.7 kW)

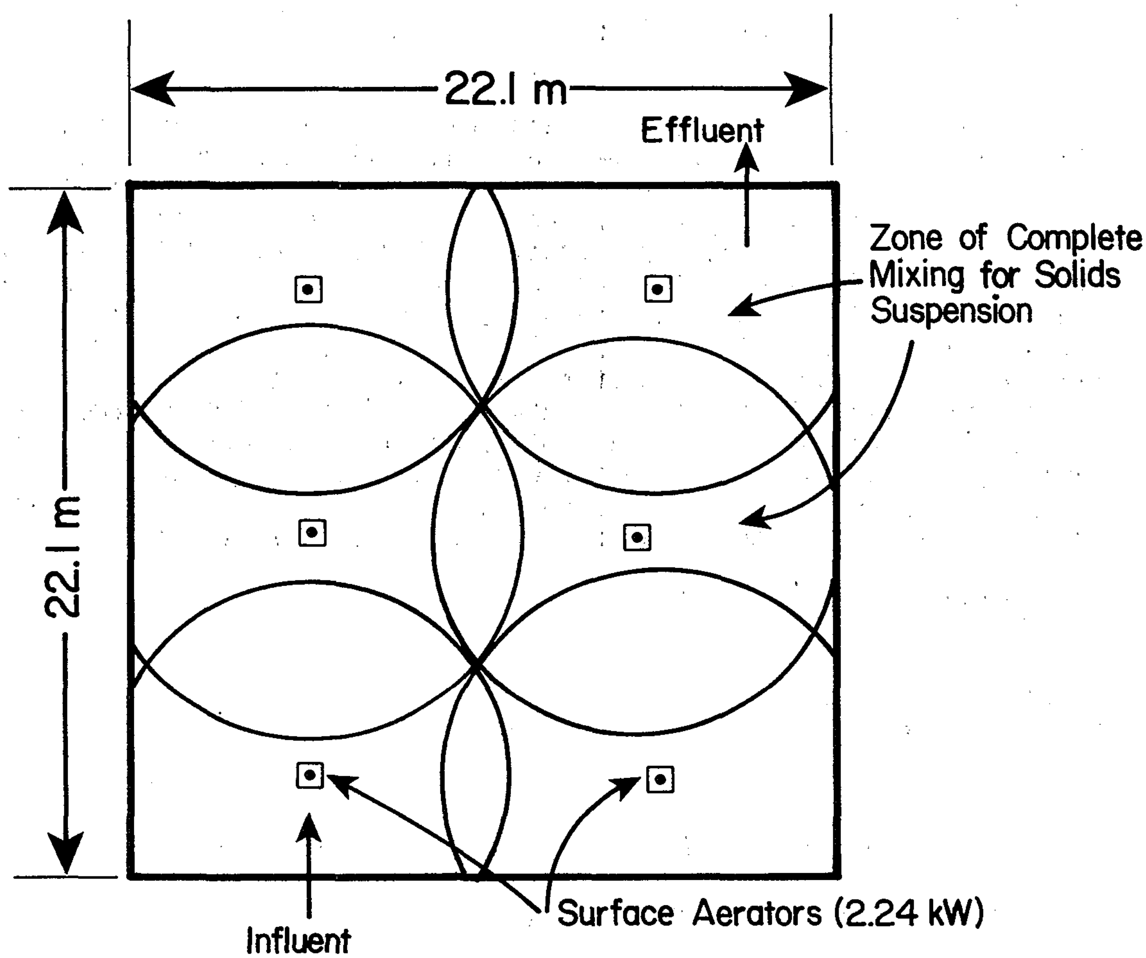
Power required for liquid mixing = 4.8 kW

Power required for solids suspension = 48.8 kW

Power supplied via six aerators per cell = 13.4 kW

FIGURE 3-3

LAYOUT OF ONE CELL OF COMPLETE MIX
AERATED POND SYSTEM



3.4 Partial Mix Aerated Ponds

In the partial mix aerated pond system, no attempt is made to keep all of the solids in the aerated ponds suspended. Aeration serves only to provide oxygen transfer adequate to oxidize the BOD entering the pond. Some mixing obviously occurs and keeps portions of the solids suspended; however, in the partial mix aerated pond, anaerobic degradation of the organic matter that settles does occur. The system is frequently referred to as a facultative aerated pond system.

Other than the difference in mixing requirements, the same factors considered in the complete mix aerated pond system are applicable to the design of a partial mix system, i.e., BOD removal, effluent characteristics, oxygen requirements, temperature effects, and solids separation. BOD removal is normally estimated using the complete mix hydraulic model and first order reaction kinetics. Recent studies (17) have shown that the plug flow model and first order kinetics more closely predict the performance of partial mix ponds using both surface and diffused air aeration. However, most of the ponds evaluated were lightly loaded and the reaction rates calculated from the performance data are very conservative because of the tendency of the reaction rates to decrease as the organic loading rate (surface loading or volumetric loading) decreases (16). Because of this lack of better design reaction rates, it is still necessary to design partial mix aerated ponds using the complete mix model discussed in detail in Section 3.3. The only difference in applying this model to partial mix systems is the selection of a reaction rate coefficient applicable to partial mix systems. Section 3.3 should be consulted to determine the effect of the number of ponds in series (3.3.3), unequal volumes in each pond (3.3.4), and temperature effects (3.3.5).

3.4.1 Selection of k_{pm}

As mentioned several times, the selection of the reaction rate coefficient is the most important decision in the design of any pond system. All other considerations in the design will be influenced by this selection. If possible, a design k_{pm} should be determined for the wastewater in pilot or bench scale tests. Experiences of others with similar wastewaters and environmental conditions should be evaluated.

The "Ten States Standards" (1) recommends k_{pm} values of 0.276 day^{-1} at 20°C and 0.138 day^{-1} at 1°C. Using the two values to calculate the temperature coefficient (θ), yields a θ value of 1.036. Boulier and Atchison (28) recommend values of k_{pm} of 0.2 to 0.3 at 20°C and 0.1 to 0.15 at 0.5°C. A temperature coefficient of 1.036 results when the two lower or higher values of k_{pm} are used to calculate θ. Reid (29) suggested a k_{pm} value of 0.28 at 20°C and 0.14 at 0.5°C based on research with partial mix ponds in central Alaska aerated with perforated tubing. These values are essentially identical to the "Ten

States Standards" recommendations. In the example presented later, the values recommended by the "Ten States Standards" will be used.

3.4.2 Mixing and Aeration

In partial mix aerated ponds, aerators are used to transfer oxygen to the liquid at the rate necessary to maintain aerobic conditions in the ponds. In a complete mix system the power input to each pond in the series must be adequate to keep the solids suspended; whereas, in a partial mix system the power input is reduced from pond to pond because of the reduction in organic matter (BOD) to be oxidized as the wastewater flows through the system.

The oxygen requirements in each pond can be calculated using rational formulas developed for activated sludge systems (3) (24); however, it is doubtful that any of the relationships are more accurate than assuming that all of the influent BOD_5 entering the pond is to be oxidized. After calculating the required rate of oxygen transfer, equipment manufacturers' catalogs should be consulted to determine the zone of complete oxygen dispersion by surface, helical, or air gun aerators or the proper spacing of perforated tubing.

3.4.3 Design Example

The design of a partial mix aerated pond is performed the same as that shown for the complete mix system with the exceptions of different reaction rate coefficients and less power input requirements. The power input is reduced from one pond to the next to account for the reduction in organic matter (BOD) to be oxidized as the wastewater flows through the system.

The complete mix model with four equal volume ponds in series will be used (see Section 3.3.7 for justification). The following environmental and wastewater characteristics are given:

Q = 1893 m^3/d (0.5 mgd)

C_o = influent BOD_5 = 200 mg/l

C_n = effluent BOD_5 from the nth pond in an n-pond series pond system $\leqq$ 30 mg/l

$$k_{pm} = k_{pm_{20}} (1.036)^{T_w - 20}$$

k_{pm} = reaction rate at design temperature, $days^{-1}$

$k_{pm_{20}}$ = reaction rate at 20°C = 0.276 day^{-1}

T_w = temperature of pond water, °C

T_{a_1} = ambient air temperature in winter = -5°C

T_{a_2} = ambient air temperature in summer = 30°C

T_i = influent wastewater temperature = 15°C

f = proportionality factor = 0.5

Elevation = 100 m

Maintain a minimum dissolved oxygen concentration in ponds = 2.0 mg/l

Requirements: Size a partial mix aerated pond system to treat the wastewater and determine the following parameters for the system.

1) Total detention time, t

2) Volume, total and for each cell, V, V_1, etc.

3) Surface area, total and for each cell, A, A_1, etc.

4) Depth, d

5) Length of each cell, L

6) Width of each cell, W

7) Aeration requirements

Solution:

First iteration

$$t = \frac{n}{k_{pm}} \left[\left(\frac{C_o}{C_n} \right)^{1/n} -1 \right]$$

$$t = \frac{4}{k_{pm}} \left[\left(\frac{200}{30} \right)^{1/4} -1 \right]$$

Assume that t_w = 10°C during the winter.

$$k_{pm} = 0.276\ (1.036)^{10-20}$$

$$k_{pm} = 0.194\ \text{day}^{-1}$$

$$t = \frac{4}{0.194}\left[\left(\frac{200}{30}\right)^{1/4} -1\right] = 12.5\ \text{d}$$

$$t_1 = t_2 = t_3 = t_4 = 12.5/4 = 3.1\ \text{d}$$

$$V_1 = 3.1\ (1893\ \text{m}^3/\text{d}) = 5868\ \text{m}^3\ (2.07 \times 10^5\ \text{ft}^3)$$

Acceptable depths from partial mix aerated ponds ranged from 1.5 to 4.5 m (5 to 15 ft). A 3-m (10-ft) depth is used in this example.

Calculate the area of the pond.

The ideal configuration of a pond designed on the basis of complete mix hydraulics is a circular or a square pond; however, even though partial mix ponds are designed using the complete mix model, it is recommended that the ponds be configured with a length to width ratio of 3:1 or 4:1. This is done because it is recognized that the hydraulic flow pattern in partial mix systems more closely resembles the plug flow model. As more field data become available, it is likely that reliable plug flow reaction rates will be developed, and the plug flow model can be used to size partial mix systems.

Ponds with sloped side walls (3:1) and a length to width ratio of 4:1 will be used. The dimensions of the ponds can be calculated using the formula for the volume of a rectangular pond with side slopes that was presented in Sections 3.2.1 and 3.3.7.

$$V = \left[(LxW) + (L-2sd)(W-2sd) + 4\ (L-sd)(W-sd)\right]\frac{d}{6}$$

where,

V = volume of pond = 5868 m^3 (2.07×10^5 ft^3)

L = length of pond at water surface, m

W = width of pond at water surface, m

s = horizontal slope factor, i.e., 3:1 slope, $s = 3$

d = depth of pond = 3 m (10 ft)

$L = 4W$

$$V\frac{6}{3} = (4WxW) + (4W-2x3x3)(W-2x3x3) + 4\ (4W-2x3)(W-2x3)$$

$$4W^2 + 4W^2 - 90W + 324 + 16W^2 - 120W + 144 = 2V$$

$$24W^2 - 210W = 2V - 468$$

$$W^2 - 8.75W = 0.08333V - 19.5$$

Solve the quadratic equation by completing the square.

$$W^2 - 8.75W + 19.14 = 469.5 + 19.14$$

$$(W - 4.375)^2 = 488.6$$

$$W - 4.375 = 22.10$$

$$W = 26.5 \text{ m } (86.9 \text{ ft})$$

$$L = 26.5 \times 4 = 106.0 \text{ m } (348 \text{ ft})$$

$$A = 106.0 \times 26.5 = 2809 \text{ m}^2 \; (3.02 \times 10^4 \text{ ft}^2)$$

Check the pond water temperature using the surface area of 2809 m^2 (30,236 ft^2) and the other known characteristics in the Mancini and Barnhart equation (Section 3.3.5).

$$T_w = \frac{AfT_a + QT_i}{Af + Q}$$

$$T_w = \frac{2809(0.5)(-5) + 1893(15)}{2809(0.5) + 1893}$$

$$T_w = 6.5°C \; (44°F)$$

Second iteration

The temperature of the pond water will be colder than originally estimated; therefore, the pond area required will increase and more heat will be lost. The next estimate will be lower than the value resulting from the first iteration.

Assume $T_w = 5°C$

$$k_{pm} = 0.276(1.036)^{5-20}$$

$$k_{pm} = 0.162 \text{ day}^{-1}$$

$$t = \left[\frac{4}{0.162} \left(\frac{200}{30}\right)^{1/4} -1\right] = 15.0 \text{ d}$$

$$t_1 = t_2 = t_3 = t_4 = 15/4 = 3.75 \text{ d}$$

$$V_1 = 3.75(1893) = 7099 \text{ m}^3 (2.51 \times 10^5 \text{ ft}^3)$$

$$W^2 - 8.75W = 0.08333V - 19.5$$

$$(W - 4.375)^2 = 572.06 + 19.14$$

$$W - 4.375 = 591.2$$

$$W = 28.7 \text{ m } (93 \text{ ft})$$

$$L = 28.7 \times 4 = 114.8 \text{ m } (371 \text{ ft})$$

$$A = 28.7 \times 114.8 = 3295 \text{ m}^2 \ (3.55 \times 10^4 \text{ ft}^2)$$

$$T_w = \frac{3295(0.5)(-5) + 1893(15)}{3295(0.5) + 1893}$$

$$T_w = 5.7°C \ (42°F)$$

The second value of T_w of 5.7°C is in close agreement with the assumed value of 5.0°C, and further refinement is not justified. The temperature will decrease as the wastewater flows from one pond to the next, but the change is not large enough to significantly affect the design based on an average temperature.

<u>Final Size of Ponds</u>

$$t = 15.0 \text{ d}$$

$$t_1 = t_2 = t_3 = t_4 = 3.75 \text{ d}$$

$$V = 28{,}396 \text{ m}^3 \ (1.00 \times 10^6 \text{ ft}^3)$$

$$V_1 = V_2 = V_3 = V_4 = 7{,}099 \text{ m}^3 \ (2.51 \times 10^5 \text{ ft}^3)$$

$$A = 13{,}180 \text{ m}^2 \ (1.42 \times 10^5 \text{ ft}^2)$$

$$A_1 = A_2 = A_3 = A_4 = 3295 \text{ m}^2 \ (3.55 \times 10^4 \text{ ft}^2)$$

A freeboard of 0.6 m (2 ft) should be provided. The dimensions of a single cell at the top of the inside of the dike will be 40.7 m x 126.8 m (131.4 ft x 409.5 ft).

The advantage of a four cell system can be demonstrated by considering the detention time and area that would be required for a two-cell system. Using the same assumptions as those used in the previous case, with n = 2 and T_w = 5°C:

<u>First iteration</u>

$$t = \frac{2}{0.162}\left[\left(\frac{200}{30}\right)^{1/2} - 1\right] = 19.5 \text{ d}$$

$t_1 = t_2 = 9.75$ d

$V_1 = 9.75\ (1893) = 18{,}457\ m^3\ (6.5x10^5\ ft^3)$

$W^2 - 8.75W = 0.08333V - 19.5$

$$W^2 - 8.75W = 1518.5$$

$$(W - 4.375)^2 = 1518.5 + 19.14$$

$$W - 4.375 = 39.2$$

$W = 43.6$ m (143 ft)

$L = 4 \times 43.6 = 174.4$ m (572 ft)

$A_1 = 7604\ m^2\ (81{,}850\ ft^2)$

$$T_w = \frac{7604(0.5)(-5) + 1893(15)}{7604(0.5) + 1893}$$

$T_w = 1.6°C\ (35°F)$

Second iteration

Let $T_w = 1°C$

$$k_{pm} = 0.276\ (1.036)^{1-20}$$

$$k_{pm} = 0.141\ day^{-1}$$

$$t = \frac{2}{0.141}\left[\left(\frac{200}{30}\right)^{1/2} -1\right] = 22.4 \text{ d}$$

$t_1 = t_2 = 11.2$ d

$V_1 = 11.2\ (1893) = 21{,}202\ m^3\ (7.49x10^5\ ft^3)$

$W^2 - 8.75W = 0.08333(21{,}202) - 19.5$

$$(W - 4.375)^2 = 1747 + 19.14$$

$$W - 4.375 = 42.0$$

$$W = 46.4 \text{ m (152 ft)}$$

$$L = 4 \times 46.4 = 185.6 \text{ m (609 ft)}$$

$$A = 46.4 \times 185.6 = 8612\ m^2\ (9.3 \times 10^4\ ft^2)$$

$$T_w = \frac{8612(0.5)(-5) + 1893(15)}{8612(0.5) + 1893}$$

$$T_w = 1.1°C\ (34°F)$$

The assumed value of T_w of 1.0°C is in excellent agreement with the calculated value of 1.1°C; therefore, the design is satisfactory. Using only 2 cells instead of 4 will increase the detention time required by approximately 50% and will increase the surface area and volume required by a factor of approximately 3. This would be undesirable for winter operations in cold climates because of the enhanced potential for ice formation and the additional cost for construction.

Equations are available to estimate the oxygen requirements in aerated ponds; however, the use of these equations requires assuming two or more parameters which have a wide range of values from which to choose. Experience has shown that basing the biological oxygen requirements on 1.5 times the mass of BOD_5 entering each cell is satisfactory in a mild climate such as the one described in this example. In cold climates where solids accumulation will be greater, the factor should be increased to 2.0 kg O_2/kg of BOD_5 applied to each cell.

Equipment manufacturers' catalogs must be consulted when selecting aerators. All types of equipment must be evaluated to ensure that the most economical system is selected. An economic analysis and good engineering judgment is required to select aeration equipment.

The following relationship is used to estimate aeration requirements:

$$N = \frac{N_a}{\alpha \left[\frac{C_{sw} - C_L}{C_s} \right] (1.025)^{T_w - 20}}$$

where,

N = equivalent oxygen transfer to tapwater at standard conditions, kg/hr

N_a = oxygen required to treat the wastewater, kg/hr

$\alpha = \frac{\text{oxygen transfer in wastewater}}{\text{oxygen transfer in tapwater}} = 0.9$

C_L = minimum DO concentration maintained in the waste, assume 2.0 mg/l

C_s = oxygen saturation value of tapwater at 20°C and one atmosphere pressure = 9.17 mg/l

T_w = wastewater temperature, °C

$C_{sw} = \beta (C_{ss})P$ = oxygen saturation value of the waste, mg/l

$$\beta = \frac{\text{wastewater oxygen saturation value}}{\text{tapwater oxygen saturation value}} = 0.9$$

C_{ss} = tapwater oxygen saturation value at temperature, T_w

P = ratio of barometric pressure at plant site to barometric pressure at sea level, assume 1.0 for the elevation of 100 m

The maximum oxygen transfer will be required in the summer months. The summer water temperature in the ponds can be estimated using the Mancini and Barnhart equation.

$$T_w = \frac{3295(0.5)(30) + 1893(15)}{3295(0.5) + 1893}$$

T_w = 21.98 22°C (72°F)

C_{ss} = 8.72 mg/l

BOD_5 in the influent wastewater = $C_o \times Q$

= (200 g/m^3)(1893 m^3/d)(kg/1000 g)(d/24 hr)

= 16 kg/hr

BOD_5 in the effluent from pond number one can be calculated as follows:

$$\frac{C_n}{C_o} = \frac{1}{\left[\frac{k_c t}{n} + 1\right]^n}$$

$$\frac{C_1}{200} = \frac{1}{\left[\frac{0.162(3.75)}{1} + 1\right]^1}$$

C_1 = 124 mg/l

BOD_5 in the influent to pond number two =

$C_1 \times Q$ = 124 mg/l (1893 m^3/d)(kg/1000 g)(d/24 hr) = 10 kg/hr

BOD_5 in effluent from pond number two:

$$\frac{C_2}{124} = \frac{1}{\left[\frac{0.162(3.75)}{2} + 1\right]^2}$$

C_2 = 73 mg/l

BOD_5 in influent to pond number three =

$C_2 \times Q = 73(1893)(1/1000)(1/24) = 6$ kg/hr

BOD_5 in effluent from pond number three:

$$\frac{C_3}{73} = \frac{1}{\left[\frac{0.162(3.75)}{3} + 1\right]^3}$$

C_3 = 42 mg/l

BOD_5 in influent to pond number four =

$C_3 \times Q = 42(1893)(1/1000)(1/24) = 3$ kg/hr

Assume that the oxygen demand of the wastewater and solids at peak flow will be 1.5 times the mean oxygen demand entering each cell. Therefore,

N_{a_1} = 1.5 x 16 kg/hr = 24 kg/hr

N_{a_2} = 1.5 x 10 kg/hr = 15 kg/hr

N_{a_3} = 1.5 x 6 kg/hr = 9 kg/hr

N_{a_4} = 1.5 x 3 kg/hr = 4.5 kg/hr

where, the subscripts 1 through 4 represent ponds 1 through 4.

C_{sw} = 0.9 (8.72 mg/l) 1.0 = 7.85 mg/l

<u>O_2 Requirements</u>

Pond #1:

$$N_1 = \frac{24 \text{ kg } O_2/\text{hr}}{0.9\left[\frac{7.85 - 2.0}{9.17}\right](1.025)^{22-20}} = 39.7 \text{ kg } O_2/\text{hr}$$

Pond #2:

$$N_2 = \frac{15 \text{ kg } O_2/\text{hr}}{0.9\left[\frac{7.85 - 2.0}{9.17}\right](1.025)^{22-20}} = 24.8 \text{ kg } O_2/\text{hr}$$

Pond #3:

$$N_3 = \frac{9 \text{ kg } O_2/\text{hr}}{0.9\left[\frac{7.85 - 2.0}{9.17}\right](1.025)^{22-20}} = 14.9 \text{ kg } O_2/\text{hr}$$

Pond #4:

$$N_4 = \frac{4.5 \text{ kg } O_2/\text{hr}}{0.9\left[\frac{7.85 - 2.0}{9.17}\right](1.025)^{22-20}} = 7.4 \text{ kg } O_2/\text{hr}$$

The use of both surface and diffused air aerators will be illustrated. Using surface aerators, a value of 1.9 kg O_2/kWh (1.4 kg/hr/hp) is recommended to estimate the power requirements. A value of 2.7 kg O_2/kWh (2 kg/hp/hr) is recommended for diffused air aeration systems by the manufacturers. The gas transfer rate must be verified for the equipment selected. The total power required to satisfy the oxygen demand in the ponds using surface aerators is:

Pond #1:

$$\frac{39.7 \text{ kg } O_2/\text{hr}}{1.9 \text{ kg } O_2/\text{kWh}} = 20.9 \text{ kW } (28.0 \text{ hp})$$

Pond #2:

$$\frac{24.8 \text{ kg } O_2/\text{hr}}{1.9 \text{ kg } O_2/\text{kWh}} = 13.1 \text{ kW } (17.5 \text{ hp})$$

Pond #3:

$$\frac{14.9 \text{ kg } O_2/\text{hr}}{1.9 \text{ kg } O_2/\text{kWh}} = 7.8 \text{ kW } (10.5 \text{ hp})$$

Pond #4:

$$\frac{7.4 \text{ kg } O_2/\text{hr}}{1.9 \text{ kg } O_2/\text{kWh}} = 3.9 \text{ kW } (5.2 \text{ hp})$$

If diffused air aerators are to be used, the same procedure is performed as shown above with the exception being to divide by the value of 2.7 kg O_2/kWh. The power requirements for a diffused air system is calculated as follows:

Pond #1:

$$\frac{39.7 \text{ kg } O_2/\text{hr}}{2.7 \text{ kg } O_2/\text{kWh}} = 14.7 \text{ kW } (19.7 \text{ hp})$$

Pond #2:

$$\frac{24.8 \text{ kg } O_2/\text{hr}}{2.7 \text{ kg } O_2/\text{kWh}} = 9.2 \text{ kW } (12.3 \text{ hp})$$

Pond #3:

$$\frac{14.9 \text{ kg } O_2/\text{hr}}{2.7 \text{ kg } O_2/\text{kWh}} = 5.5 \text{ kW } (7.4 \text{ hp})$$

Pond #4:

$$\frac{7.4 \text{ kg } O_2/\text{hr}}{2.7 \text{ kg } O_2/\text{kWh}} = 2.7 \text{ kW } (3.7 \text{ hp})$$

The surface and diffused air aerators power requirements must be corrected for gearing or blower efficiency. Assuming 90 percent efficiency for both gearing and blower efficiency; the total motor power required is calculated as follows:

Pond #1:

$$\frac{20.9 \text{ kW}}{0.9} = 23.2 \text{ kW } (31.1 \text{ hp})$$

The values for the other ponds are calculated as shown above. The motor power requirements are summarized in Table 3-5.

The values shown in Table 3-5 represent approximate power requirements and are used to select aeration equipment. The actual power requirements for the ponds using surface aeration will be determined by using the zone of complete oxygen dispersion reported by equipment manufacturers and the above power requirements.

TABLE 3-5

MOTOR POWER REQUIREMENTS FOR SURFACE AND DIFFUSED AIR AERATORS

Pond Number	POWER REQUIREMENTS, kW Surface Aerators	Diffused Air Aerators
#1	23.2	16.3
#2	14.6	10.2
#3	8.7	6.1
#4	4.3	3.0
TOTAL	50.8	35.6

Using data presented in a catalog by Aqua-Aerobic Systems, Inc. (27), ten 2.2-kW (3-hp) surface aerators in the first pond would provide a zone of complete oxygen dispersion with a diameter of 41.2 m (135 ft) and a zone of complete mixing with a diameter of 13 m (42 ft) without draft tubes or anti-erosion assemblies in a pond with a depth of 3 m (10 ft). The arrangement of the aerators is shown in Figure 3-4. The aerators provide considerable overlap in the zones of complete oxygen dispersion as well as providing a zone of complete mixing at even intervals along the tank to disperse any channeling of the flow that may develop. Similar selections and arrangements can be developed for the remaining three cells but with decreasing power requirements as shown in Table 3-5. The main concern in the selection of aerators is that there is considerable overlap of the zones of complete oxygen dispersion.

Diffused air aeration requirements for all four ponds are 35.6 kW (47.7 hp). Manufacturers supply motor-blower combinations in 10, 15, 20, etc. hp; therefore, to meet the 47.7-hp (35.6-kW) requirement for aeration and to provide flexibility in maintenance and operation, three 15-hp (11.2-kW) and two 10-hp (7.5-kW) motor-blowers are specified. The blowers are to be operated in combinations providing 50 hp (37.3 kW) of aeration with the remaining blowers in reserve.

When the air is distributed with fine bubble perforated tubing (Hinde Engineering Company) the quantity of air added to a pond is assumed to be directly proportional to the length of tubing placed in a pond. Approximately 50-60 percent of the tubing in the first pond is placed within the first third of the pond. The tubing in the remaining ponds can be spaced at equal intervals along the length of the ponds in proportion to the power requirements in each cell. The distribution of the tubing is shown in Figure 3-5.

Use of the fine bubble perforated tubing requires that a diligent maintenance program be established. Many communities have experienced clogging of the perforations, particularly in hard water areas. If this method of aeration is specified, the design engineer must emphasize the importance of adhering to the maintenance schedule.

FIGURE 3-4

LAYOUT OF SURFACE AERATORS IN
FIRST CELL OF PARTIAL MIX SYSTEM

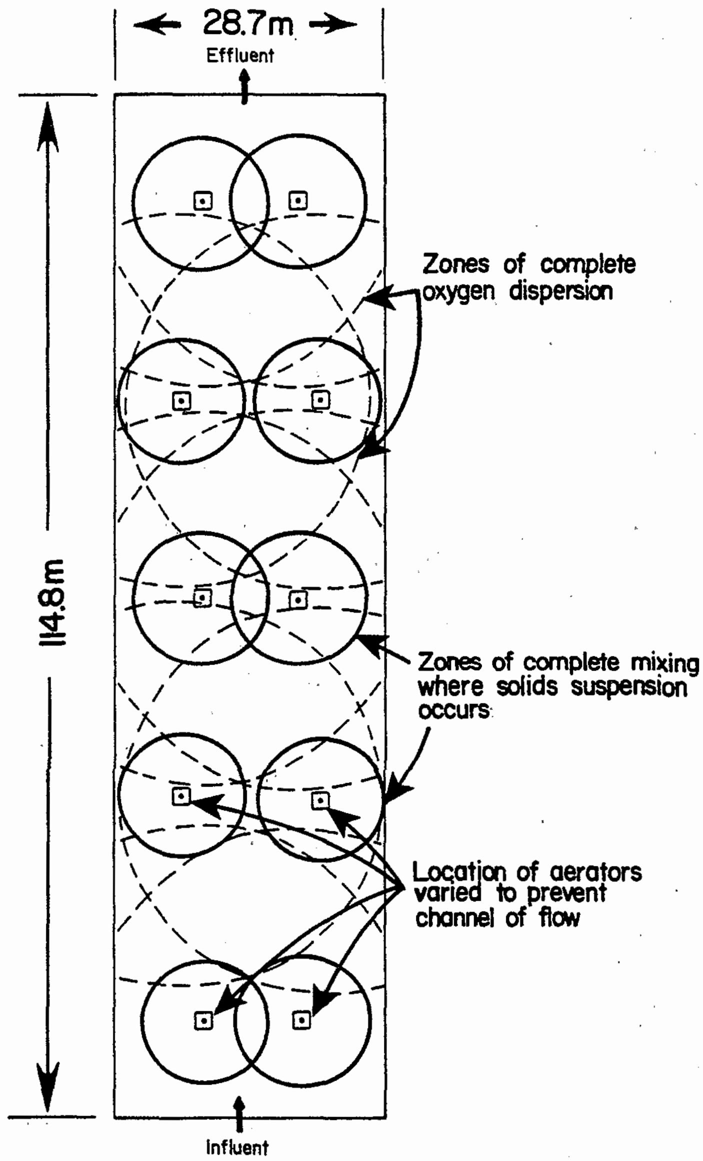
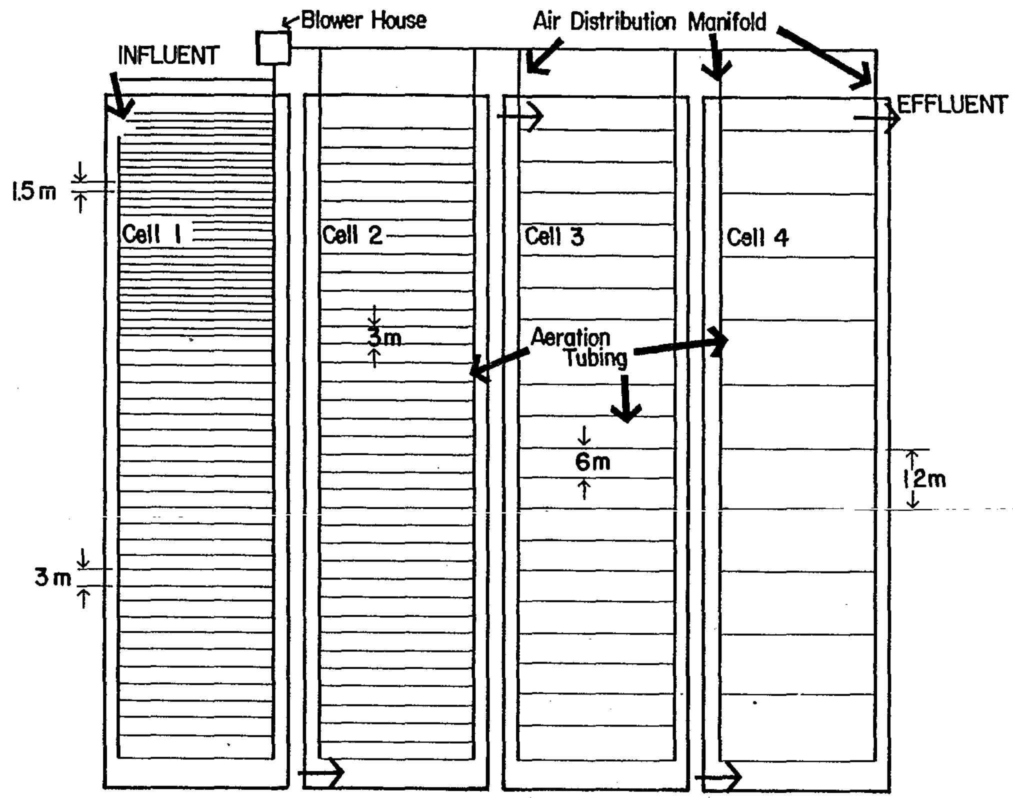


FIGURE 3-5

LAYOUT OF AERATION SYSTEM FOR PARTIAL MIX
DIFFUSED AIR AERATED POND SYSTEM



A settling pond with a two-day detention time is provided after the final aerated pond.

Summary

$V = 28,396\ m^3$

$A = 13,180\ m^2$

$t = 15.0\ d$

O_2 required for treatment (See Table 3-5)

$n = 4$

3.5 Controlled Discharge Ponds

No rational or empirical design model exists specifically for the design of controlled discharge wastewater ponds. However, rational and empirical design models applied to facultative pond design may also be applied to the design of controlled discharge ponds provided allowance is made for the required larger storage volumes. These larger volumes result from the long storage periods relative to the very short discharge periods. Application of the ideal plug flow model developed for facultative ponds can be applied to controlled discharge ponds if hydraulic residence times of less than 120 days are considered. A study of 49 controlled discharge ponds in Michigan indicated that discharge periods very from less than five days to more than 31 days, and residence times were 120 days or greater (30).

The following design and operating information for controlled discharge ponds were extracted from a report entitled "Wastewater Treatment Ponds" (15). The unique features of controlled discharge ponds are long-term retention and periodic, controlled discharge usually once or twice a year. Ponds of this type have operated satisfactorily in the north-central U.S. using the following design criteria:

Overall organic loading: 22-28 kg BOD_5/ha/d (20-25 lb BOD_5/ac/d).

Liquid depth: not more than 2 m (6 ft) for the first cell, not more than 2.5 m (8 ft) for subsequent cells.

Hydraulic detention: At least 6 months of storage above the 0.6-m (2-ft) liquid level (including precipitation), but not less than the period of ice cover.

Number of cells: At least 3 for reliability, with piping flexibility for parallel or series operation.

The design of the controlled discharge pond must include an analysis showing that receiving stream water quality standards will be maintained during discharge intervals, and that the receiving watercourses can accommodate the discharge rate from the pond. The design must also include a recommended discharge schedule.

Selecting the optimum day and hour for release of the pond contents is critical to the success of this method. The operation and maintenance manual must include instructions on how to correlate pond discharge with effluent and stream quality. The pond contents and stream must be carefully examined, before and during the release of the pond contents.

In a typical program, discharge of effluents follows a consistent pattern for all ponds. The following steps are usually taken:

1. Isolate the cell to be discharged, usually the final one in the series, by valving-off the inlet line from the preceding cell.

2. Arrange to analyze samples for BOD, suspended solids, volatile suspended solids, pH, and other parameters which may be required for a particular location.

3. Plan to work so as to spend full time on control of the discharge throughout the period.

4. Sample contents of the cell to be discharged for dissolved oxygen, noting turbidity, color, and any unusual conditions.

5. Note conditions in the stream to receive the effluent.

6. Notify the state regulatory agency of results of these observations and plans for discharge and obtain approval.

7. If discharge is approved, commence discharge, and continue so long as weather is favorable, dissolved oxygen is near or above saturation values and turbidity is not excessive following the prearranged discharge flow pattern among the cells. Usually this consists of drawing down the last two cells in the series (if there are three or more) to about 46 to 60 cm (18 to 24 in) after isolation, interrupting the discharge for a week or more to divert raw waste to a cell which has been drawn down and resting the initial cell before its discharge. When this first cell is drawn down to about 60 cm (24 in) depth, the usual series flow pattern, without discharge, is resumed. During discharge to the receiving waters, samples are taken at least three times each day near the discharge pipe for immediate dissolved oxygen analysis. Additional testing may be required for suspended solids.

Experience with these ponds is limited to northern states with seasonal and climatic influences on algae growth. The concept will be quite effective for BOD removal in any location. The process will also work with a more frequent discharge cycle than semi-annually, depending on receiving water conditions and requirements. Operating the isolation cell on a fill-and-draw batch basis is similar to the "phase isolation" technique discussed in Chapter 5.

3.5.1 Design Example

In areas of high evaporation rates or high rainfall, the volume of the pond should be adjusted to compensate for the water loss or gain. In this example, it is assumed that rainfall is equal to evaporation, producing no net change in volume. This example illustrates the design of a controlled discharge pond using a minimum discharge period criterion.

Design Conditions:

Minimum discharge period = 30 d

Q = Design flow rate = 1893 m^3/d (0.5 mgd)

C_o = influent BOD_5 = 150 mg/L

C_e = effluent BOD_5 = 30 mg/L

$k_{P_{20}}$ = reaction rate for plug flow at 20°C = 0.1 day^{-1}

T_w = water temperature critical period of the year = 2°C

Requirements: Size a controlled discharge wastewater pond system to treat the wastewater and specify the following parameters:

1) Detention time, t

2) Volume, V

3) Surface area, A

4) Depth, d

5) Length, L

6) Width, W

Solution: t = 365 d - minimum discharge period

$$= 365 \text{ d} - 30 \text{ d} = 335 \text{ d}$$

Discharge can occur when the effluent quality satisfies standards or the receiving stream flow rate is adequate to receive the effluent. More frequent discharge periods than once a year can be employed, but it is necessary to evaluate the performance of the system for shorter hydraulic residence times. The methods used to design facultative ponds can be used to estimate the performance of a controlled discharge pond.

Raw wastewater is not added to the pond being emptied. Raw wastewater inlets and effluent withdrawal ports are provided in each cell of the system. The cells are connected in series to facilitate operation and flexibility. Three cells are used in this example.

An effective depth (d') of 1.5 m (5 ft) and a total depth (d) of 2 m (6.6 ft) is used. This depth allows for adequate light penetration to sustain photosynthetic oxygen production, providing an aerobic environment through much of the pond contents. The aerobic environment enhances treatment and reduces odor problems. Also, to control odors during discharge periods, the pond is emptied to a minimum depth of 0.5 m (1.5 ft). Additional volume must be provided to compensate for this minimum withdrawal depth.

$$A/cell = \frac{\text{effective volume}}{\text{n x effective depth}} = \frac{Q \times t}{3 \times d'} \quad \frac{1893 \text{ m}^3/\text{d} \times 335 \text{ d}}{3 \times 1.5 \text{ m}}$$

$$= 140{,}900 \text{ m}^2 \text{ (35 ac)}$$

This area is used to calculate the total volume for the pond total depth:

$$d = 1.5 \text{ m} + 0.5 \text{ m} = 2 \text{ m (6.6 ft)}$$

$$V/cell = (A/cell)(d) = (140{,}900 \text{ m}^2)(2 \text{ m})$$

$$= 281{,}800 \text{ m}^3 \text{ } (74.4 \times 10^6 \text{ gal})$$

Significant volumes of wastewater may be lost through seepage if the pond bottom is not scaled. For this example seepage rates are considered minimal.

The length to width ratio of the cells in a controlled discharge pond has less affect on the performance of the system than in flow through systems. Dimensions for the cells are selected to avoid short circuiting during discharge or inter-basin transfer. A length to width ratio of 2:1 was selected for this example.

Dimensions of Ponds

The dimensions of each pond with side slopes of 4:1 and a length to width ratio of 2:1 can be calculated using the formula first presented in Section 3.2.

$$V = \left[(L \times W) + (L - 2sd)(W - 2sd) + 4\,(L - sd)(W - sd)\right]\frac{d}{6}$$

where,

V_1 = volume of pond #1 - 281,800 m^3

L = length of pond at water surface, m

W = width of pond at water surface, m

s = horizontal slope factor, i.e., 4:1 slope, s = 4

d = depth of pond = 2 m (6.6 ft)

$$(281{,}800)\ \frac{6}{2} = (L \times \frac{L}{2}) + (L - 2x4x2)(\frac{L}{2} - 2x4x2)$$

$$+ 4\ (L - 4x2)(\frac{L}{2} - 4x2)$$

$$3L^3 - 72L + 512 = 845{,}400$$

$$L^2 - 24L = 281{,}630$$

Solve the quadratic equation by completing the square.

$$L^2 - 24L + 144 = 281{,}630 + 144$$

$$(L-12)^2 = 281{,}774$$

$$L-12 = 530.8$$

$$L = 542.8 \text{ m (1780 ft)}$$

$$W = 542.8/2 = 271.4 \text{ m (890 ft)}$$

A freeboard of 0.6 m (2 ft) should be provided. The dimensions of each pond at the top of the inside of the dike will be 547.6 m x 276.2 m. The three ponds shall be interconnected by piping for parallel and series operation.

Effluent Quality Prediction

In a pond with a hydraulic residence time of over 300 days, it is obvious that an effluent with a BOD_5 concentration of less than 30 mg/l can be achieved. However, if it becomes necessary to discharge at other intervals of time, some method of estimating the effluent quality is needed. Controlled discharge ponds are basically a facultative pond, and the effluent quality can be predicted using the plug flow model used to design a facultative pond in Section 3.2.

$$\frac{C_e}{C_o} = e^{-k_p t}$$

where,

C_e = effluent BOD_5 concentration, mg/l

C_o = influent BOD_5 concentration, mg/l

e = base of natural logarithms, 2.7183

k_p = plug flow first order reaction rate, day^{-1}

t = hydraulic residence time, d

$$k_{p_t} = k_{p_{20}} (1.09)^{T_w - 20}$$

k_{p_t} = reaction rate at minimum operating water temperature, day^{-1}

$k_{p_{20}}$ = reaction rate at 20°C = 0.1 day^{-1}

T_w = minimum operating water temperature, °C.

Assume that it becomes necessary to discharge from the ponds after a mean hydraulic residence time of 100 days when the mean water temperature during the period was 2°C. What would be the concentration of BOD_5 in the effluent?

$$k_{p_t} = 0.1\ (1.09)^{2-20}$$

$$k_{p_t} = 0.021\ day^{-1}$$

$$\frac{C_e}{150} = e^{-0.021(100)}$$

$$C_e = 18 \text{ mg/l}$$

The BOD_5 concentration of 18 mg/l in the effluent will easily satisfy the standard of 30 mg/l. Suspended solids concentrations will have to be monitored on site to ensure that the standards for discharge are met. The guidelines presented at the beginning of this example must be followed in operating the controlled discharge pond system.

Summary

$V = 845{,}400\ m^3$

$A = (542.8)(271.4)(3) = 441{,}950\ m^2$

$t = 335\ d$

3.6 Complete Retention Ponds

In areas of the U.S. where the moisture deficit, evaporation minus rainfall, exceeds 75 cm (30 in) annually, a complete retention wastewater pond may prove to be the most economical method of disposal. Complete retention ponds must be sized to provide the necessary surface area to evaporate the total annual wastewater volume plus the precipitation that would fall on the pond. The system should be designed for the maximum wet year and minimum evaporation year of record if overflow is not permissible under any circumstances. Less stringent design standards may be appropriate in situations where occasional overflow is acceptable or an alternative disposal area is available under emergency conditions.

Monthly evaporation and precipitation rates must be known to properly size the system. Complete retention ponds usually require large land areas, and these areas are not productive once they have been committed to this type of system. Land for this system must be naturally flat or be shaped to provide ponds that are uniform in depth, and have large surface areas. The design procedure for a complete retention wastewater pond system is presented in the following example.

Design Conditions:

Table 3-6 presents data from NOAA (31) for estimating evaporation and precipitation in southern Arizona. The air temperature and wind speed data represent mean values over a 54- and 61-year period, respectively. The precipitation data are the mean of the five wettest years over a 60-year period. The pan evaporation data represent the year with the lowest evaporation for a 10-year period. These values generally represent the worst case, thus providing for a conservative design.

The difference between the surface water temperature and the air temperature is assumed to be 1°C. The selection of this value can have a significant effect on the evaporation losses as shown in Figure 3-7; therefore, the value must be selected to reflect local conditions.

Surface water temperature = T_o = air temperature (T_a) minus 1°C.

T_o - Ta = -1°C.

Q = 950 m^3/d (0.25 mgd)

Influent BOD_5 = 150 mg/l

Seepage = 0.80 mm/d (0.2 in/wk) (32). Seepage is prohibited in some areas. State agency wastewater facility standards may require the pond bottom be sealed with an impervious liner, reducing seepage to zero.

Elevation = 300 m (980 ft) above MSL.
Requirements: Size a complete retention wastewater pond with no overflow for the given geographic area. Specify the following:

1. Area, A = $\dfrac{\text{Q (365 d/yr)}}{\text{d-(Annual Precip.-Annual Evap.-Annual Seepage)}}$

2. Surface area, A

3. Depth, d

4. Length, L

5. Width, W

Solution: The design procedure consists of the following steps:

1. Using the data in Table 3-6 with Figure 3-6 (Elevation = 305 m) and Figure 3-7, determine the mean monthly evaporation from the pond. The calculation of pond evaporation is shown on the figures by dashed lines. The results are presented in Table 3-7.

2. Using the data presented in Tables 3-6 and 3-7, calculate the area required for an assumed mean depth for one year of operation under design conditions. The mean depth (d) may range from 0.1 to 1.5 m (0.3 to 5.0 ft). The mean depth is usually near 1 m (3 ft).

3. Use the A value determined in step 2 to calculate the stage of the pond at the end of each month of operation during the design year.

4. Calculate the monthly stage of the pond under average conditions. If the pond is designed to never overflow, the average yearly evaporation and seepage must exceed the inflow and precipitation entering the pond.

5. Repeat steps 2 and 3 until a satisfactory pond depth is obtained.

TABLE 3-6

CLIMATOLOGICAL DATA FOR CALCULATING POND EVAPORATION AND PRECIPITATION

Month (Days in Month)	Mean Precipitation mm/month	Air Temp. °C	Wind Speed kts., day[a]	Minimum Ten-Year Pan Evaporation mm/month	Minimum Ten-Year Pan Evaporation mm/day	Mean Ten-Year Pan Evaporation mm/month
January (31)	12.3	12.4	140.4	87.5	2.82	105.0
February (28)	12.1	14.9	148.7	130.5	4.66	177.5
March (31)	10.1	17.7	154.2	198.2	6.39	220.0
April (30)	4.2	21.0	157.0	238.1	7.94	271.4
May (31)	2.9	24.6	154.2	332.0	10.71	365.2
June (30)	2.2	29.3	134.9	374.4	12.48	423.1
July (31)	6.8	32.8	140.4	416.0	13.42	449.3
August (31)	15.9	32.4	134.9	347.8	11.22	389.5
September (30)	10.6	29.1	115.6	278.5	9.28	323.1
October (31)	7.8	22.6	110.1	210.4	6.82	219.9
November (30)	8.3	16.8	126.6	137.4	4.58	163.5
December (31)	14.6	12.9	143.2	95.2	3.07	131.4
TOTAL	107.8			2847.0		3238.9

[a]Kts = knots = total of nautical miles/hr of wind per day.

FIGURE 3-6

PORTION OF ADVECTED ENERGY (INTO A CLASS A PAN) UTILIZED FOR EVAPORATION IN METRIC UNITS (33)

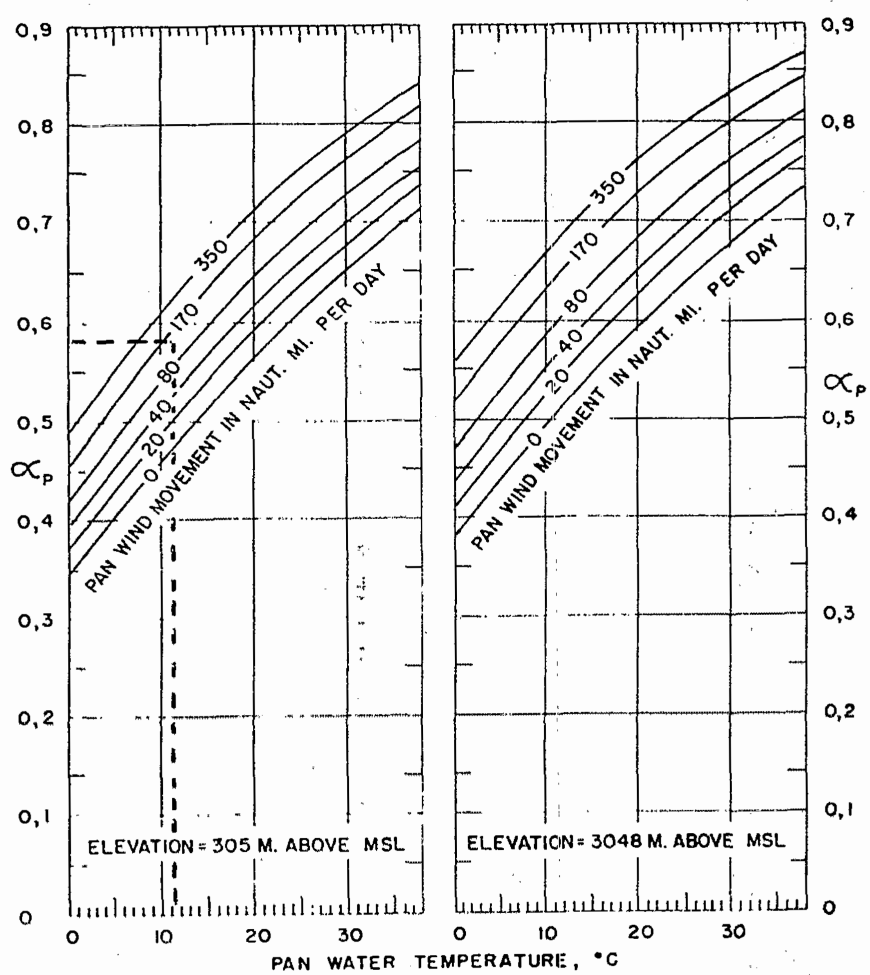


FIGURE 3-7

SHALLOW LAKE EVAPORATION AS A FUNCTION OF CLASS A PAN EVAPORATION AND HEAT TRANSFER THROUGH THE PAN IN METRIC UNITS PER DAY (33)

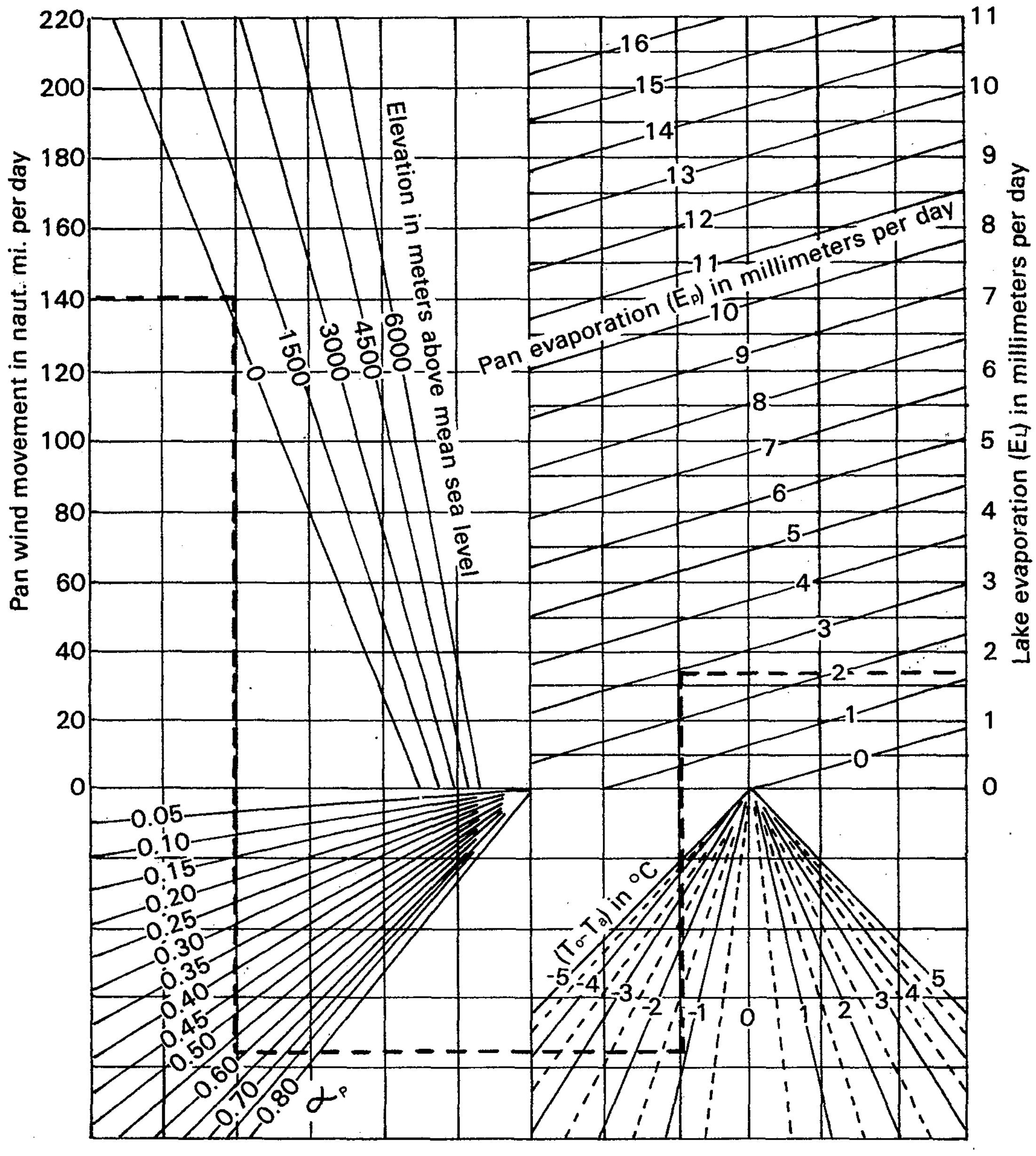


Table 3-7

CALCULATED POND EVAPORATION DATA

Month	αp (Fig. 3-6)	Pond Evaporation (Fig. 3-7) mm/day	Pond Evaporation (Fig. 3-7) mm/month
January	0.58	1.7	53
February	0.62	3.0	84
March	0.64	4.1	127
April	0.66	5.2	156
May	0.71	7.2	223
June	0.74	8.4	252
July	0.77	9.0	279
August	0.77	7.4	229
September	0.73	6.2	186
October	0.58	4.4	136
November	0.62	2.9	87
December	0.58	1.8	56
TOTAL			1868

As a starting point, select a mean depth of 0.4 m to estimate the required surface area to evaporate the wastewater.

$$A = \frac{(946\ m^3/d)(365\ d/yr)}{0.4 - (0.1078 - 1.868 - 0.277)}$$

$$= 142{,}259\ m^2\ (35\ ac)$$

Use A = 142,300 m^2 (35 ac) to calculate the stage of the pond at the end of each month of operation. Table 3-8 contains a summary of the results of this procedure for the design year of operation assuming the pond is empty at the beginning of the year.

An examination of the pond stage results in Table 3-8 shows that the maximum depth of water in the pond during the design year (conservative design data) would be 0.60 m (2 ft) plus the depth at the beginning of the design year. The pond stage under average conditions is shown in Table 3-9. Average evaporation and seepage are within 5 percent of inflow and precipitation. Assuming that several average years would occur in sequence, there would be a small accumulation of water in the pond. Because of the imprecise methods available to predict the sequence of occurrence of the design year, maximum, and average years, the pond surface area of 142,300 m^2 is large enough to prevent overflow of the pond.

The depth of complete retention ponds is limited only by groundwater conditions, economics, and evaporation rates. Generally maximum depths range from 1.0 to 3 m (3 to 10 ft) with a freeboard of 0.6 m (2 ft).

TABLE 3-8

VOLUME AND STAGE OF POND AT MONTHLY INTERVALS FOR DESIGN CONDITIONS AND A = 142,300 m^2

Month (No. of Days in Month)	Inflow + Precipitation[a] (m^3)	Evaporation + Seepage[b] (m^3)	Storage Volume[c] (m^3)	Pond Stage[d] (m)
Starting Date 1				
September (30)	29,888	29,712	176	0.00
October (31)	30,436	22,706	7,906	0.06
November (30)	29,561	15,624	21,843	0.15
December (31)	31,404	11,322	41,925	0.29
January (31)	31,076	10,895	62,106	0.44
February (28)	28,210	14,981	75,335	0.53
March (31)	30,763	21,425	84,673	0.60
April (30)	28,978	25,443	88,208	0.62
May (31)	29,739	35,086	82,861	0.58
June (30)	28,693	39,104	72,450	0.51
July (31)	30,294	43,055	59,689	0.42
August (31)	31,589	35,940	55,338	0.39
TOTAL	360,631	305,293		
Starting Date 2				
January (31)	31,076	10,895	20,181	0.14
February (28)	28,210	14,981	33,410	0.23
March (31)	30,763	21,425	42,748	0.30
April (30)	28,978	25,443	46,283	0.33
May (31)	29,739	35,086	40,936	0.29
June (30)	28,693	39,104	30,525	0.21
July (31)	30,294	43,055	17,764	0.12
August (31)	31,589	35,940	13,413	0.09
September (30)	29,888	29,712	13,589	0.10
October (31)	30,436	22,706	21,319	0.15
November (30)	29,561	15,624	35,256	0.25
December (31)	31,404	11,322	55,338	0.39

[a] Inflow = Q(no. of days/month); precipitation = (monthly precipitation) (A).
[b] Seepage = 0.00076 m/d(no. of days/month)(A); evaporation = (monthly evaporation)(A).
[c] Storage V = cumulative sum of (inflow + precipitation) - (evaporation + seepage).
[d] Pond stage = storage V/A.

TABLE 3-8 (continued)

Month (No. of Days in Month)	Inflow + Precipitation[a] (m^3)	Evaporation + Seepage[b] (m^3)	Storage Volume[c] (m^3)	Pond Stage[d] (m)
Starting Date 3				
December (31)	31,404	11,322	20,082	0.14
January (31)	31,076	10,895	40,263	0.28
February (28)	28,210	14,981	53,492	0.38
March (31)	30,763	21,425	62,830	0.44
April (30)	28,978	25,443	66,365	0.47
May (31)	29,739	35,086	61,018	0.43
June (30)	28,693	39,104	50,607	0.36
July (31)	30,294	43,055	37,846	0.27
August (31)	31,589	35,940	33,495	0.24
September (30)	29,888	29,712	33,671	0.24
October (31)	30,436	22,706	41,401	0.29
November (30)	29,561	15,624	55,338	0.39

[a]Inflow = Q(no. of days/month); precipitation = (monthly precipitation)(A).
[b]Seepage = 0.00076 m/d(no. of days/month)(A); evaporation = (monthly evaporation)(A).
[c]Storage V = cumulative sum of (inflow + precipitation) - (evaporation + seepage).
[d]Pond stage = storage V/A.

The maximum depth required will depend upon the time of year that filling of the pond begins and the initial depth of water in the pond when the design year occurs. It is impossible to predict accurately the water stage in the pond; therefore, it is necessary to exercise good judgment based upon the constraints at particular locations. Estimates beyond the average and design year conditions can be made by analyzing historical data for the site, but this still is no guarantee of accuracy.

The water depth in the pond after one year of operation under design conditions will be equal to the mean depth plus the depth of water at the beginning of the year. During certain months of the year, the depth may exceed the mean depth when filling of the pond occurs at the beginning of the wet season (Table 3-8). Three beginning dates are shown in Table 3-8 to illustrate the effects of startup date. Using the above procedure, it is possible for the design engineer to estimate the stage of the pond under as many conditions as considered necessary.

A maximum depth of 1.2 m (4 ft) would be adequate to avoid overflow from the pond by providing storage for five average years and a design year in sequence. It is unlikely that five average years of evaporation would precede the design year. The pond L and W values are calculated from the A. No restrictions are imposed on the length to width ratio. Also, the need to divide the pond volume to enhance hydraulic characteristics is eliminated. The most economical design consists of a single pond provided the system can be isolated enough to avoid complaints about odors when solids decompose on exposed slopes.

$$A = L \times W$$

$$L = W = A^{1/2} = (142{,}300\ m^2)^{1/2} = 377\ m\ (1{,}237\ ft)$$

Summary

1 pond $A = 142{,}300\ m^2$ (35 ac); $L = W = 377$ m (1,237 ft)

$V = 170{,}800\ m^3$ (45 Mgal); d = 1.2 m (3.9 ft)

TABLE 3-9

VOLUME AND STAGE OF POND AT MONTHLY INTERVALS FOR AVERAGE CONDITIONS AND A = 142,300 m^2

Month	Inflow + Precipitation (m^3)	Evaporation + Seepage (m^3)	Storage Volume (m^3)	Pond Stage (m)
Average Year				(no accumu-
September	29,888	33,947	-4,059	0.00 lation)
October	30,436	23,480	6,956	0.05
November	29,561	17,976	18,541	0.13
December	31,404	14,351	35,594	0.25
January	31,076	12,404	54,266	0.38
February	28,210	19,284	63,192	0.44
March	30,763	23,413	70,542	0.48
April	28,978	28,551	70,969	0.50
May	29,739	38,259	62,449	0.44
June	28,693	43,766	47,376	0.33
July	30,294	46,231	31,439	0.22
August	31,589	39,851	23,177	0.16
TOTAL	360,631	341,513		

3.7 Combined Systems

In certain situations it is desirable to design pond systems in combinations, i.e., an aerated pond followed by a facultative or a tertiary pond. Combinations of this type are designed essentially the same as the individual ponds. For example, the aerated pond would be designed as illustrated in Sections 3.3 or 3.4, and the predicted effluent quality from the aerated pond would be the influent quality for the facultative pond. The facultative pond would be designed as shown in Section 3.2. For more discussion on combined pond systems see References 20, 28, and 34.

3.8 References

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CHAPTER 4

PHYSICAL DESIGN AND CONSTRUCTION

4.1 Introduction

Regardless of the care taken to evaluate coefficients and apply biological or kinetic models, if sufficient consideration is not given to optimization of the pond layout and construction, the actual efficiency may be far less than the calculated efficiency. The physical design of a wastewater pond is as important as the biological and kinetic design. The biological factors affecting wastewater pond performance are primarily employed to estimate the required hydraulic residence time to achieve a specified efficiency. Physical factors, such as length to width ratio, determine the actual treatment efficiency achieved.

Length to width ratios are determined according to the design model used. Complete mix ponds should have a length to width ratio near 1:1, whereas plug flow ponds require length to width ratios of 3:1 or greater. The danger of groundwater contamination may impose seepage restrictions, necessitating lining or sealing the pond. Reuse of the pond effluent in dry areas where all water losses are to be avoided may also dictate the use of linings. Layout and construction criteria should be established to reduce dike erosion from wave action, weather, rodent attack, etc. Transfer structure placement and size affect flow patterns within the pond and determine operational capabilities in controlling the water level and discharge rate. These important physical design considerations are discussed in the sections that follow.

4.2 Dike Construction

Dike stability is most often affected by erosion caused by wind-driven wave action or rain and rain-induced weathering. Dikes may also be destroyed by burrowing rodents. A good design will anticipate these problems and provide a system which can, through cost-effective operation and maintenance, keep all three under control.

4.2.1 Wave Protection

Erosion protection should be provided on all slopes; however, if winds are predominantly from one direction, protection should be emphasized for those areas that receive the full force of the wind-driven waves. Protection should always extend from at least 0.3 m (1 ft) below the minimum water surface to at least 0.3 m (1 ft) above the maximum water surface (1). Wave height is a function of wind velocity and fetch (the distance over which the wind acts on the water). The size of riprap depends on the fetch length (2). Riprap varies from river run rocks that are 15-20 cm (6-8 in) to quarry boulders that are 7-14 kg (15-30 lb). Uniformly graded river run material, when used for riprap, can be quite unstable. River run rocks, if not properly mixed with smaller material and carefully placed, can be loosened by wave action and caused to slip down the steeper sloped dikes. Broken concrete pavement can often be used for riprap but can make mechanical weed control very difficult.

Asphalt, concrete, fabric, and low grasses can also be used to provide protection from wave action. When riprap is used for wave protection, the designer must take into consideration its effect on weed and rodent control, and routine dike maintenance.

4.2.2 Weather Protection

Dike slopes must be protected from weather erosion as much as from wave erosion in many areas of the country. The most common method of weather erosion protection when large dike areas are involved uses grass. Because of large variations in depth encountered in total containment ponds, they often have large sloped dike areas which cannot be protected in a more cost-effective way. Ponds which have only minimum freeboard and have constant water depth are often protected more cost effectively when the riprap is carried right to the top of the slope and serves for both wave and weather protection.

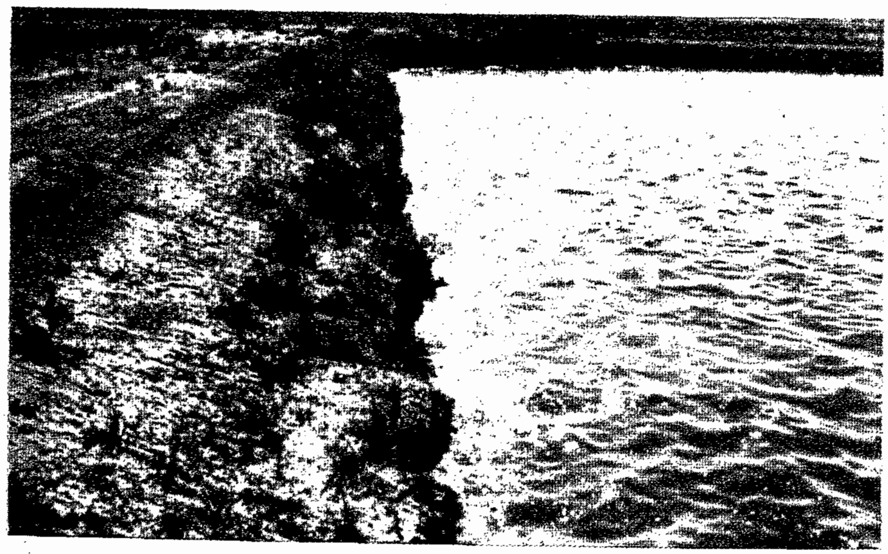
In some cases climate and soil conditions are suitable for completely bare dike slopes without major weather erosion problems. Figure 4-1 shows the erosion effects on the bare slopes of a stabilization pond.

Weather erosion, unlike wave erosion, can also affect the top and outside slopes of the pond diking sstem. The designer should make sure that the all-weather road system for the top of the dike is of sufficient width to allow traffic to pass over every part of the surface. Too narrow a road will result in ruts that can create runoff erosion problems in areas of high rain intensity. Final grading should be specified to minimize rutting and frequent maintenance and control surface runoff erosion.

It is also necessary to protect the exterior surface of dikes. A thin layer of gravel may be used; placement of topsoil and seeding with grass may be less expensive initially but grass requires periodic cutting. In some locations sheep can be used to keep exterior grass slopes maintained. Other native groundcover plantings may also be used. Local highway department experience on erosion control for cut-and-fill slopes can be a guide.

FIGURE 4-1

ERODED DIKE SLOPES ON A
RAW WASTEWATER POND IN A DRY CLIMATE



4.2.3 Rodent Protection

If a stabilization pond is located in an area that supports an exceptionally high population of burrowing animals, such as muskrats and nutria, good design can control this threat of dike stability. Broken concrete or other riprap that does not completely cover the dike soil can become a home for burrowing rodents. Riprap design and placement should be aimed toward limiting the creation of voids which allow rodents to burrow near the water surface.

Varying pond water depth can discourage muskrat infestation (3). Muskrats prefer a partially submerged tunnel, so design provisions to vary the water level over a several-week period will discourage them from burrowing in the dike. Such provisions will often add to the expense of riprap placement for wave protection but can greatly reduce operation and maintenance expenses.

4.2.4 Seepage

Dikes should be designed and constructed to minimize seepage. Vegetation and porous soils should be removed and the embankment should be well compacted.

Use of conventional construction equipment is usually suitable for this purpose.

Seepage collars should be provided around any pipes penetrating the dike (4) (5). The seepage collars should extend a minimum of 0.6 m (2 ft) from the pipe. Proper installation of transfer pipes can be assured by building up the dike at least 0.6 m (2 ft) above the pipe elevation, digging a trench for the pipe and seepage collar, backfilling the trench, and compacting the backfill.

In some circumstances it may be necessary to control seepage and ensure bank stability at the exterior toe. A filter blanket material can be used (6). Another method of preventing seepage where embankment material cannot be adequately compacted is placement of an impervious core in the levee with imported material.

4.3 Pond Sealing

4.3.1 Introduction

The need for a well-sealed stabilization pond has impacted modern pond design, construction, and maintenance. The primary motive for sealing ponds is to prevent seepage. Seepage effects treatment capabilities by causing fluctuation in the water depth and can cause pollution of groundwater. Although many types of pond sealers exist, they can be classified into one of three major categories: (1) synthetic and rubber liners, (2) earthen and cement liners, and (3) natural and chemical treatment sealers. Within each category also exists a wide variety of application characteristics. Choosing the appropriate lining for a specific site is a critical issue in pond design and seepage control. Detailed information is available from manufacturers, and in other publications (4)(6).

4.3.2 Seepage Rates

Stander et al. (7) presented a summary of information (Table 4-1) on measured seepage rates in wastewater stabilization ponds. Seepage is a function of so many variables that it is impossible to anticipate or predict rates even with extensive soils test. The importance of controlling seepage to protect groundwater dictates that careful evaluations be conducted before construction of ponds to determine the need for linings and the acceptable types.

The Minnesota Pollution Control Agency (8) initiated an intensive study to evaluate the effects of stabilization pond seepage from five municipal systems.

TABLE 4-1

REPORTED SEEPAGE RATES FROM POND SYSTEMS (7)[a]

Location	Geology of Pond Base	Initial Seepage Rate		Hydraulic Load	Seepage Rate as % of Hydraulic Load	Settling-in Period	Eventual Seepage Rate		Hydraulic Load	Seepage Rate as % of Hydraulic Load
		cm/d	$m^3/m^2/d$	$m^3/m^2/d$			cm/d	$m^3/m^2/d$	$m^3/m^2/d$	
Mojave, CA	Desert soil (sandy soil)	22.4	0.19	0.30	63	9 mo	0.9	0.007	0.36	2
Kearney, NB[b]	Sand and gravel	14.0	0.12	0.13	90	1 yr	1.5	0.013	0.04	29
Filer City, MI	Sandy soil	--	--	--	--	Average over 5 yr	0.9	0.007	0.009	84
Pretoria, SA[c]	Clay loam and shale	--	0.13	0.05	N/A	± 1 yr	0.8	0.006	0.05	13
Windhoek, SWA[d]										
Pond No. 5	Mica and schist	0.41	0.003	0.73	0.45	e	--	--	--	--
Pond No. 6	Mica and schist	0.43	0.003	1.11	0.32	e	--	--	--	--
Pond No. 7	Mica	0.04	0.0003	0.67	0.04	e	--	--	--	--
Pond No. 8	Mica and schist	0.15	0.0013	0.67	0.19	e	--	--	--	--
Pond No. 9	Mica and schist with side wall seepage to river	0.58	0.005	0.43	0.12	e	--	--	--	--

[a]Courtesy of Ann Arbor Science Publishers, Inc., Ann Arbor, MI.
[b]Evaporation and rainfall effects apparently not corrected for. Seepage losses also influenced at times by a high water table.
[c]Constructed in sandy soil for the express purpose of seeping away Paper Mill NSSC liquor.
[d]Constructed for the express purpose of water reclamation.
[e]Settling-in period is nine days after all ponds are in full operation.

The five communities were selected for study on the basis of geologic setting, age of the system, and past operating history of the pond. The selected ponds were representative of the major geomorphic regions in the state, and the age of the systems ranged from 3 to 17 years.

Estimates of seepage were calculated by two independent methods for each of the five pond systems. Water balances were calculated by taking the difference between the recorded inflows and outflows, and pond seepage was determined by conducting in-place field permeability tests of the bottom soils at each location. Good correlation was obtained with both techniques.

Field permeability tests indicated that the additional sealing from the sludge blanket was insignificant in locations where impermeable soils were used in the construction process. In the case of more permeable soils, it appeared that the sludge blanket reduced the permeability of the bottom soils from an initial level of 10^{-4} or 10^{-5} cm/sec to the order of 10^{-6} cm/sec. At all five systems evaluated, the stabilization pond was in contact with the local groundwater table. Local groundwater fluctuations had a significant impact on seepage rates. Reduced groundwater gradient resulted in a reduction of seepage losses at three of the sites. Contact with groundwater possibly explains the reduction in seepage rates in many ponds; in the past this reduction in seepage rates has been attributed totally to a sludge buildup. In an area underlain by permeable material where little groundwater mounding occurs, there is probably little influence from the water table on seepage rates. The buildup of sludge on the bottom of a pond appears to improve the quality of the seepage water leaving the pond. Sludge accumulation apparently increases the cation exchange capacity of the bottom of the pond.

Groundwater samples obtained from monitoring wells did not show any appreciable increases in nitrogen, phosphorus, or fecal coliform over the background levels after 17 years of operation. The seepage from the ponds did show an increase in soluble salts as great as 20 times over background levels. Concentrations of 25 mg/l to 527 mg/l of chloride were observed.

A comparison of observed seepage rates for various types of liner material is presented in Table 4-2 (4). If an impermeable liner is required, it appears that one of the synthetic materials must be used.

4.3.3 Natural and Chemical Treatment Sealing

The most interesting and complex techniques of pond sealing, either separately or in combination, are natural pond sealing and chemical treatment sealing (5)(9).

Natural sealing of ponds has been found to occur from three mechanisms: (1) physical clogging of soil pores by settled solids, (2) chemical clogging of soil pores by ionic exchange, and (3) biological and organic clogging caused

TABLE 4-2

SEEPAGE RATES FOR VARIOUS LINERS[a] (4)[b]

Liner Material	Thickness cm	Minimum Expected Seepage Rate at 6 m of Water Depth After 1 Yr of Service cm/d
Open sand and gravel		244
Loose earth		122
Loose earth plus chemical treatment*		30.5
Loose earth plus bentonite*		25.4
Earth in cut		30.5
Soil cement (continuously wetted)	10.2	10.2
Gunite	3.8	7.6
Asphalt concrete	10.2	3.8
Unreinforced concrete	10.2	3.8
Compacted earth	91	0.76
Exposed prefabricated asphalt panels	1.3	0.08
Exposed synthetic membranes	0.11	0.003

[a]The data are based on actual installation experience. The chemical and bentonite (*) treatments depend on pretreatment seepage rates, and in the table loose earth values are assumed.
[b]Courtesy of John Wiley & Sons, Inc., New York, NY.

by microbial growth at the pond lining. The dominant mechanism of the three depends on the characteristics of the wastewater being treated. Chemical treatment changes the nature of the bottom soil to ensure sealing.

Infiltration characteristics of anaerobic ponds were studied in New Zealand (10). Certain soil additives were employed (bentonite, sodium carbonate, sodium triphosphate) in 12 pilot ponds with varying water depth, soil type, and compacted bottom soil thickness. It was found that chemical sealing was effective for soils with a minimum clay content of 8 percent and a silt content of 10 percent. Effectiveness increased with clay and silt content.

Four different soil columns were placed at the bottom of an animal wastewater pond to study physical and chemical properties of soil and sealing of ponds (11). It was discovered that the initial sealing which occurred at the top 5 cm (2 in) of the soil columns was caused by the trapping of suspended matter in the soil pores. This was followed by a secondary mechanism of microbial growth that completely sealed off the soil from water movement.

A similar study performed in Arizona (12) also found this double mechanism of physical and biological sealing. Physical sealing of the pond was enhanced by the use of an organic polymer united with bentonite clay. This additive could have been applied with the pond full or empty, although it was more effective when the pond was empty.

An experiment was performed in South Dakota (13) in an effort to relate the sodium adsorption ratio (SAR) of the in situ soil to the sealing mechanism of stabilization ponds. No definite quantitative conclusions were formed. The general observation was made that the equilibrium permeability ratio decreases by a factor of 10 as SAR varies from 10 to 80. For 7 out of 10 soil samples, the following were concluded: (1) SAR did affect permeability of soils studied; (2) as the SAR increased, the probability that the pond would seal naturally also increased; and (3) soils with higher liquid limits[a] would probably be less affected by the SAR.

Polymeric sealants have been used to seal both filled and unfilled ponds (14). Unfilled ponds have been sealed by admixing a blend of bentonite and the polymer directly into the soil lining. Filled ponds have been sealed by spraying the fluid surface with alternate slurries of the polymer and bentonite. It has been recommended that the spraying take place in three subsequent layers: (1) polymer, (2) bentonite, and (3) polymer. The efficiency of the sealant has been found to be significantly affected by the characteristics of the impounded water. Most importantly, calcium ions in the water exchange with sodium ions in the bentonite and cause failure of the compacted bentonite linings.

Davis et al. (15) found that for liquid dairy waste the biological clogging mechanism predominated. In a San Diego County study site located on sandy loam, the infiltration rate of a virgin pond was measured. A clean water infiltration rate for the pond was 122 cm/d (48 in/d). After two weeks of manure water addition, infiltration averaged 5.8 cm/d (2.3 in/d); after four months, 0.5 cm/d (0.2 in/d).

A study performed in southern California (16) indicated similar results. After waste material was placed in the unlined pond in an alluvial silty soil, the seepage rate was reduced. The initial 11.2 cm/d (4.4 in/d) seepage rate dropped to 0.56 cm/d (0.22 in/d) after three months, and to 0.30 cm/d (0.12 in/d) after six months.

4.3.4 Design and Construction Practice

4.3.4.1 Lining Materials

Presentation of recommended pond sealing design and construction procedures is divided into two categories: (1) bentonite, asphalt, and soil cement

[a] The liquid limit is defined as the water content of a soil (expressed in percent dry weight) having a consistency such that two sections of a soil cake, placed in a cup and separated by a groove, barely touch but do not flow together under the impact of several sharp blows.

liners, and (2) thin membrane liners. This division was selected because of the major differences between the application techniques. There is some similarity between the application of asphalt panels and the elastomer liners and of necessity there will be some repetition in these two discussions. A partial listing of the trade names and sources of common lining materials is presented in Tables 4-3 and 4-4.

Regardless of the type of material selected as a liner there are many common design, specification, and construction practices. A summary of the common effective design practices in cut-and-fill reservoirs is given in Table 4-5. Most of these practices are commonsense items and would appear to not require mentioning. Unfortunately, experience has shown these items to be the most commonly ignored.

a. Bentonite, Asphalt, and Soil Cement

The application of bentonite, asphalt, and soil cement as lining materials for reservoirs and wastewater ponds has a long history (4). The following summary includes consideration of the method of using the materials, resultant costs, evaluations of durability, and effectiveness in limiting seepage. The cost analysis is somewhat arbitrary, since this cost depends primarily on the availability of the materials. A summary of state standards developed or being developed to control the application of these types of materials is presented elsewhere (6).

Bentonite is a sodium-type montmorillonite clay, and exhibits a high degree of swelling, imperviousness, and low stability in the presence of water. Different ways in which bentonite may be used to line ponds are listed below.

1. A suspension of bentonite in water (with a bentonite concentration approximately 0.5 percent of the water weight) is placed over the area to be lined, and the bentonite settles to the soil surface forming a thin blanket.

2. The same procedure as (1), except frequent harrowing of the surface produces a uniform soil bentonite mixture on the surface of the soil. The amount of bentonite used in this procedure is approximately 4.5 kg/m^2 (1 lb/ft^2) of soil.

3. A gravel bed approximately 15 cm (6 in) deep is first prepared and the bentonite application performed as in (1). The bentonite will settle through the gravel layer and seal the void spaces.

4. Bentonite is spread as a membrane 2.5 to 5 cm (1 to 2 in) thick and covered with a 20 to 30 cm (8 to 12 in) blanket of earth and gravel to protect the membrane. A mixture of earth and gravel is more satisfactory than soil alone, because of the stability factor and resistance to erosion.

TABLE 4-3

TRADE NAMES OF COMMON LINING MATERIALS (4)[a]

Trade Name	Product Description	Manufacturer
Aqua Sav	Butyl rubber	Plymouth Rubber Canton, MA
Armor last	Reinforced neoprene and Hypalon	Cooley, Inc. Pawtucket, RI
Armorshell	PVC-nylon laminates	Cooley, Inc. Pawtucket, RI
Armortite	PVC coated fabrics	Cooley, Inc. Pawtucket, RI
Arrowhead	Bentonite	Dresser Minerals Houston, TX
Biostate Liner	Biologically stable PVC	Goodyear Tire & Rubber Co. Akron, OH
Careymat	Prefabricated asphalt panels	Phillip Carey Co. Cincinnati, OH
CPE (resin)	Chlorinated PE resin	Dow Chemical Co. Midland, MI
Coverlight	Reinforced butyl and Hypalon	Reeves Brothers, Inc. New York, NY
Driliner	Butyl rubber	Goodyear Tire & Rubber Co. Akron, OH
EPDM (resin)	Ethylene propylene diene monomer resins	U.S. Rubber Co. New York, NY
Flexseal	Hypalon and reinforced Hypalon	B. F. Goodrich Co. Akron, OH
Geon (resin)	PVC resin	B. F. Goodrich Co. Akron, OH
Griffolyn 45	Reinforced Hypalon	Griffolyn Co., Inc. Houston, TX
Griffolyn E	Reinforced PVC	Griffolyn Co., Inc. Houston, TX

TABLE 4-3 (continued)

Trade Name	Product Description	Manufacturer
Griffolyn V	Reinforced PVC, oil resistant	Griffolyn Co., Inc. Houston, TX
Gundline	High density polyethylene (HDPE)	Gundle Lining Systems, Inc. Houston, TX
Hydroliner	Butyl rubber	Goodyear Tire & Rubber Co. Akron, OH
Hydromat	Prefabricated asphalt panels	W. R. Meadows, Inc. Elgin, IL
Hypalon (resin)	Chlorosulfonated PE resin	E. I. Du Pont Co. Wilmington, DE
Ibex	Bentonite	Chas. Pfizer & Co. New York, NY
Koroseal	PVC films	B. G. Goodrich Co. Akron, OH
Kreene	PVC films	Union Carbide & Chemical Co. New York, NY
Meadowmat	Prefabricated asphalt panels with PVC core	W. R. Meadows, Inc. Elgin, IL
National Baroid	Bentonite	National Lead Co. Houston, TX
Nordel (resin)	Ethylene propylene diene monomer resin	E. I. Du Pont Co. Wilmington, DE
Panelcraft	Prefabricated aspahlt panels	Envoy-APOC Long Beach, CA
Paraqual	EPDM and butyl	Aldan Rubber Co. Philadelphia, PA
Petromat	Polypropylene woven fabric (base fabric-spray linings)	Phillips Petroleum Co. Bartlesville, OK
Pliobond	PVC adhesive	Goodyear Tire & Rubber Co. Akron, OH
Polyliner	PVC-CPE, alloy film	Goodyear Tire & Rubber Co. Akron, OH
Red Top	Bentonite	Wilbur Ellis Co. Fresno, CA

TABLE 4-3 (continued)

Trade Name	Product Description	Manufacturer
Royal Seal	EPDM and butyl	U.S. Rubber Co. Mishawaka, IN
SS-13	Waterborne dispersion	Lauratan Corp. Anaheim, CA
Sure Seal	Butyl, EPDM, neoprene, and Hypalon, plain and reinforced	Carlisle Corp. Carlisle, PA
Vinaliner	PVC	Goodyear Tire & Rubber Co. Akron, OH
Vinyl Clad	PVC, reinforced	Sun Chemical Co. Paterson, NJ
Visqueen	PE resin	Ethyl Corp. Baton Rouge, LA
Volclay	Bentonite	American Colloid Co. Skokie, IL
Water Seal	Bentonite	Wyo-Ben Products Billings, MT

[a]Courtesy of John Wiley & Sons, Inc., New York, NY.

TABLE 4-4

SOURCES OF COMMON LINING MATERIALS (4)[a]

Material	Manufacturer	Location
Bentonite	American Colloid Co.	Skokie, IL
	Archer-Daniels-Midland	Minneapolis, MN
	Ashland Chemical Co.	Cleveland, OH
	Chas. Pfizer & Co.	New York, NY
	Dresser Minerals	Houston, TX
	National Lead Co.	Houston, TX
	Wilbur Ellis Co.	Fresno, CA
	Wyo-Ben Products, Inc.	Billings, MT
Butyl and EPDM	Carlisle Corp.	Carlisle, PA
	Goodyear Tire & Rubber Co.	Akron, OH
Butyl and EPDM, reinforced	Aldan Rubber Co.	Philadelphia, PA
	Carlisle Corp.	Carlisle, PA
	Plymouth Rubber Co.	Canton, MA
	Reeves Brothers, Inc.	New York, NY
CPE, reinforced	Goodyear Tire & Rubber Co.	Akron, OH
Hypalon	Burke Rubber Co.	San Jose, CA
	B. F. Goodrich Co.	Akron, OH
Hypalon, reinforced	Burke Rubber Co.	San Jose, CA
	Carlisle Corp.	Carlisle, PA
	B. F. Goodrich Co.	Akron, OH
	Plymouth Rubber Co.	Canton, MA
	J. P. Stevens Co.	New York, NY
EPDM	See "Butyl and EPDM"	
EPDM, reinforced	See "Butyl and EPDM, reinforced"	
Neoprene	Carlisle Corp.	Carlisle, PA
	Firestone Tire & Rubber Co.	Akron, OH
	B. F. Goodrich Co.	Akron, OH
	Goodyear Tire & Rubber Co.	Akron, OH
Neoprene, reinforced	Carlisle Corp.	Carlisle, PA
	B. F. Goodrich Co.	Akron, OH
	Firestone Tire & Rubber Co.	Akron, OH
	Plymouth Rubber Co.	Canton, MA
	Reeves Brothers, Inc.	New York, NY

TABLE 4-4 (continued)

Material	Manufacturer	Location
PE	Monsanto Chemical Co.	St. Louis, Mo.
	Union Carbide, Inc.	New York, NY
	Ethyl Corp.	Baton Rouge, LA
PE, high quality	Gundle Lining Systems, Inc.	Houston, TX
PE, reinforced	Griffolyn Co., Inc.	Houston, TX
PVC	Firestone Tire & Rubber Co.	Akron, OH
	B. F. Goodrich Co.	Akron, OH
	Goodyear Tire & Rubber Co.	Akron, OH
	Pantasote Co.	New York, NY
	Stauffer Chemical Co.	New York, NY
	Union Carbide, Inc.	New York, NY
PVC, reinforced	Firestone Tire & Rubber Co.	Akron, OH
	B. F. Goodrich Co.	Akron, OH
	Goodyear Tire & Rubber Co.	Akron, OH
	Reeves Brothers, Inc.	New York, NY
	Cooley, Inc.	Pawtucket, RI
	Sun Chemical Co.	Paterson, NJ
Prefabricated asphalt panels	Envoy-APOC	Long Beach, CA
	Gulf Seal, Inc.	Houston, TX
	W. R. Meadows, Inc.	Elgin, IL
	Phillip Carey Co.	Cincinnati, OH
3110	E. I. Du Pont Co.	Louisville, KY

[a]Courtesy of John Wiley & Sons, Inc., New York, NY.

TABLE 4-5

SUMMARY OF EFFECTIVE DESIGN PRACTICES FOR PLACING LINING IN CUT-AND-FILL RESERVOIRS

1. Lining must be placed in a stable structure.

2. Facility design and inspection should be the responsibility of professionals with backgrounds in liner applications and experience in geotechnical engineering.

3. A continuous underdrain to operate at atmospheric pressure is recommended.

4. A leakage tolerance should be included in the specifications. The East Bay Water Company of Oakland, CA (East Bay Municipal Utility District) developed the following formula for leakage tolerance which can be modified by inserting more stringent factors in the denominator, i.e., 100, 200, etc. The equation is empirical and its use must be based on experience.

$$Q = \frac{A\sqrt{H}}{80}$$

where,

Q = maximum permissible leakage tolerance, gallons/minute
A = lining area, 1000 ft^2
H = maximum water depth, ft

5. Continuous, thin, impermeable-type linings should be placed on a smooth surface of concrete, earth, Gunite, or asphalt concrete.

6. Except for asphalt panels all field joints should be made perpendicular to the toe of the slope. Joints of Hypalon formulations and 3110 materials can run in any direction, but generally joints run perpendicular to the toe of the slope.

7. Formal or informal anchors may be used at the top of the slope. See details in Figures 4-2 to 4-6.

8. Inlet and outlet structures must be sealed properly. See details in Figures 4-7 to 4-11.

9. All lining punctures and cracks in the support structure should be sealed. See details in Figures 4-12 and 4-13.

10. Emergency discharge quick-release devices should be provided in large reservoirs [$7.6(10^4)$ to $1.2(10^5)$ m^3].

11. Wind problems with exposed thin membrane liners can be controlled by installing vents built into the lining. See details in Figure 4-14.

12. Adequate protective fencing must be installed to control vandalism.

5. Bentonite is mixed with sand at aproximately one to eight volume ratio. The mixture is placed in a layer approximately 5 to 10 cm (2 to 4 in) in thickness on the reservoir bottom and covered with a protective cover of sand or soil. This method takes about 13.5 kg/m^2 (3 lb/ft^2) of bentonite (17).

In methods (4) and (5) above, certain construction practices are recommended. They are as follows:

1. The section must be overexcavated [30 cm (12 in)] with drag lines or graders.

2. Side slopes should not be steeper than two horizontal to one vertical.

3. Subgrade surface should be dragged to remove large rocks and sharp angles. Normally two passes with adequate equipment are sufficient to smooth the subgrade.

4. Subgrade should be rolled with a smooth steel roller.

5. The subgrade should be sprinkled to eliminate dust problems.

6. The membrane of bentonite or soil bentonite should then be placed.

7. The protective cover should contain sand and small gravel, in addition to cohesive, fine grained material so that it will be erosion resistant and stable.

The performance of bentonite linings is greatly affected by the quality of the bentonite. Some natural bentonite deposits may contain quantities of sand, silt, and clay impurities. Wyoming-type bentonite, which is a high swelling sodium montmorillonite clay, has been found to be very satisfactory. Fine ground bentonite is generally more suitable for the lining than pit run bentonite. If the bentonite is finer than a No. 30 sieve, it may be used without specifying size gradation. But if the material is coarser than the No. 30 sieve, it should be well graded. Bentonite should usually contain a moisture content of less than 20 percent. This is especially important for thin membranes. Some disturbance, and possibly cracking of the membrane, may take place during the first year after construction due to settlement of the subgrade upon saturation. A proper maintenance program, especially at the end of the first year, is necessary (16).

Bentonite linings may be effective if the sodium bentonite used has an adequate amount of exchangeable sodium. Deterioration of the linings has been observed to occur in cases where magnesium or calcium has replaced sodium as adsorbed ions. A thin layer, less than 15 cm (6 in), of bentonite on the soil surface tends to crack if allowed to dry. Because of this, a bentonite soil mixture with a cover of fine grained soil on top, or a thicker bentonite layer, is usually placed (18). Surface bentonite cannot be expected to be effective longer than two to four years. A buried bentonite blanket may last from 8 to 12 years.

The quality of the bentonite used is a primary consideration in the success of bentonite membranes. Poor quality bentonite deteriorates rapidly in the presence of hard water, and it also tends to erode in the presence of currents or waves. Bentonite linings must often be placed by hand and this is a costly procedure in areas of high labor costs.

Seepage losses through buried bentonite blankets are approximately 0.2 to 0.25 $m^3/m^2/d$ (0.7 to 0.85 $ft^3/ft^2/d$). This figure is for thin blankets and represents about a 60 percent improvement over ponds with no lining.

Asphalt linings may be buried on the surface and may be composed of asphalt or a prefabricated asphalt. Some possibilities are as follows:

1. An asphalt membrane is produced by spraying asphalt at high temperatures. This lining may be either on the surface or buried. A large amount of special equipment is needed for installation. Useful lives of 18 years or greater have been observed when these membranes are carefully applied and covered with an adequate layer of fine grained soil.

2. Asphaltic Membrane Macadam. This is similar to the asphaltic membrane, but it is covered with a thin layer of gravel, penetrated with hot blown asphalt cement.

3. Buried Asphaltic Membrane. This is similar to (1), except a gravel-sand cover is applied over the asphaltic membrane. This cover is usually more expensive than cover in (2) and less effective in discouraging plant growth.

4. Built-up Linings. These include several different types of materials. One type could be a fiberglass matting, which is applied over a sprayed asphalt layer and then also sprayed or broomed with a sealed coat of asphalt or clay. A 280-g (10-oz) jute burlap has also been used as the interior layer between two hot sprayed asphalt layers. In this case the total asphalt application should be about 11.3 liters/m^2 (2.5 gal/yd^2). The prefabricated lining may be on the surface or buried. If buried, it could be covered with a layer of soil or, in some cases, a coating of Allox, which is a stabilized asphalt (19).

5. Prefabricated Linings. Prefabricated asphalt linings consist of a fiber or paper material coated with asphalt. This type of liner has been used as both exposed and covered with soil. Joints between the material have an asphaltic mastic to seal the joint. When the asphaltic material is covered, it is more effective and

durable. When it is exposed it should be coated with aluminized paint every three to four years to retard degradation. This is especially necessary above the water line. Joints also have to be maintained when not covered with fine grained soil. Prefabricated asphalt membrane lining is approximately 0.32 to 0.64 cm (0.13 to 0.25 in thick. It may be handled in much the same way as rolled roofing with lapped and cemented joints. Cover for this material is generally earth and gravel, although shot-crete and macadam have been utilized.

Installation procedures for prefabricated asphalt membrane linings and for buried asphalt linings are similar to those stated for buried bentonite linings. The preparation of the subgrade is important and it should be stable and adequately smooth for the lining.

Best results are obtained with soil cement when the soil mixed with the cement is sandy and well graded to a maximum size of about 2 cm (0.75 in).

Soil cement should not be placed in cold weather and it should be cured for about seven days after placing. Some variations of the soil cement lining are listed below.

1. Standard soil cement is compacted using a water content of the optimum moisture content[a] of the soil. The mixing process is best accomplished by traveling mixing machines and can be handled satisfactorily in slopes up to four to one. Standard soil cement may be on the surface or buried.

2. Plastic soil cement (surface or buried) is a mixture of soil and cement with a consistency comparable to that of Portland cement concrete. This is accomplished by adding a considerable amount of water. Plastic soil cement contains from three to six sacks of cement per cubic meter and is approximately 7.5 cm (3 in) thick.

3. Cement modified soil contains two to six percent volume of cement. This may be used with plastic fine grained soils. The treatment stabilizes the soil in sections subject to erosion. The lining is constructed by spreading cement on top of loose soil layers by a fertilizer-type spreader. The cement is then mixed with loose soil by a rotary traveling mixer and compacted with a sheeps foot roller. A seven-day curing period is necessary for a cement modified soil.

Soil cement has been used successfully in some cases in mild climates. Where wetting or drying is a factor, or if freezing-thawing cycles are present, the lining will deteriorate rapidly (20).

---

[a] Water content (expressed in percent dry weight) at which a given soil can be compacted to its maximum density by means of a standard method of compaction.

Linings of bentonite and asphalt are sometimes unsuitable in areas of high weed growth, since weeds and tree roots puncture the material readily (20).

Many lining failures occur as a result of rodent and crayfish holes in embankments. Asphalt membrane lining tends to decrease the damage, but, in some cases, harder surface linings are necessary to prevent water loss from embankment failures.

Linings of hot applied buried asphalt membrane provide one of the tightest linings available. These linings deteriorate less than other flexible membrane linings (20).

Asphalt linings composed of prefabricated buried materials are best for small jobs, since there is a minimum amount of special equipment and labor connected with installation. For larger jobs sprayed asphalt is more economical.

When fibers and filler are used in asphalt membranes, there is greater tendency to deteriorate when these fillers are composed of organic materials. Inorganic fibers are, therefore, more useful (20). Typical seepage volume through one buried asphalt membrane after 10 years of service was consistently 0.02 $m^3/m^2/d$ (0.08 $ft^3/ft^2/d$) (21).

Asphalt membrane linings can be constructed at any time of the year. It is usually convenient, because of low water levels in ponds, to use the late fall and winter seasons for installing linings. Fall and winter installation may dictate the use of the buried asphalt membrane lining (20).

Buried asphalt membranes usually perform satisfactorily for more than 15 years. When these linings fail, it is generally due to one or more of the following causes:

1. Placement of lining on unstable side slopes
2. Inadequate protection of the membrane
3. Weed growth
4. Surface runoff
5. Type of subgrade material
6. Cleaning operations
7. Scour of cover material
8. Membrane puncture

b. Thin Membrane Liners

Plastic and elastomeric membranes are popular in applications requiring essentially zero permeability. These materials are economical, resistant to most chemicals if selected and installed properly, available in large sheets simplifying installation, and essentially impermeable. As environmental standards continue to become more stringent, the application of plastic and elastomeric membranes as pond liners will increase because of the need to guarantee protection against seepage. This is particularly true for sealing ponds containing toxic wastewaters or the sealing of landfills containing toxic solids and sludges.

Typical standards being developed for the application of liners are presented elsewhere (6). A partial listing of the trade names, product description, and manufacturer of plastic and elastomer lining materials is presented in Tables 4-3 and 4-4.

The most difficult design problem encountered in liner application involves placing a liner in an existing pond. Effective design practices are essentially the same as those used in new systems, but additional care must be exercised in the evaluation of the existing structure and the required results. Lining materials must be selected so that compatibility is obtained. For example, a badly cracked concrete lining to be covered with a flexible synthetic material must be properly sealed and the flexible material placed in such a way that additional movement will not destroy the new liner. Sealing around existing columns, footings, etc., are other examples of items to be considered.

The following paragraphs are a condensation of the discussion by Kays (4) of effective design practices which have been summarized in Table 4-5. Emphasis is placed on the details describing the installation of plastic or elastomeric materials.

Formal and informal anchor systems are used at the top of the slope or dikes. Details of three types of formal anchors are presented in Figures 4-2 through 4-4. Recommended informal anchors are shown in Figures 4-5 and 4-6.

When the lining is pierced, seals can be made in two ways. The techniques illustrated in Figures 4-7 and 4-8 are commonly used, and a second technique utilizes a pipe boot which is sealed to the liner and clamped to the entering pipe as shown in Figure 4-9.

It is recommended that inlet-outlet pipes enter a reservoir through a structure such as that shown in Figure 4-10. A better seal can be produced when the liner is attached to the top of the structure. However, such an arrangement can result in solids accumulation, and direct free entry into a wastewater pond is better.

A drain near the outlet can be constructed as shown in Figure 4-11. Large reservoirs containing 7.5×10^4 to 1.0×10^5 m^3 (2.5 to 4.0 mgal) should be equipped to empty quickly in case of an emergency.

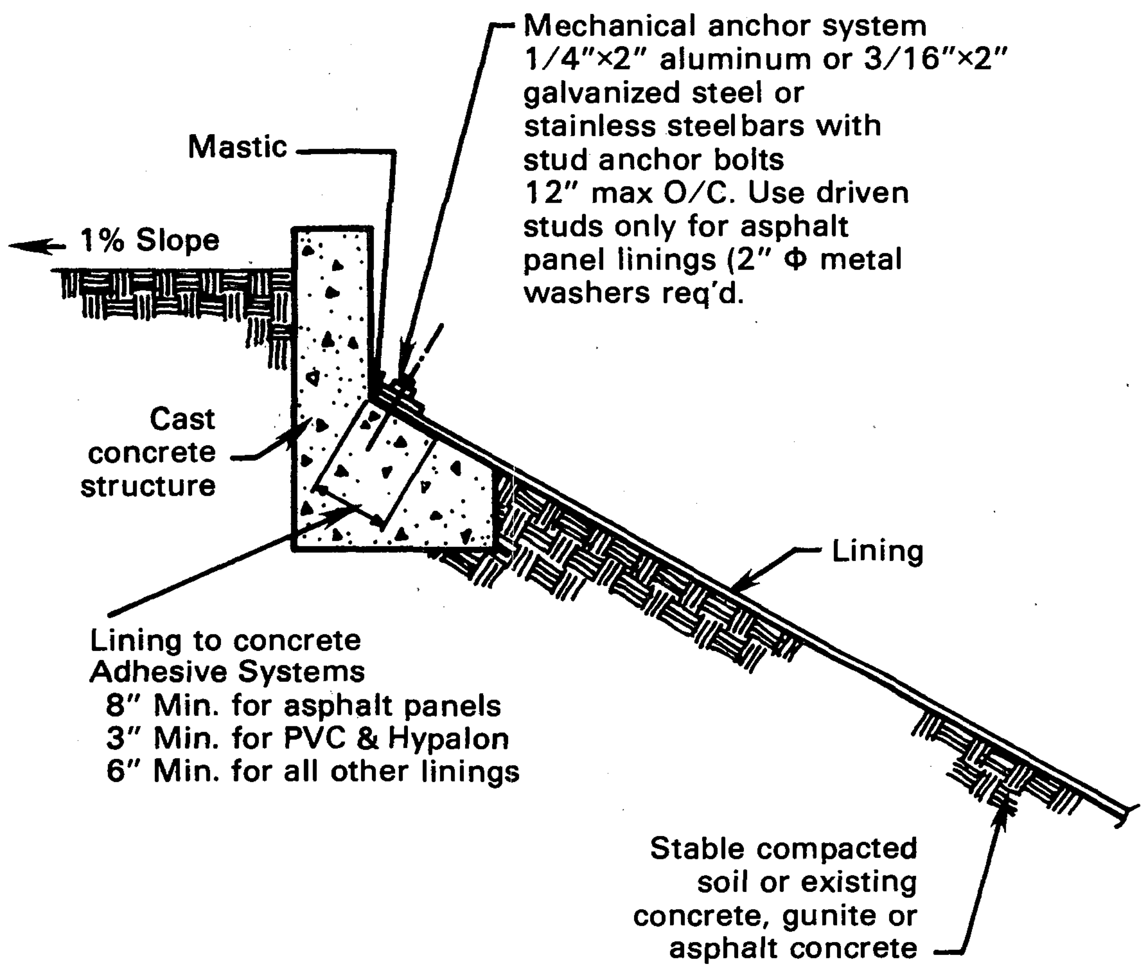
The structure supporting the liner must be smooth enough to prevent damage to the liner. Rocks, sharp protrusions, and other rough surfaces must be controlled. In areas with particularly rough surfaces, it may be necessary to add padding to protect the liner. Cracks can be repaired as shown in Figures 4-12 and 4-13.

Thin membrane liners may have problems with wind on the leeward slopes. Vents built into the lining control this problem and serve as an outlet for gases trapped beneath the liner (Figure 4-14).

FIGURE 4-2

TOP ANCHOR DETAIL--ALTERNATIVE 1, ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



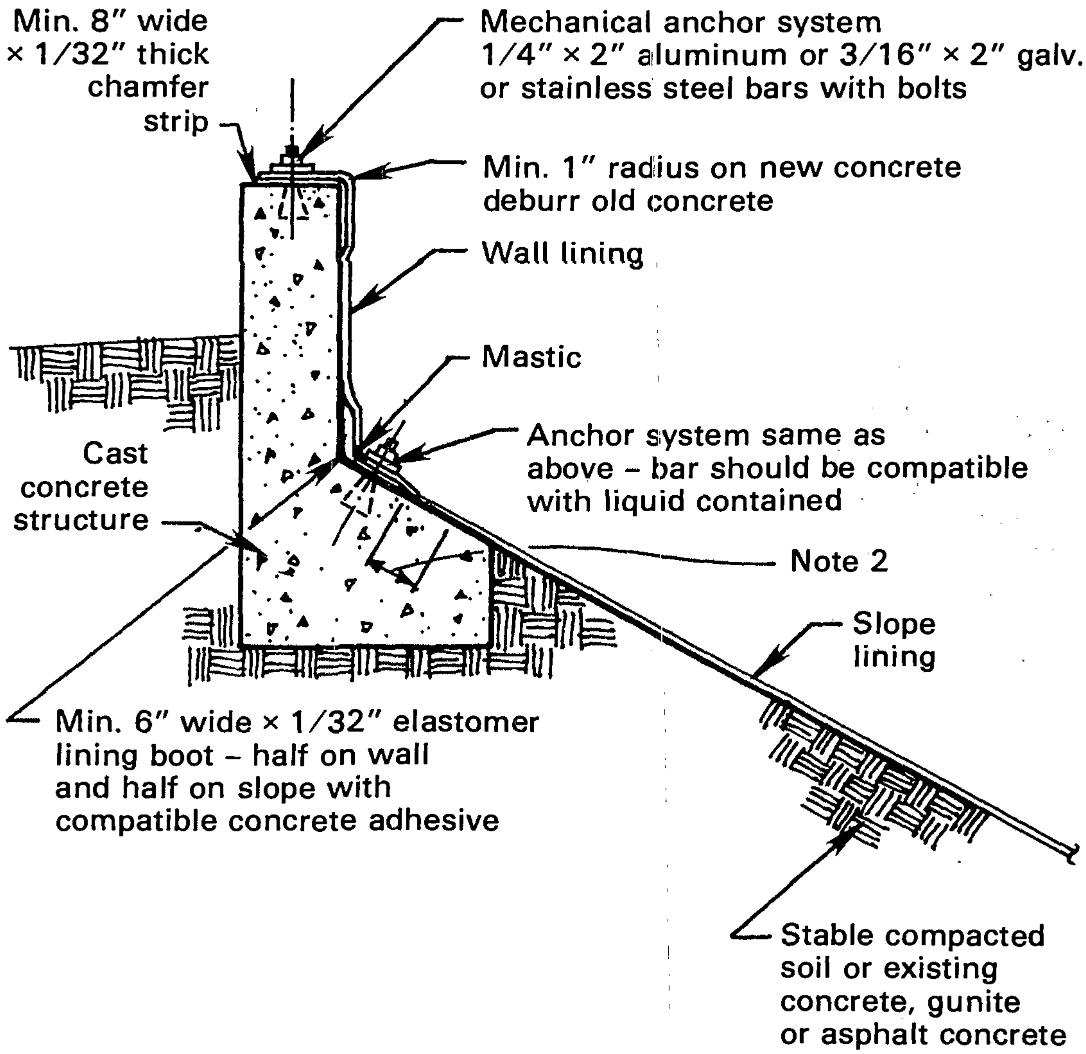
NOTE

1. Top of concrete should be smooth and free of all curing compounds.
2. Use min. 1/32" × 2" gasket (mat'l compatible with lining) between bar and lining, except no gasket required for asphalt panels or other linings thicker than .040".

FIGURE 4-3

TOP ANCHOR DETAIL--ALTERNATIVE 2, ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



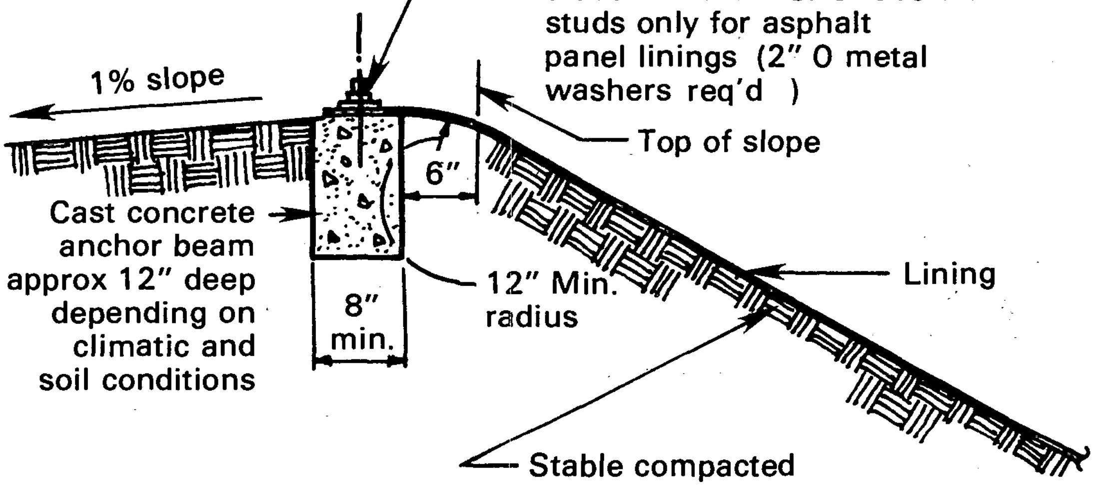
NOTE

1. All concrete at seals shall be smooth and free of all curing compounds.
2. Use compatible adhesive between slope lining and elastomer boot, and 3" min. width of compatible adhesive between slope lining and concrete.

FIGURE 4-4

TOP ANCHOR DETAIL--ALTERNATIVE 3, ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



NOTE

1. Top of concrete should be smooth and free of all curing compounds.
2. Use min. 1/32" × 2" gasket (mat'l compatible with lining) between bar 8 lining except no gasket required for asphalt panels or other linings thicker than .040".

FIGURE 4-5

TOP ANCHOR DETAIL--ALTERNATIVE 4, ALL LININGS EXCEPT ASPHALT PANELS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY

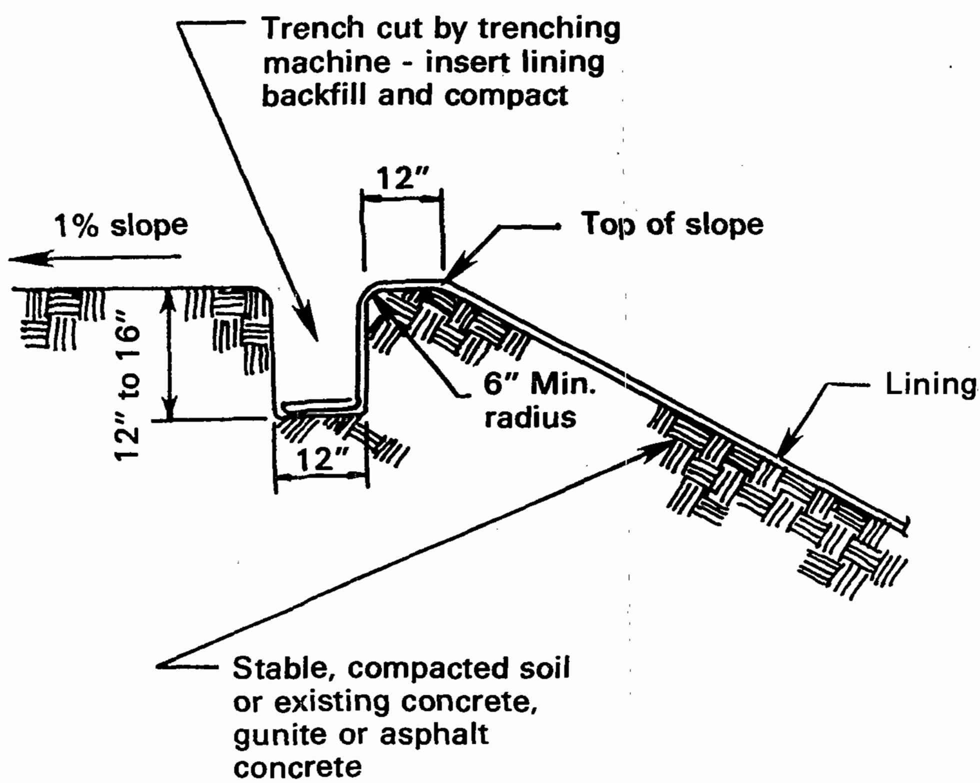


FIGURE 4-6

TOP ANCHOR DETAIL--ALTERNATIVE 5, ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY

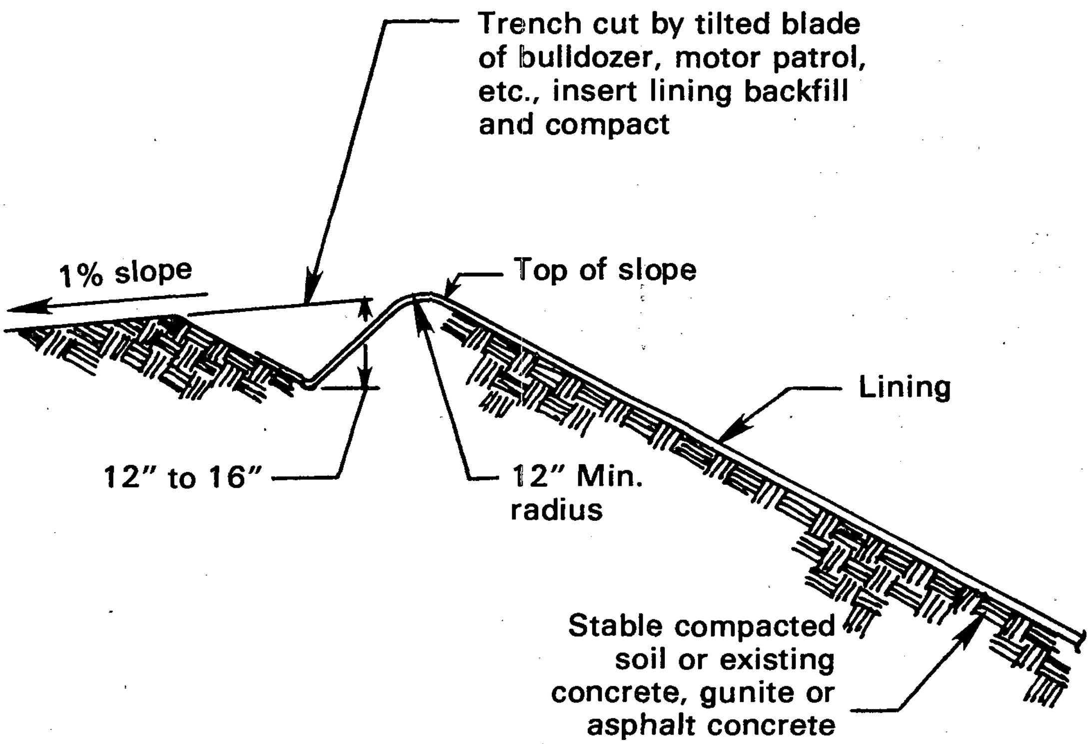
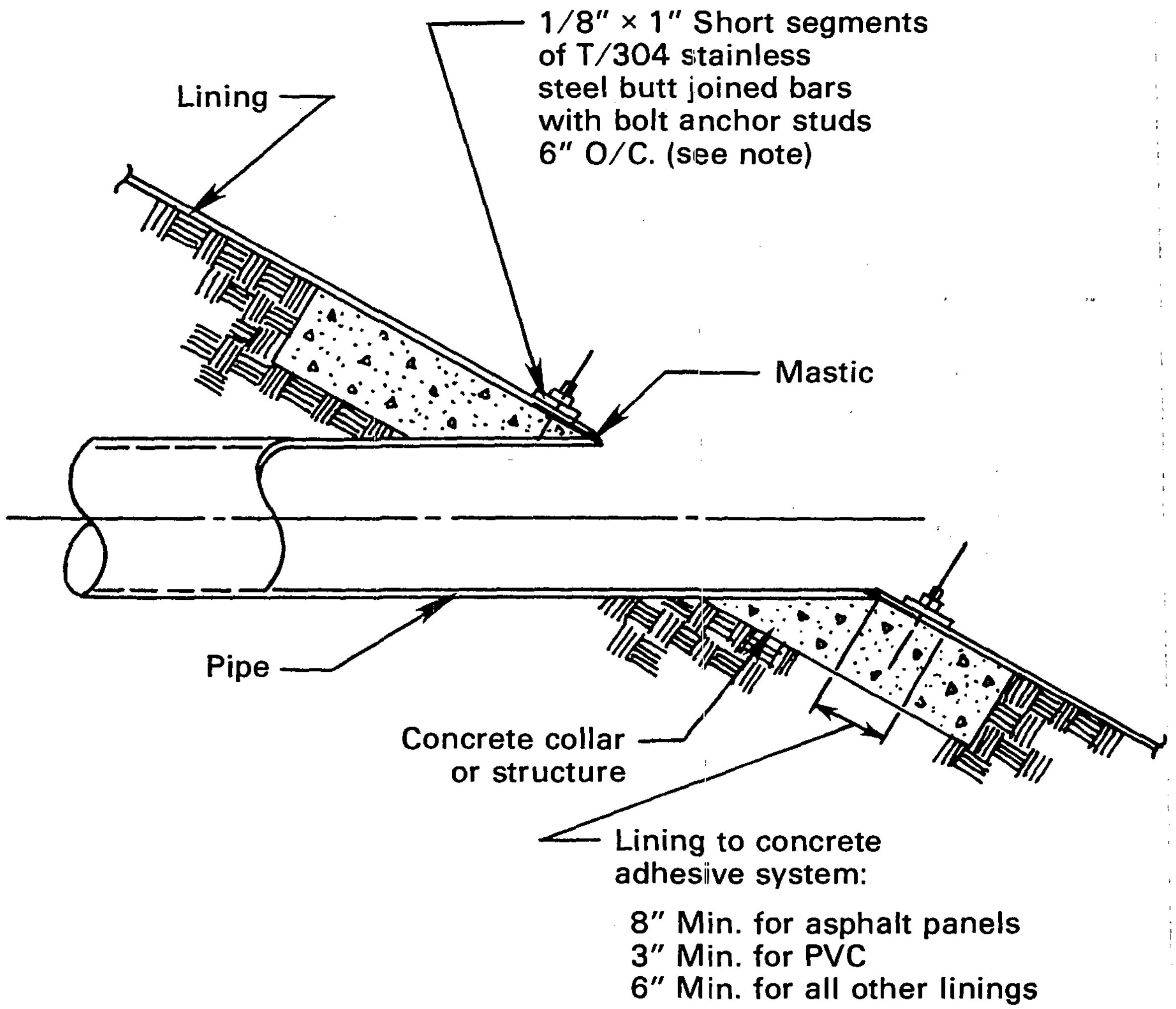


FIGURE 4-7

SEAL AT PIPES THROUGH SLOPE, ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



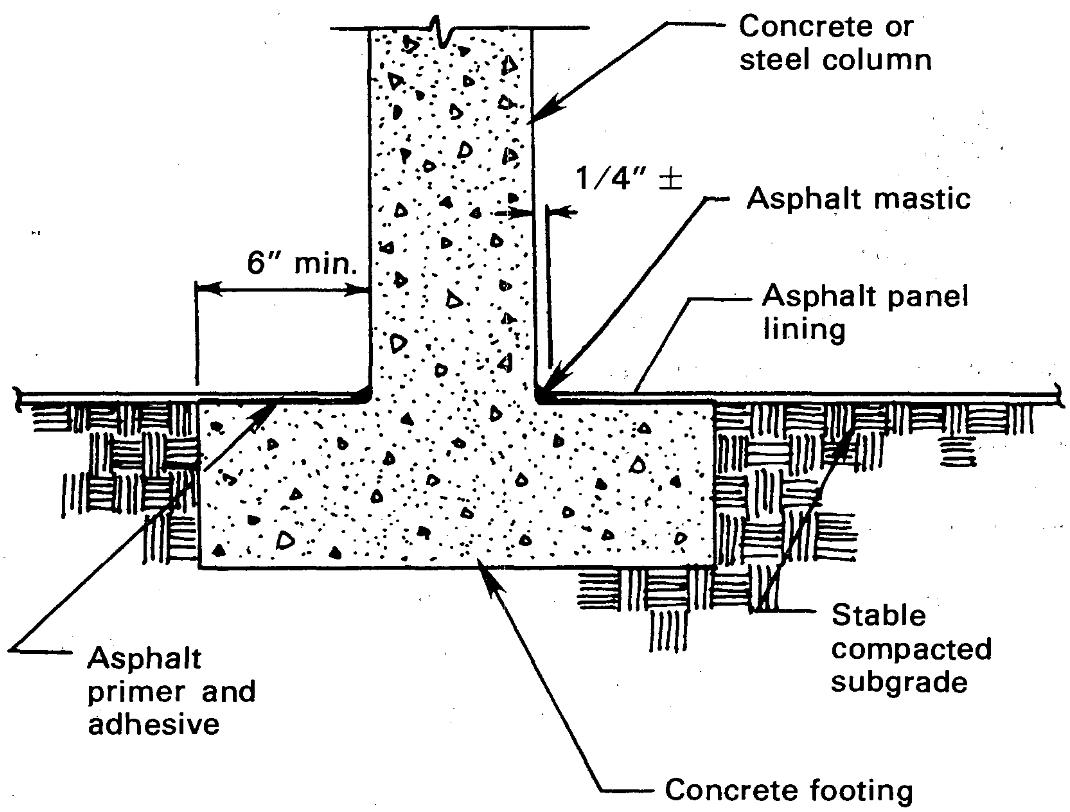
NOTE

For asphalt panel linings, percussion driven studs thru 2" min. dia. × 1/16" thick galvanized metal discs at 6" O/C encased in mastic may be substituted for anchor shown.

FIGURE 4-8

SEAL AT FLOOR COLUMNS--ASPHALT PANELS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



NOTE

Mechanical fasteners not required

FIGURE 4-9

PIPE BOOT DETAIL--ALL LININGS EXCEPT ASPHALT PANELS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY

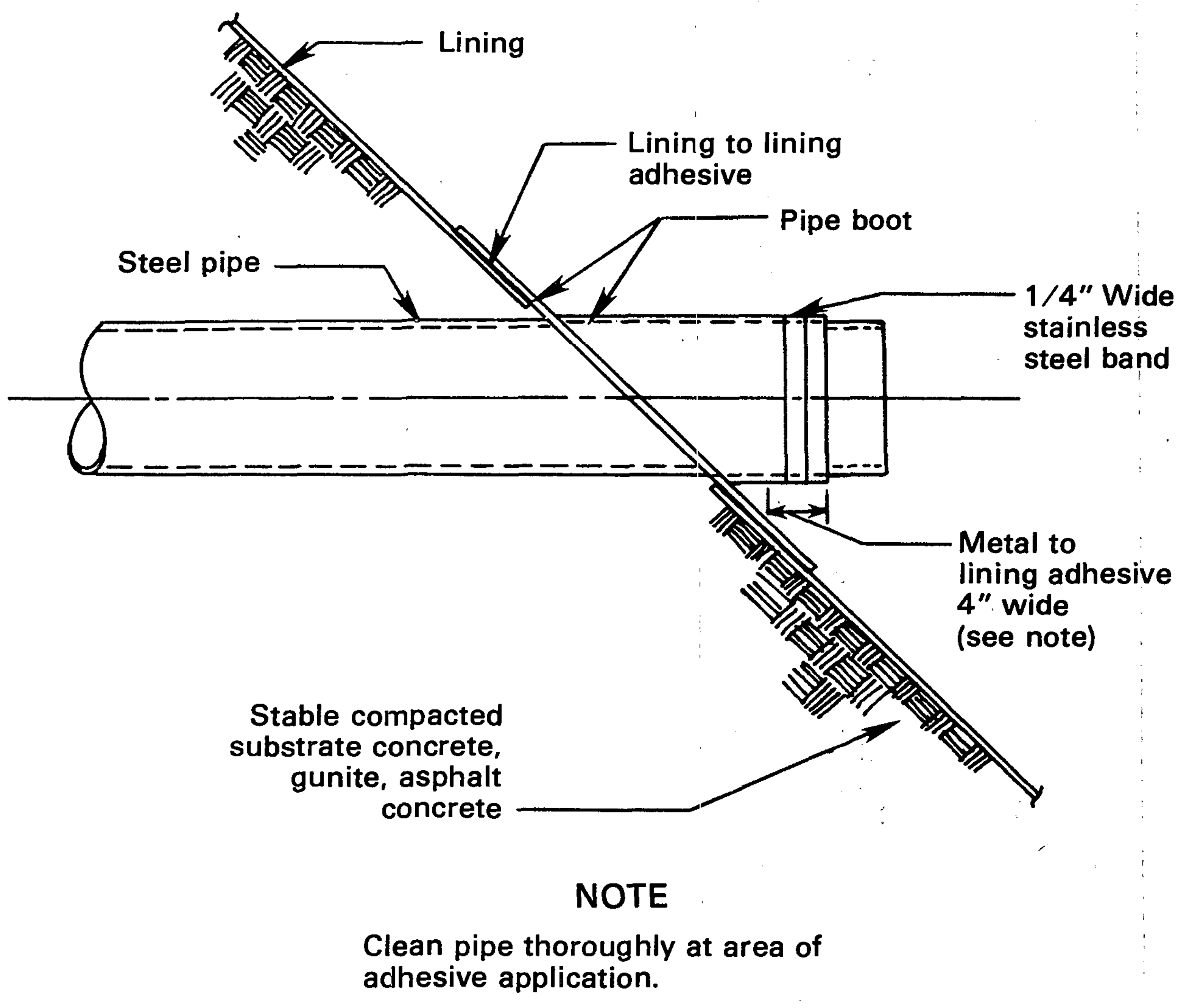
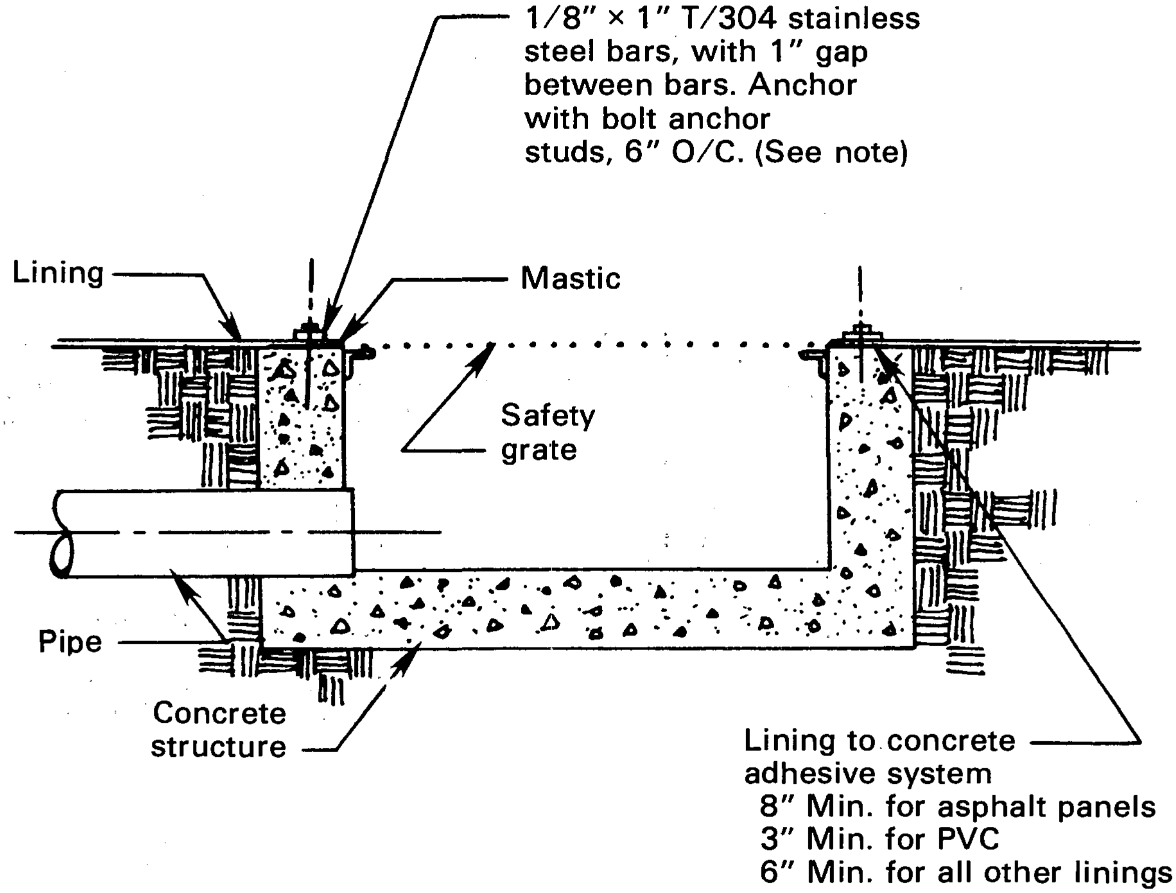


FIGURE 4-10

SEAL AT INLET-OUTLET STRUCTURE--ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



NOTE

With asphalt panel linings, percussion driven studs thru 1" min. dia. 1/16" thick galvanized metal discs at 6" O/C, encased in mastic may be substituted for anchor shown.

FIGURE 4-11

MUD DRAIN DETAIL--ALL LININGS (4)

Courtesy of John Wiley & Sons, Inc., New York, NY

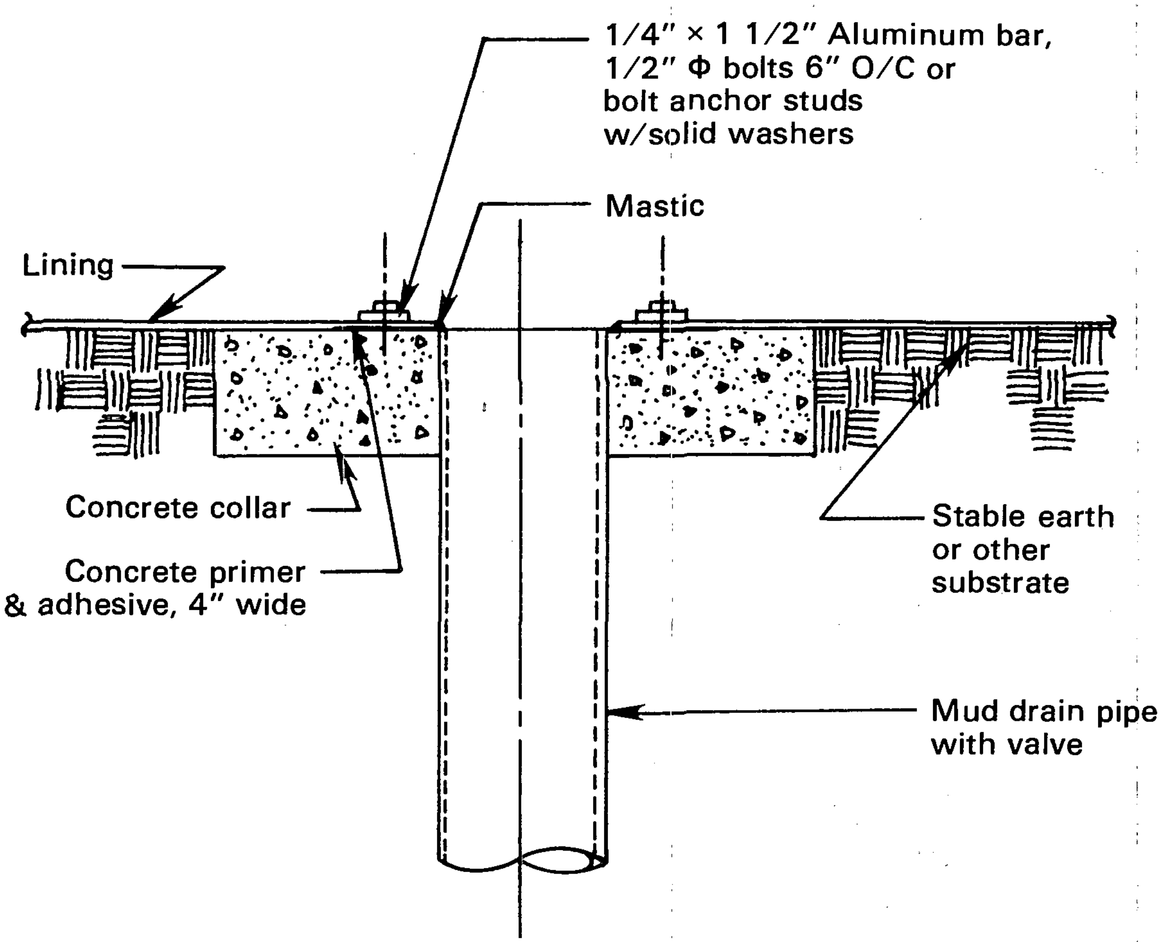
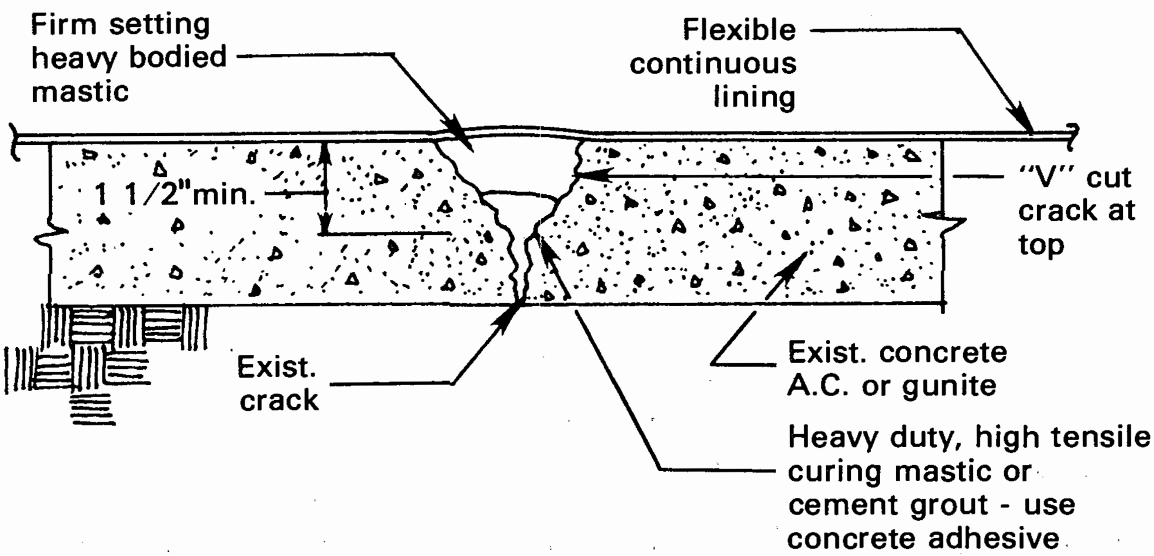


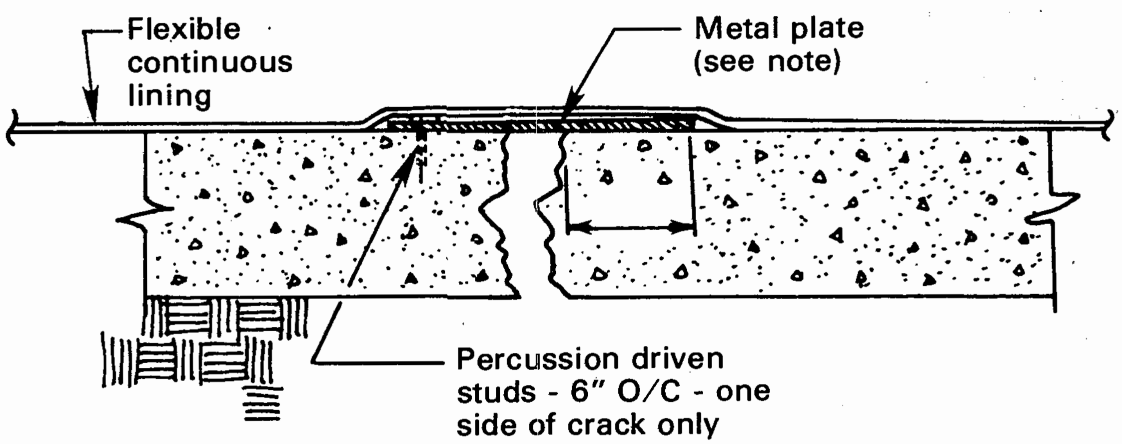
FIGURE 4-12

CRACK TREATMENT--ALTERNATIVES A AND B (4)

Courtesy of John Wiley & Sons, Inc., New York, NY



Alternative A



NOTE

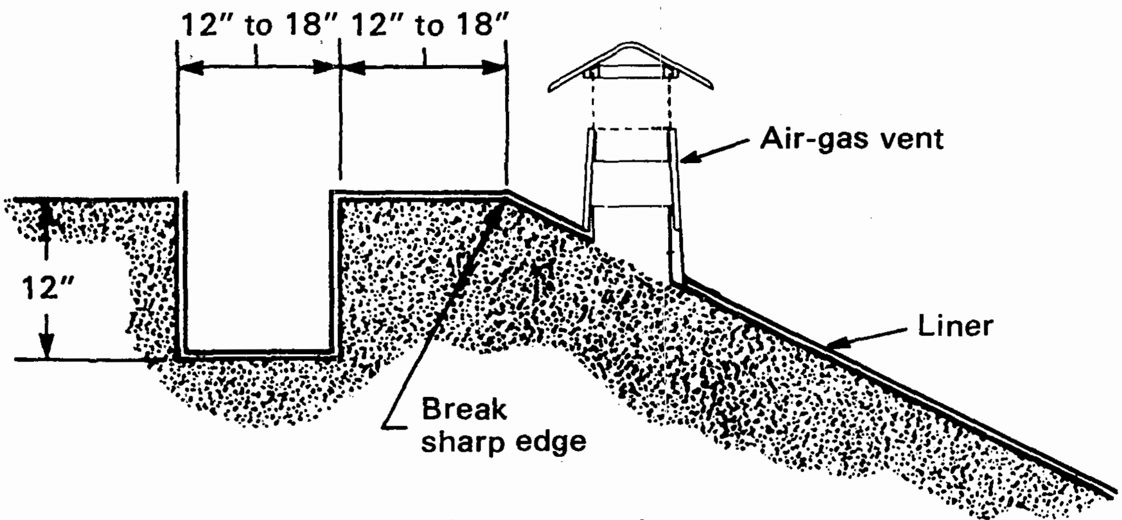
Metal plate must be able to span crack without buckling from weight of water bridging the crack. Copper 8 stainless steel are most common choices.

Alternative B

FIGURE 4-13

WIND AND GAS CONTROL

Courtesy of Burke Rubber Company, Burke Industries, San Jose, CA

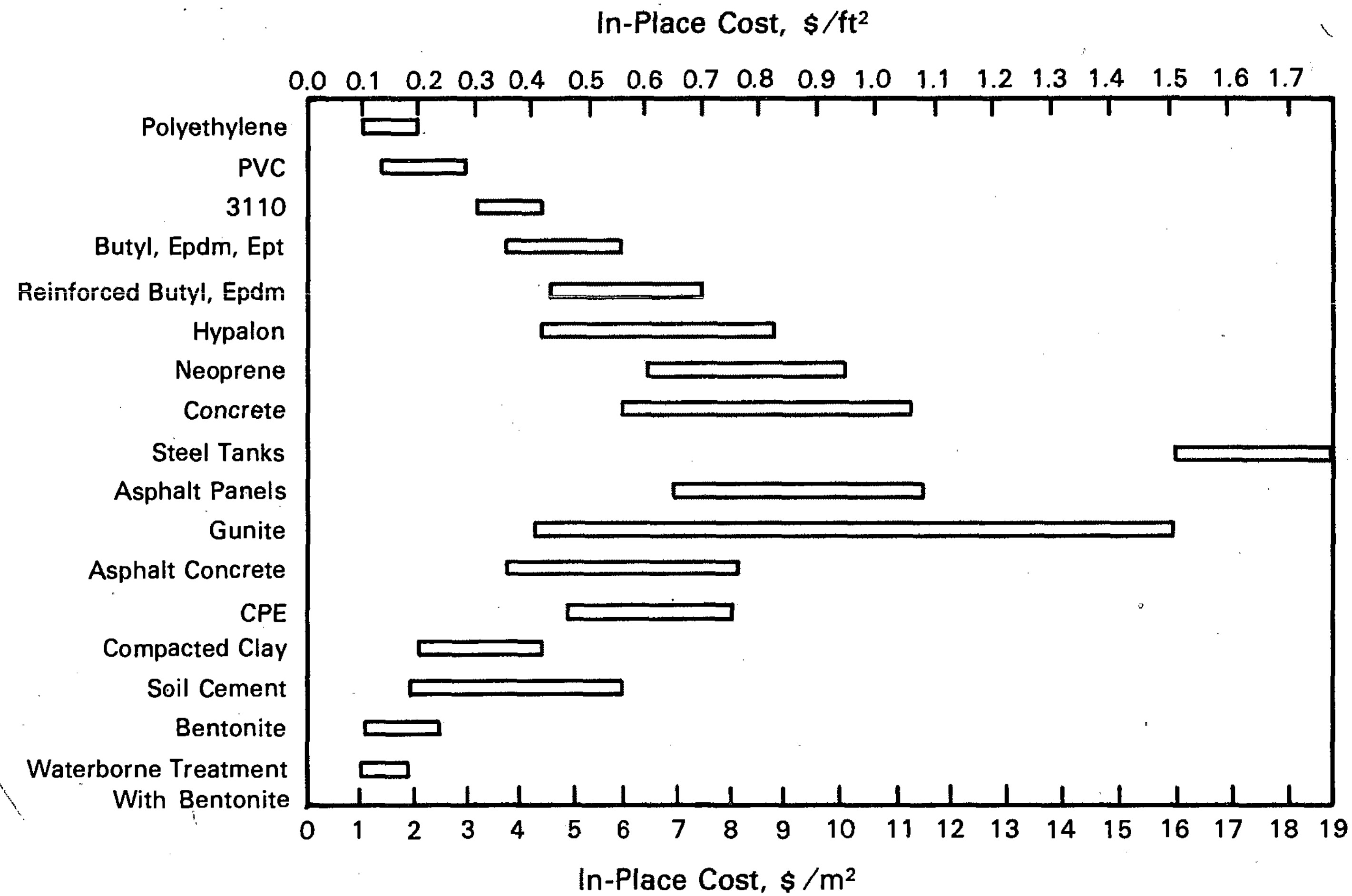


Anchor trench
&
Air-gas vent

FIGURE 4-14

COST COMPARISON FOR LININGS IN THE UNITED STATES (1977 U.S. $)

Courtesy of John Wiley & Sons, Inc., New York, NY



Protection of a thin membrane lining is essential, and Kays (4) recommends that a fence at least 2 m (6.6 ft) high be placed on the outside berm slope with the top of the fence below the top elevation of the dike to keep the membrane out of sight of vandals.

In addition to those manufacturers presented in Table 4-4, there are many firms specializing in the installation of lining materials. Most of the installation companies and the manufacturers publish specifications and installation instructions and design details. Most of the recommendations by the manufacturers and installers are similar, but there are differences worthy of consideration when designing a system requiring a liner. Consult the manufacturers for details.

New products continue to be developed, and with each new material the options available to designers continue to improve. The future should bring even more versatile and effective liners for seepage control. If care and common-sense are applied to the application of liners, the control of seepage pollution should become a minor problem in the future.

4.3.4.2 Failure Mechanisms

Kays (4) presented a classification of the principal failure mechanism observed in cut-and-fill reservoirs (Table 4-6). The list is extensive and case histories involving all of the categories are available; however, the most frequently observed failure mechanisms were the lack of integrity in the lining suppport structure and abuse of the liner.

4.3.4.3 Cost of Linings

The cost of linings for ponds are approximations and are estimated based on values at specific jobs (1978 U.S. $).

Bentonite linings cost approximately $1.10 to $2.20/m^2 ($0.10 to $0.20/ft^2) when applied on the surface. The greater cost will occur for harrowed blankets. Buried blankets cost approximately $3.10/m^2 ($0.30/ft^2).

The average cost of buried asphalt membrane linings with adequate cover is about $4.20/m^2 ($0.40/ft^2).

Prefabricated asphalt materials are generally cheaper than buried asphalt membrane linings if the prefabricated material can be obtained for less than $1.10/m^2 ($0.10/ft^2).

Figure 4-14 presents a cost comparison for the various types of liners used in the United States (4).

TABLE 4-6

CLASSIFICATION OF THE PRINCIPAL FAILURE MECHANISMS FOR CUT-AND-FILL RESERVOIRS[a]

Supporting structure problems	Lining problems
Underdrains	Mechanical difficulties
Substrate	Field seams
Compaction	Fish mouths[b]
Texture	Structure seals
Voids	Bridging
Subsidence	Porosity
Holes and cracks	Holes
Groundwater	Pinholes
Expansive clays	Tear strength
Gassing	Tensile strength
Sloughing	Extrusion and extension
Slope anchor stability	Rodents, other animals, and birds
Mud	Insects
Frozen ground and ice	Weed growths
Appurtenances	Weather
	General weathering
Operating problems	Wind
	Wave erosion
Cavitation	Ozone
Impingement	
Maintenance cleaning	
Reverse hydrostatic uplift	
Vandalism	
Seismic activity	

[a]Courtesy of John Wiley & Sons, Inc., New York, NY.
[b]Separation of butyl-type cured sheets at the joint because of unequal tension in the two sheets.

Cover material over buried membranes is the most expensive part of the placing procedure. The cover material should, therefore, be as thin as possible and still provide adequate protection for the membrane. If a significant hydraulic current is present in the pond, the depth of coverage should be greater than 25 cm (1 ft), and this minimum depth should only be used when the material is erosion resistant and also cohesive. Such a material as a clayey gravel is suitable. If the material is not cohesive, or if it is fine grained, a higher amount of cover is needed (20).

Maintenance costs for different types of linings are difficult to estimate. Maintenance should include repair of holes, cracks, and deterioration, weed control expenses and animal damages, and damages caused by cleaning the pond, if that is necessary. Climate, type of operation, type of terrain, and surface conditions also influence maintenance costs. Plastic soil cement 7.5 cm (3 in) thick costs about $3.50/m^2 ($0.33/ft^2).

Cost comparisons of various liners indicate that natural and chemical sealants are the most economical sealers. Unfortunately, natural and chemical sealers are dependent upon local soil conditions for seal efficiency and never form a complete seal. Asphalt-type and synthetic liners compete competitively on a cost basis, but have different practical applications. Synthetic liners are most practical for zero or minimum seepage regulations, for industrial waste that might degrade concrete or earthen liners, and for extremes in climatic conditions.

4.4 Pond Hydraulics

4.4.1 Inlet and Outlet Configuration

In the past, the majority of ponds were designed to receive influent wastewater through a single pipe, usually located toward the center of the pond. Hydraulic and performance studies (22-25) have shown that the center discharge is not the most efficient method of introducing wastewater to a pond. Multiple inlet arrangements are preferred even in small ponds [<0.5 ha (1.2 ac)]. Outlets should be located as far as possible and preferably by means of a long diffuser. The inlets and outlets should be placed so that flow through the pond has a uniform velocity profile between the next inlet and outlet.

One form of multiple inlet, used for ponds as large as 20 acres, uses inlet head loss to induce internal pond circulation and initial mixing. The inlet pipe, laid on the pond bottom, has multiple ports or nozzles all pointing in one direction and at a slight angle above the horizontal. Port head loss is designed for about 0.3 m (1 ft) at average flow, resulting in a velocity of 2.4 m/sec (8 ft/sec).

Single inlets can be used successfully if the inlet is located the greatest distance possible from the outlet structure and baffled or the flow directed to avoid currents and short circuiting. Outlet structures should be designed for multiple depth withdrawal, and all withdrawals should be a minimum of 0.3 m (1 ft) below the water surface.

4.4.1.1 Pond Transfer Inlets and Outlets

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

Transfer pipes should be numerous and large enough to limit peak head loss to about 7-10 cm (3-4 in) with the pipes flowing about two-thirds to three-quarters full. Supply- and return-channel sizing should assure that the total channel loss is no more than one-tenth of the transfer-pipe losses. When such a ratio is maintained, uniform distribution is assured.

By operating with the transfer pipes less than full, unobstructed water surface is maintained between the channels and ponds, which controls scum buildup in any one area. If the first cell is designed to remove scum, then the transfer pipes must be submerged.

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

Specially made fiberglass plugs can be provided to close the pipes (Figure 4-15). The plugs may be installed from a boat. Such plugs permit any pipe to be closed without expensive construction of sluice gates and access platforms at each transfer point. Launching ramps into each pond and channel are recommended to assure easy boat access for sampling, aquatic plant control, and pond maintenance.

4.4.2 Baffling

Better treatment is obtained when the flow is guided more carefully through the pond. In addition to treatment efficiency, economics and esthetics play an important role in deciding whether or not baffling is desirable.

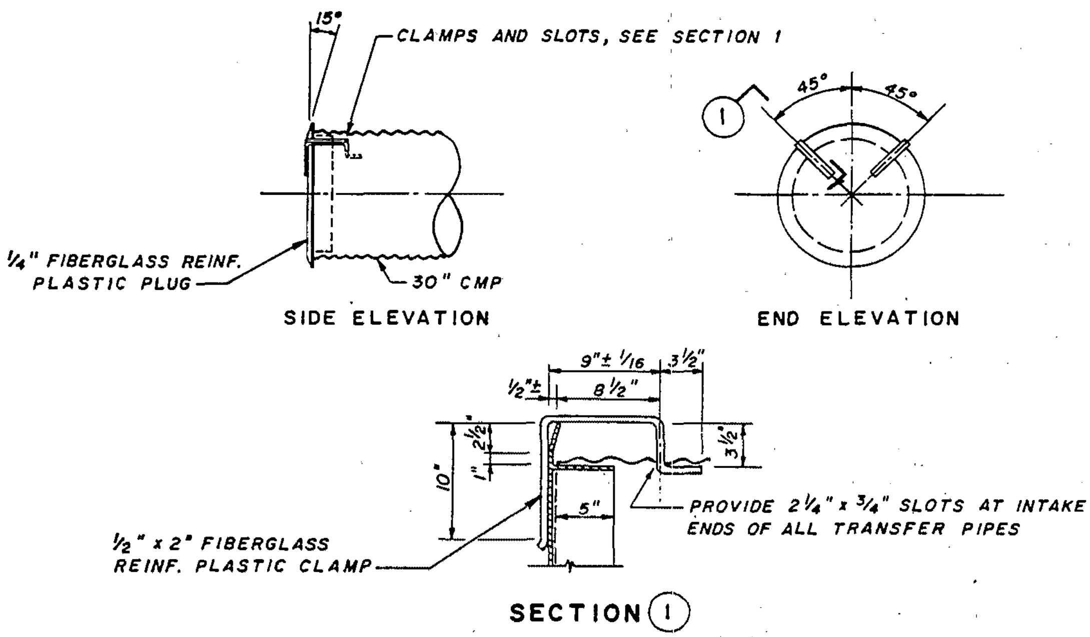
Because there is little horizontal force on baffling except that caused by wave action, the baffle structure need not be particularly strong. It may also be placed below the pond surface to help overcome esthetic objections. A typical type of baffle to consider might be a submerged fence attached to posts driven into the pond bottom and covered with a heavy plastic, flexible membrane. Commercial float-supported plastic baffling for ponds also is available.

In general, the more baffling that is used, the better the flow guidance and treatment efficiency. The lateral spacing and length of the baffle should be specified so that the cross-sectional area of the flow is as close to a constant as possible.

Baffling has an additional virtue. The spiral flow induced when flow occurs around the end of the baffles enhances mixing and tends to break up or prevent any stratification or tendency to stratify.

FIGURE 4-15

SPECIAL FIBERGLASS PLUG



TYPICAL PLUG DETAIL

NO SCALE

Winter ice can damage or destroy baffles in cold climates. Care must be exercised when designing baffles for a cold climate.

4.4.3 Wind Effects

Wind generates a circulatory flow in bodies of water. To minimize short circuiting due to wind, the pond inlet-outlet axis perpendicular to the prevailing wind direction should be aligned. If for some reason the inlet-outlet axis cannot be oriented properly, baffling can be used to control, to some extent, the wind-induced circulation. It should be kept in mind that in a constant depth pond, the surface current is in the direction of the wind and the return flow is in the upwind direction along the bottom.

4.4.4 Stratified Ponds

Ponds that are stratified because of temperature differences between the inflow and the pond contents tend to behave differently in winter and summer. In summer, the inflow is generally colder than the pond so it sinks to the pond bottom and flows toward the outlet. In the winter, the reverse is generally true and the inflow rises to the surface and flows toward the outlet.

A likely consequence of this behavior is that the effective volume of the pond is reduced to that of the stratified inflow layer (density current). The result can be a drastic decrease in detention time and an unacceptable level of treatment.

One strategy to employ is to use selective pond outlets positioned vertically so that outflow is drawn from the layer with density different from that of the inflow. For example, under summer conditions the inflow will occur along the pond bottom. Hence, the outlets should draw from water near the pond surface. This concept has not been tested but seems likely to improve performance over "full-depth" outlet structures.

Another approach is to premix the inflow with pond water while in the pipe or diffuser system, thereby decreasing the density difference. This could be accomplished by regularly constricting the submerged inflow diffuser pipe and locating openings in the pipe at the constrictions. The low pressure at the pipe constrictions would draw in pond water and mix it with the inflow to alter the density. However, clogging of openings with solid material could be a problem.

4.5 Pond Recirculation and Configuration

Pond recirculation involves interpond and intrapond recirculation as opposed to mechanical mixing in the pond cell. The effluents from pond cells are mixed with the influent to the cells. In intrapond recirculation, effluent from a single cell is returned to the influent to that cell. In interpond recirculation, effluent from another pond is returned and mixed with influent to the pond (Figure 4-16).

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

Both interpond and intrapond recirculation can affect stratification in ponds, and thus gain some benefits ascribed to pond mixing, which is discussed later. Pond recirculation is not generally as efficient as are mechanical systems in mixing facultative ponds.

Recirculation is used principally in overloaded or improperly sized ponds. Other than for dilution in the case where pond influents with very high concentrations of wastes are being treated, in most cases the increased energy costs associated with recirculation would dictate against its use.

Three common types of interpond-recirculation systems (series, parallel, and parallel series) are shown in Figure 4-16. Others have been suggested but seldom used.

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

$$t = \frac{V}{(1 + r)\ F}$$

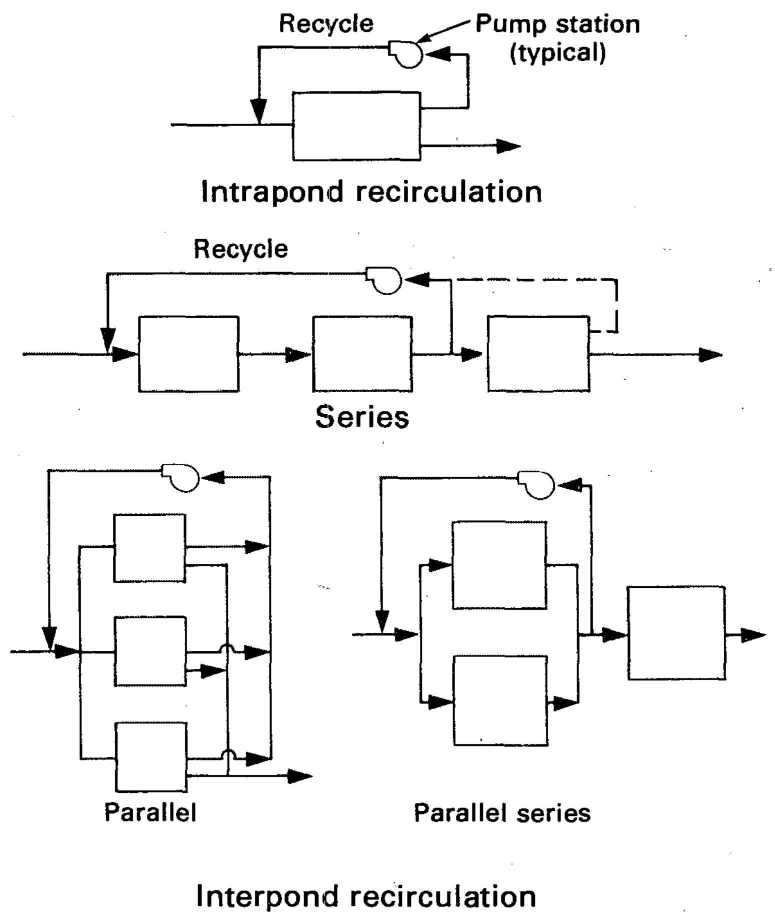
where V is the volume of pond cell, F is the influent flow rate, r, or R/F, is the recycle ratio, and R is the recycle flow rate.

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

$$S_m = \frac{S_{in}}{1 + r} + \left(\frac{r}{1 + r}\right) S_3$$

FIGURE 4-16

COMMON POND CONFIGURATIONS AND RECIRCULATION SYSTEMS



where S_m is the BOD of the mixture, S_3 is the effluent BOD from the third cell, and S_{in} is the influent BOD. Thus, S_m would be only 20 percent of S_{in} with a 4:1 recyle ratio, as S_3 would be negligible in almost all cases. Thus, the application of organic load in the pond is spread more evenly throughout the ponds, and organic loading and odor generation near the feed points are less. Recirculation in the series mode has been used to reduce odors in those cases where the first pond is anaerobic. The recirculation ratio is selected based on the loading rate applied to the cell that will not cause a nuisance.

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

For example, consider three ponds, either in series or parallel. In the parallel configuration, the surface loading (kg BOD_5/ha/d) on the three ponds is one-third that of the first pond in the series configuration. The parallel configuration, therefore, is less likely to produce odors than the series configuration. However, the hydraulic improvements in design using a series configuration generally will offset the benefits of reduced loading in parallel configuration.

Based upon the analyses of performance data from selected aerated and facultative ponds (see the Appendix and Chapter 2), four ponds in series are desirable to give the best BOD_5 and fecal coliform removals for ponds designed as plug flow systems. Good performance can be obtained in a smaller number of ponds if baffles or dikes are used to optimize the hydraulic characteristics of the system.

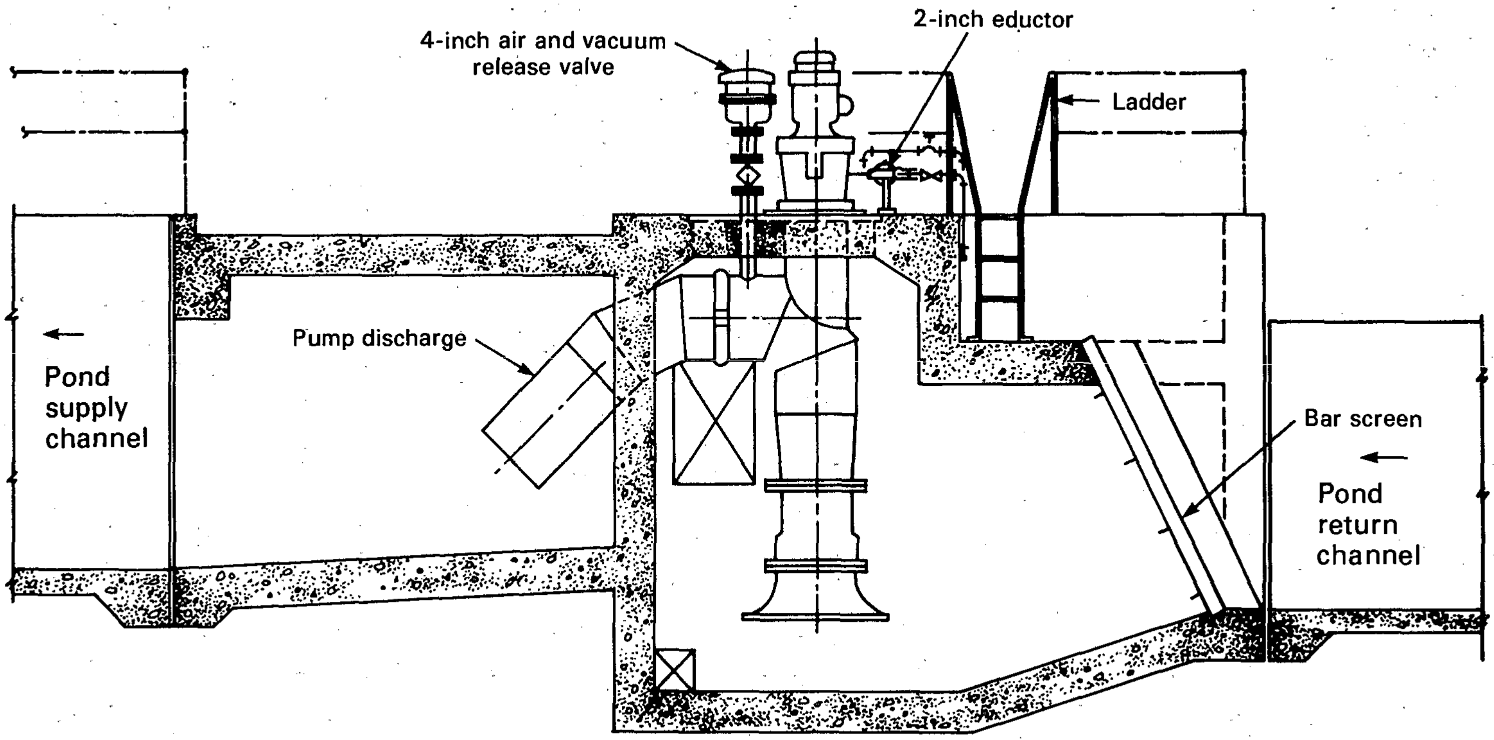
Recirculation usually is accomplished with high-volume, low-head propeller pumps. Figure 4-17 presents a simplified cross section of such an installation. In this design, the cost and maintenance problems associated with large discharge flap gates are eliminated by the siphon discharge. An auxiliary pump with an air eductor maintains the siphon. Siphon breaks are provided to ensure positive backflow protection.

Pumping stations of this type can be designed to maintain full capacity with minimal increase in horsepower even when the inlet and discharge surface levels fluctuate over a range of 1.0-1.2 m (3-4 ft). Multiple- and/or variable-speed pumps are used to adjust the recirculation rate to seasonal load changes.

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

FIGURE 4-17

CROSS SECTION OF A TYPICAL RECIRCULATION PUMPING STATION



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CHAPTER 5

ALGAE, SUSPENDED SOLIDS, AND NUTRIENT REMOVAL

5.1 Introduction

Stabilization ponds are an effective means of treating wastewater, reducing BOD_5 and coliforms, but can have an occasional high concentration of suspended solids (SS) in the effluent. During some months of the year, the SS concentration exceeds the secondary effluent standards specified by regulatory agencies. SS concentrations can exceed 100 mg/l, but such high levels are usually limited to two to four months during the year. In the standards established on October 7, 1977, small flow pond systems were excluded from the Federal SS effluent requirements. For discharge to water quality limited streams, removal may be required.

A discussion taken from many sources is presented of the various alternatives available for upgrading the effluent quality from existing ponds and designing original systems to meet water quality standards (1-3).

5.2 In-Pond Removal Methods

There are several factors to be considered for in-pond removal of particulate matter:

1. Subsequent degradation of settled matter by microorganisms to produce dissolved BOD_5, which could then have an effect on the receiving water.

2. Possibility that settled material will not remain settled.

3. Lack of positive control of effluent SS.

4. Problem of eventually filling the pond.

5. Possibility that anaerobic reactions within the settled material will produce malodors.

At first glance, it seems that some of these problems could be resolved by rather simple changes in operation. In ponds that have cells in series, the settled material could be removed from the bottom of the last cell and transferred to the anaerobic cell or primary cell in which biological degradation is encouraged. Positive control could be achieved by adding coagulants to the final cell to ensure that settling of the SS takes place. For example, chemicals such as lime, ferric chloride, and alum might be used in this manner. Generally, complete containment ponds have a life expectancy of about 20 years (4). In areas short of land, filling could be a problem, but it might be possible to dredge and remove the solid material after 10 to 20 years have elapsed and to restore the pond to its initial state. In areas where land is available and land cost is not prohibitive, filling would not be a problem.

5.2.1 Series Ponds

Series ponds are recommended by some state regulatory agencies to provide for algae sedimentation within cells. The efficiency of sedimentation in cells, however, is limited by factors such as wind mixing and algae species. Small cells usually result in less mixing (5).

5.2.2 Series Ponds with Intermediate Chlorination

Chlorination is normally used to disinfect effluent, but it has been observed that chlorine added to pond effluent will also kill algae and cause settling. In 1946 a study was conducted of a series of four oxidation ponds followed by a chlorine-contact pond near Dublin, CA. At a flow of 14,200 m^3/d (3.75 mgd) the chlorine-contact pond had a retention time of 13.5 hr. All the algae were reported killed with a chlorine dose of 12 mg/l. In addition, between chlorine-contact pond inlet and outlet, the BOD_5 was reduced from 45 to 25 mg/l, SS from 110 to 40 mg/l, and turbidity from 170 to 40 JTUs (6). Similar reductions were reported in a later study, which found that volatile SS could be reduced 52 percent and turbidity 32 percent through chlorination (7). Laboratory tests with varying chlorine doses showed that 18 to 28 mg/l of chlorine could be added without producing a chlorine residual above 0.5 mg/l in the effluent. In these tests, a 67 percent SS removal (24 mg/l in the effluent) and 68 percent BOD_5 removal (5 mg/l in the effluent) were reported. The flocculating effect of chlorine is thought to result from rupture of the algae cell wall and release of cellular metabolites that may serve as flocculants (8).

Recent studies at Utah State University on the chlorination of pond effluent do not confirm the concerns expressed earlier by Echelberger et al. (8) and Hom (9) that destruction of algae and lysis of cells will occur with high doses of chlorine. Rather, these were found to occur only when free residual chlorine is available (10)(11). These studies have shown relatively little COD released to the effluent by the chlorination of algae-laden waters with chlorine dosages adequate for disinfection, and dosages as high as 30 mg/l with 63 mg/l of SS have produced very little change in the COD. In general, COD increased with free available chlorine residual but, at residual concentrations less than 2 mg/l, there appeared to be no consistent pattern. With a reasonable degree of dosage control, disinfection or killing of algae should improve settling and reduce effluent BOD_5 and SS. The use of high doses of chlorine could result in the production of toxic chlorinated organic compounds which may cause problems in receiving streams.

5.2.3 Controlled Discharge Ponds

Controlled discharge is defined as limiting the discharge from a pond system to those periods when the effluent quality will satisfy discharge requirements. The usual practice is to prevent discharge from the pond during the winter, and during spring and fall overturn periods.

The operations of 49 controlled discharge ponds in Michigan have been well documented (12). Ponds at that latitude usually have low BOD_5 loadings, averaging about 22 kg/ha/d (20 lb/ac/d). All the systems studied were designed for discharge twice yearly, with effluent retention between late November and mid-April and from about mid-May to mid-October. Discharge times coincided with periods of low algae-SS levels. Mean effluent concentrations were about 15 mg/l BOD and 30 mg/l SS.

A similar study of controlled discharge pond systems was also conducted in Minnesota (12). The discharge practices of the 39 installations studied were similar to those in Michigan. The results of that study from the fall discharge period indicated that the effluent BOD_5 concentrations for 36 of the 39 installations sampled were less than 25 mg/l and the effluent SS concentrations were less than 30 mg/l. In addition, effluent fecal coliform concentrations were measured at 17 of the installations studied. All of the 17 installations reported effluent fecal coliform concentrations of less than 200/100 ml.

The controlled discharge of pond effluent is a simple, economical and practical method of protecting receiving water quality. Routine monitoring of the pond contents is necessary to determine the proper discharge period. These discharge periods may extend throughout the major portion of the year. It may be necessary to increase the storage capacity of continuous-flow pond systems if conversion to controlled discharge is contemplated. However, many pond systems already have sufficient freeboard and thus storage capacity which could be utilized without significant physical modification.

Controlled discharge is an excellent way to control algae concentrations in pond effluents where the problem is seasonal. Most facultative ponds can be operated as controlled discharge ponds for certain periods of the year, and in many cases this is all that would be required to control the effluent SS.

5.2.3.1 Chemical Addition

A series of reports distributed by the Canadian government has indicated success with the treatment of controlled discharge ponds by adding various coagulants from a motorboat (13-15). Excellent quality effluents are produced, and the costs are relatively inexpensive. The cost of in-pond treatment and the long detention times required must be balanced against the alternatives available. Man-hour requirements for the full-scale batch treatment systems employed in Canada are summarized in Table 5-1.

TABLE 5-1

LABOR REQUIREMENTS FOR FULL-SCALE BATCH TREATMENT OF INTERMITTENT DISCHARGE PONDS (13)

	Man-hours per		
	acre	Mgal	application
Alum, liquid	2	1.6	16
Ferric Chloride,			
liquid	1.5	1.2	16
powder	13	9.6	16
Lime,			
dry chemical method	24	17.7	125
Haliburton method	1.7	1.4	16

In addition to the usual design considerations applied to controlled discharge ponds, the following physical design requirements were recommended (13):

1. A roadway to the edge of each cell with a turn-about area sufficient to carry 45 metric tons (50 tons) in early spring and late fall or a piping system to deliver the chemical to each cell and a road adequate enough to get the boats to the pond edge.

2. A boat ramp and a small dock installed in each cell.

3. Separate inlet and outlet facilities to allow diversion of raw wastewater during treatment and draw-down in multiple-cell installations for maintaining optimum effluent quality.

4. A low-level outlet pipe in the pond to allow complete drainage of the cell contents.

5. An outlet pipe from the pond of sufficient size and design to allow drainage of the treated area over a 5- to 10-day period.

6. In new, large installations, a number of medium-sized cells of 4-6 ha (10-15 ac) would be better suited to this type of treatment than one or two large cells. These medium-sized cells

could be treated individually and drawn down over a relatively short period of time, thus maintaining optimum water quality in the effluent.

Using the typical wastewater stabilization pond design required in the State of Utah with 120 days retention time for cold weather storage and the most severe conditions likely to occur, three chemical treatments per year would be required. Designing a system for 1,140 m^3/d (0.3 mgd) operation would result in approximately 136,000 m^3 (36 x 10^6 gal) of stored wastewater per treatment. Applying alum at a rate of 150 mg/l would result in 20,400 kg (44,900 lb) of alum required per treatment assuming that the hydraulic design would control the sizing of the storage ponds and neglecting evaporation. The installations would require approximately 8 ha (20 ac) of pond surface with a depth of 1.8 m (6 ft). Assuming that a relatively small boat and supply system would be adequate to distribute and mix the chemicals with the pond water, a capital investment of approximately $33,000 (1978 US $) would be required to obtain the tank trucks, storage facilities, boat and motor to carry out the operation. Amortizing the equipment for a useful life of 10 years and assuming 7 percent interest, it would cost $4,700/yr. Liquid alum costs approximately $116/metric ton ($105/ton) (equivalent dry) and using 62 metric tons (68 tons) annually would cost $7,140. Approximately 136,000 m^3 (36 x 10^6 gal) of wastewater would be treated before each discharge. Using the requirements shown in Table 5-1 of 1.6 man-hours/10^6 gal and 16 man-hours for setup and cleanup per application results in a total labor requirement of 221 man-hours/yr. At labor costs of $20/hr, the cost would be $4,420/yr. Adding all of the above costs, exclusive of the capital cost of the pond system, results in an annual cost of $16,260 or $0.040/$m^3$ ($150/$10^6$ gal) of wastewater treated.

The above costs do not include storage facilities for the alum and the additional design requirements to accommodate the alum handling equipment and the boats. However, even doubling the estimated costs it is apparent that intermittent discharge with chemical treatment is a viable alternative where applicable.

In addition to the cost advantages outlined above, batch chemical treatment of intermittent discharge ponds can produce an effluent containing less than 1 mg/l of total phosphorus. SS and BOD_5 concentrations of less than 20 mg/l can be produced consistently and only occasionally did a bloom occur during draw down of the pond. Rapid draw down would overcome this disadvantage. Sludge buildup was insignificant and would allow years of operation before cleaning would be required.

5.2.4 Continuous Overflow Ponds with Chemical Addition

Studies of in-pond precipitation of phosphorus, BOD_5, and SS were conducted over a two-year period in Ontario, Canada (16). The primary objective of the chemical dosing process was to test removal of phosphorus with ferric chloride, alum, and lime. Ferric chloride doses of 20 mg/l and alum doses of 225 mg/l, when continuously added to the pond influent, effectively maintained pond effluent phosphorus levels below 1 mg/l over a two-year period. Hydrated lime, at dosages up to 400 mg/l, was not effective in reducing phosphorus below 1 mg/l (1 to 3 mg/l was achieved) and produced no BOD_5 reduction and a slight increase in SS concentration. Ferric chloride reduced effluent BOD_5 from 17 to 11 mg/l and SS from 28 to 21 mg/l; alum produced no BOD_5 reduction and a slight SS reduction (43 to 28-34 mg/l). Consequently, direct chemical addition appears to be effective only for phosphorus removal.

A six-cell pond system located in Waldorf, MD, was modified to operate as two three-cell systems in parallel (17). One system was used as a control and alum was added to the other for phosphorus removal. Each system contained an aerated first cell. Alum addition to the third cell of the system proved to be more efficient in removing total phosphorus, BOD, and SS than alum addition to the first cell. Total phosphorus reduction averaged 81 percent when alum was added at the inlet to the third cell and 60 percent when alum was added to the inlet of the first cell. Total phosphorus removal in the control ponds averaged 37 percent when alum was added to the third cell and 50 percent when alum was added to the first cell. The effluent total phosphorus concentration during the period when alum was added to the first cell was 4.1 mg/l compared to 4.8 mg/l total phosphorus concentration in the control system effluent. When alum was added to the third cell, the effluent total phosphorus concentration averaged 2.5 mg/l with the control pond effluent averaging 8.3 mg/l. Improvements in BOD and SS removal by alum addition were more difficult to detect and, at times, increases in effluent concentrations were observed.

5.2.5 Autoflocculation and Phase Isolation

Autoflocculation of algae, predominantly Chlorella, has been observed (18-21). Laboratory-scale continuous-flow experiments with mixtures of activated sludge and algae have produced large bacteria-algae flocs with good settling characteristics (21)(22).

Floating algae blankets in the presence of chemical coagulants have been reported (23)(24). This phenomenon may be caused by the entrapment of gas bubbles produced during metabolism or by the fact that in a particular physiological state the algae have a neutral buoyancy. In a 3.2-liter/sec (50-gpm) pilot plant (combined flocculation and sedimentation), a floating algal blanket occurred with alum doses of 125 to 170 mg/l. About 50 percent of the algae removed were skimmed from the surface (24).

Because of the infrequent occurrence of conditions necessary for autoflocculation, and poor understanding of the actual mechanism involved, it is not a viable alternative for removal of algae from ponds at this time.

Phase isolation is an attempt to design a pond system in which the various processes involved in wastewater ponds are separated, and ponds performing these special functions are placed in series. Field studies of phase isolation have yielded inconclusive results (25)(26).

5.2.6 Aquaculture

Aquaculture is a centuries-old technique for producing food and fiber products for direct or indirect human consumption. Application of waste materials, for their nutritive value, to aquaculture systems has been a common practice for many years in some parts of the world. The use of designed aquaculture systems for treatment and management of municipal wastewaters is a relatively new concept. As recently as 1978, Duffer and Moyer (27) concluded that additional developmental research was needed to establish reliable design criteria. They presented a comprehensive literature review and were optimistic that several types of aquatic organisms could be developed that would be attractive in terms of treatment effectiveness, cost, and energy usage. A 1980 engineering assessment of aquaculture systems for wastewater treatment is available (28).

Although aquaculture shows promise as a potential wastewater treatment process, much remains to be learned at this time regarding removal mechanisms, design parameters, and overall applicability. The following sections present results from recent research utilizing various types of organisms (29-33).

5.2.6.1 Invertebrates

Invertebrate organisms which feed on algae include Daphnia and related species (water fleas), Artemia (brine shrimp), and assorted bivalve mollusks (oysters, clams, and mussels).

Design considerations for culturing invertebrates must take into account their environmental requirements. These include pond site selection, construction of berms and baffles, inlet and outlet structures, mixing and depth, substrate, and pH regulation. Culture ponds require rigid operational control and extensive management (34).

Based upon the experiences with invertebrates in wastewater ponds to remove algae, their use does not appear to be feasible at this time.

5.2.6.2 Water Hyacinth

Water hyacinth is an aquatic plant native to South America that was introduced into the United States in 1884. The species currently grows throughout Florida, Southern Georgia, Alabama, Mississippi, Louisiana, and in parts of Texas and California. Temperatures below freezing will kill the plant. The plants form dense mats, interfering with most uses of waterways; the hyacinth has been designated a noxious weed by the U.S. Government. Under favorable conditions, the total plant mass can double in periods of a few weeks. In order to support this rapid plant growth, hyacinths consume large amounts of nitrogen and phosphorus, making it a potentially useful means for nutrient removal from pond effluent.

Dinges (35) experimented with using water hyacinths cultivated in shallow basins for treating pond effluent from the Williamson Creek Wastewater Treatment Facility at Austin, TX. Effluent BOD_5 levels through the experimental system were reduced by 97 percent; SS by 95 percent; and COD by 90 percent. Mean effluent BOD_5 and SS levels were less than 10 mg/l and effluent total nitrogen was less than 5 mg/l.

Wolverton and McDonald (36) diked off a 2.0 ha (5.0 ac) portion of a pond with an average depth of 1.2 m (4 ft) and stocked it with water hyacinths. Before the hyacinths were introduced the pond produced no reduction in effluent SS and a 76 percent reduction in BOD_5. After the water hyacinths were introduced, SS were reduced an average of 87 percent and BOD_5 an average of 94 percent.

Chambers (37) experimented with water hyacinths at the Exxon Baytown Refinery over a two-year period. Following introduction of water hyacinths in August, a 78 percent reduction in effluent SS was obtained in September. The tops of plants were damaged by coots, a duck-like bird, by early November. Subfreezing weather killed the tops in the winter and surface coverage by the hyacinths was reduced from 80 percent of the pond to 50 percent in January. After mechanical harvesting in late January as planned, surface coverage dropped to zero in March and effluent quality was the same as in control ponds. Experience the next year was similar. This study demonstrates the seasonal nature and some of the potential management problems associated with water hyacinths.

Water hyacinth systems are capable of removing high levels of BOD, SS, metals, and nitrogen, and significant removal of refractory trace organics (28). Removal of phosphorus is limited to the plant needs and probably will not exceed 50 to 75 percent of the phosphorus present in the wastewater. Phosphorus removal will not even approach that range unless there is a very careful management program with regular harvests. In addition to plant uptake, the root system of the water hyacinth supports a very active mass of organisms that assist in the treatment. The plant leaves also shade the water surface and limit algae growth by restricting light penetration.

Multiple-cell pond systems where water hyacinths are used on one or more of the cells are the most common system design (28). Based on current experience, a pond surface area of approximately 1600 ha/10^6 m^3 (15 ac/10^6 gal) seems reasonable for treating primary effluent to secondary or better quality. An area of about 500 ha/10^6 m^3 (5 ac/10^6 gal) should be suitable for systems designed to polish secondary effluent to achieve higher levels of BOD and SS removals. For enhanced nutrient removal from secondary effluent, an area of approximately 1300 ha/10^6 m^3 (12 ac/10^6 gal) seems reasonable. Effluent quality from such a system might achieve: <10 mg/l for BOD and SS, <5 mg/l for N, and approximately 60 percent P removal. This level of nutrient removal can only be obtained with careful management and harvest to yield 110 dry metric tons/ha (50 tons/ac) per year.

The organic loading rates and detention times used for water hyacinth systems are similar to those used for conventional stabilization ponds that treat raw wastewater (28). However, the effluent from the water hyacinth system can be much better in quality than from a conventional stabilization pond, particularly with respect to SS (algae), metals, trace organics, and nutrients.

Harvest of the water hyacinth or duckweed plants may be essential to maintain high levels of system performance (28). It is essential for high levels of nutrient removal. Equipment and procedures have been demonstrated for accomplishing these tasks. Disposal and/or reuse of the harvested materials is an important consideration. The water hyacinth plants have a moisture content similar to that of primary sludges. The amount of plant biomass produce (dry basis) in a water hyacinth pond system is about four times the quantity of waste sludge produced in conventional activated sludge secondary wastewater treatment. Composting, anaerobic digestion with methane production, and processing for animal feed are all technically feasible. However, the economics of these reuse and recovery operations do not seem favorable at this time. Therefore only a portion of the solids disposal costs will be recovered unless the economics can be improved.

The major cost and energy factors for water hyacinth systems are construction of the pond system, water hyacinth harvesting and disposal operations, aeration (if provided), and greenhouse covers where utilized (28). Evapotranspiration in arid climates can be a critical factor. The water loss from a water hyacinth system will exceed the evaporation from a comparably sized pond with open water. Greenhouse structures may be necessary where such water loss and related increase in effluent TDS are a concern.

Mosquito control is essential for water hyacinth systems and can usually be effectively handled with Gambusia or other mosquito fish. Legal aspects are also a concern. The transport or sale of water hyacinth plants is prohibited by Federal and state law in many situations. The inadvertent release of the plants from a system to local waterways is a potential concern to a number of different agencies. A fixed barrier can be used to prevent escape of the plants to the outside environment. Water hyacinth plants cannot survive or reproduce in cool waters so the concept is limited to "warm" areas unless climate control is provided. Other floating plants, such as duckweed, alligator weed, and water primrose, have a more extensive natural range but only limited data on their performance in wastewater treatment are available.

5.2.6.3 Fish

In Asia, fish, most commonly members of the carp family, have been cultured in highly enriched water for centuries. Schroeder (38) has shown that fish can be effectively combined with plankton and bottom fauna to produce a system that is biologically balanced with stable DO and pH levels.

Experiments at the Exxon Baytown Refinery using Golden Shiners, fathead minnows, Tilapia noloticia, and mullet were unsuccessful; in fact, effluent SS levels increased. Stomach analysis of the fish revealed that, in addition to algae, they fed on alga-feeding invertebrates. Reid (39) concluded that there was a serious gap between fish culture and constraints of sanitary engineering, pointing towards the need for further research.

Preliminary experiments at Benton, AR, compared parallel three-cell stabilization ponds receiving equal volumes of the same wastewater (BOD - 260 mg/l, SS - 140 mg/l) (28). The cells in one set were stocked with silver, grass, and bighead carp while the other set received no fish and was operated as a conventional stabilization pond. The comparative study continued for a full annual cycle. Results indicated generally similar performance of the two systems but the fish culture units consistently performed somewhat better than the conventional pond. For example, the effluent BOD from the fish system ranged from about 7 to 45 mg/l with values less than 15 mg/l obtained more than 50 percent of the time. The conventional pond system had effluent BOD ranging from 12 to 52 mg/l with values less than 23 mg/l about 50 percent of the time. SS were very similar in the effluents for both systems except in July when the concentration was about 110 mg/l for the conventional pond and 60 mg/l for the fish system.

In the second phase of the study at Benton, the six cells were all connected in series and a baffle constructed in each to reduce short circuiting. Silver carp and bighead carp were stocked in the last four cells and additional grass carp, buffalofish, and channel catfish in the final cell. No supplemental feed or nutrients were added to the fish culture cells. Estimated fish production after 8 months was over 3,300 kg/ha (7,200 lb/ac).

Effluent quality steadily improved during passage through the six-cell system. BOD removal for the entire system averaged 96 percent for the 12-month study period. About 89 percent of that removal was achieved in the first two conventional cells. SS removal averaged 88 percent in the entire system, with 73 percent occuring in the two conventional cells. It is not clear whether the fish or the additional detention time or some combination is responsible for the additional 7 percent BOD removal in the final four fish culture cells. The final average effluent BOD concentration of about 9 mg/l is typical for a six-cell conventional stabilization pond system of comparable detention time. It seems very likely that the fish contributed significantly to the low SS in the final effluent (17 mg/l) via algal predation. A value two or three times that high might be expected for conventional stabilization ponds.

5.2.6.4 Integrated Systems

Experiments have been conducted on combinations of several types of algae-reducing organisms. Ryther (40) concluded that highly enriched environmental systems were relatively unstable and difficult to control, often failing to develop a diversified biological community.

A prototype integrated aquaculture treatment system was constructed in Hercules, CA, by Solar AquaSystems, Inc. (41). Startup problems were encountered because of poor construction practices and efforts were unsuccessful to make the system function parallel to the experiences of the pilot facilities. The system has been abandoned.

5.2.7 Baffles

The encouragement of attached microbial growth in ponds is an apparent practical solution for maintaining biological populations, obtaining the desired treatment, and reducing the SS level. Although baffles are considered useful primarily to ensure good mixing and eliminate the problem of short-circuiting, they may also provide a surface on which bacteria, algae, and other microorganisms can grow. In a study of anaerobic and facultative ponds with baffling, the microbiological community consisted of an algae gradient, from photosynthetic chromogenic bacteria to nonphotosynthetic, nonchromogenic bacteria (42)(43). In these baffle experiments, the presence of growth attached to the baffles was the reason attributed for the higher efficiency of treatment than that found in a nonbaffled system.

5.3 Filtration Processes

5.3.1 Intermittent Sand Filtration

5.3.1.1 Summary of Investigations

Literature reviews are available discussing the history, theory, design, operation, performance, modeling, and economics of intermittent sand, slow sand, rapid sand, and other media filtration of potable water and wastewater (44-58). The following is a condensation of these reviews and contains a brief history of intermittent sand filtration of wastewater as well as a summary of studies concerning intermittent sand filtration to upgrade pond effluents (see Table 5-2).

These studies indicate that, with proper design and operation, intermittent sand filtration is an effective and economical process to upgrade wastewater stabilization pond effluent to meet present and future discharge requirements. The effective sand size has been found to be the most important variable relative to quality of effluent and the ability of the process to meet effluent requirements. Hydraulic loading rate does not have a great effect on the effluent quality but does play an important role in the economics of filter run time. Lengthening the filter run time requires either a decrease in loading rate, which in turn creates a larger initial construction cost along with increased maintenance costs, or a sacrifice in the quality of effluent. Neither variation guarantees any consistent run time because pond effluent quality can fluctuate greatly during the year and can increase or decrease the filter run time.

TABLE 5-2

INTERMITTENT SAND FILTRATION STUDIES

Pond Type	u*	Loading Rate mgad	SS Influent mg/l	SS Effluent mg/l	SS Removal percent	VSS Influent mg/l	VSS Effluent mg/l	VSS Removal percent	BOD Influent mg/l	BOD Effluent mg/l	BOD Removal percent	Reference
Facultative	5.8	0.1	13.7	4.0	71	9.2	2.0	78	6.3	1.2	82	44
		0.2	13.7	4.8	65	9.2	2.1	77	6.3	1.3	80	
		0.3	13.7	6.0	56	9.2	2.3	75	6.3	2.0	69	
Facultative	9.74	0.2	30.3	3.5	88	23.0	1.3	94	19.5	1.9	90	47
		0.4	30.1	2.9	90	22.5	3.4	85	20.6	2.5	88	
		0.6	34.0	5.9	83	25.9	3.1	88	25.6	4.2	84	
		0.8	23.9	4.7	80	15.2	1.2	92	2.8	1.8	36	
		1.0	28.5	5.1	82	21.5	2.5	88	13.5	2.6	81	
		1.0	24.3	3.7	85	18.6	1.6	91	6.1	2.2	64	
Facultative	6.2	0.5	32.4	8.6	74	21.9	3.3	85	10.7	1.8	83	48
		1.0	32.4	7.8	76	21.9	3.2	85	10.7	2.0	82	
		1.5	32.4	6.4	80	21.9	3.3	85	10.7	2.3	79	
Facultative	9.73	0.25	70.7	10.1	86	38.8	6.5	83	20.2	6.6	67	49
		0.5	197	15.6	92	155	11.9	92	71.4	9.4	87	
		1.0	108	11.8	89	83.0	8.8	89	34.0	13.0	62	
Aerated	9.73	0.5	158	52.5	67	71.1	13.2	81	34.4	5.1	85	49
		1.0	68.7	32.9	52	36.6	11.3	69	19.6	11.7	40	
Anaerobic[b]	NA	0.1	353	45.5	87	264	28.1	89	123	19.5	84	50
		0.35	208	46.5	78	162	35.3	78	108	43.7	60	
		0.5	194	45.1	77	175	35.7	80	107	67.6	37	
Facultative	9.7	0.2	23.0	2.7	88	17.8	1.0	95	10.9	1.1	90	51
		0.4	20.8	3.5	83	18.5	2.3	88	11.5	2.6	77	

[a]Results for best overall performing 0.17 mm e.s. filters.

[b]Dairy waste.

*u = uniformity coefficient.

5.3.1.2 General Design Considerations

In general, the design and construction of intermittent sand filters for polishing pond effluents is similar to that used for conventional slow sand filters used in potable water treatment. Because pond systems are designed to be relatively low-maintenance systems, intermittent sand filters designed to augment pond systems should also be designed as low-maintenance systems. The initial capital cost of constructing intermittent sand filters will be reduced substantially if filter material (i.e., embankment, sand and gravel) are locally available or can be processed on site. It is obvious that importation of these materials to the construction site will significantly increase the cost of construction. In such instances, a complete economic comparison of various pond effluent polishing techniques should be performed.

Basically, there are two different configurations employed for intermittent sand filtration of pond effluent, single-stage and series. Single-stage intermittent sand filtration consists of passing pond effluent through a single intermittent sand filter employing a reasonably small (0.20 mm to 0.30 mm) effective sand size. Series intermittent sand filtration consists of passing pond effluent through two or more separate intermittent sand filters with each filter employing a different effective sand size. The initial filter sand size in a series operation is relatively large (0.60 mm to 0.70 mm) while the subsequent filters employ smaller sands (0.15 mm to 0.40 mm).

5.3.1.3 Configuration

The decision to use a single-stage or a series intermittent sand filtration system should be based on the effluent quality required, desired length of filter run, hydraulic head available, and availability of various filter sand sizes. Each of the above considerations has a direct impact on the economics of the configuration selected.

The effluent quality produced by an intermittent sand filter is almost totally a function of the filter sand size employed. The smaller filter sand sizes also plug faster and thus reduce the length of filter run. In areas where a high quality effluent is not required (i.e., BOD_5 $\leq$30 mg/l) and SS $\leq$30 mg/l), a single-stage filter with a medium filter sand size will produce a reasonable filter run length and the required effluent quality. If a high quality effluent is desired (BOD_5 $\leq$10 mg/l and SS $\leq$10 mg/l), then series intermittent sand filtration with a small final stage filter sand size should be considered. In addition, series intermittent sand filtration is applicable in cases where the lower operation and maintenance costs associated with long filter run lengths are desirable at the expense of initial capital costs.

Series intermittent sand filtration may also not be economically feasible where sufficient hydraulic head is not available to allow flow from one filter to the next. Siphons and pumps may be used to transfer effluent from one filter to the next in series; however, the operational costs involved should be closely examined.

Series intermittent sand filters may not be economically feasible in areas where different filter sand sizes are not readily available. Again, an economic comparison should be made.

In general, series intermittent sand filtration is capable of producing an effluent equal to or better than a single-stage filter and will generally have longer filter runs. The costs of series intermittent sand filtration may be more than those associated with single-stage intermittent sand filtration. To date, there is more reliable design and operational data for single-stage intermittent sand filtration than for series filtration.

5.3.1.4 Hydraulic Loading Rate

The removal efficiency by an intermittent sand filter does not appear to be seriously affected by variations in hydraulic loading rates (44-47)(59)(60). Effluent quality deteriorated only slightly with significant increases in hydraulic loading rate (48).

The length of filter run is also not directly affected solely by hydraulic loading rate. Rather, length of filter run is affected by a combination of hydraulic loading rate and influent SS concentration. Attempts have been made to relate the time a filter performs between cleanings to the mass of organic material removed or applied, i.e., (hydraulic loading rate) x (influent SS concentration) (45-47). Experience with full-scale units summarized in Figures 5-1 through 5-4 represent the relationship between mass loading and the time between cleaning the filters (47)(51)(61). Figures 5-1 through 5-3 represent the performance of filters with sands of various effective size located in an area with relatively soft water (total hardness <250 mg/l), and Figure 5-4 applies to areas with hard waters where calcium carbonate precipitation is likely to occur during periods of active algal growth (61).

a. Single-Stage

Information available from full-scale operations indicates that hydraulic loading rates of 0.37 to 0.56 $m^3/m^2/d$ (0.4 to 0.6 mgad) may be employed using single-stage intermittent sand filtration. In areas where high influent SS concentrations are anticipated (above 50 mg/l average) lower hydraulic loading rates, 0.19 to 0.37 $m^3/m^2/d$ (0.2 to 0.4 mgad), are recommended. These lower hydraulic loading rates are suggested to increase the time of filter run. If the time a filter will perform is not a significant design consideration, i.e., when filters are very small, less than 90 m^2 (1000 ft^2), the higher loading rates may be employed; however, operation and maintenance costs will increase.

In areas where land is relatively inexpensive, lower hydraulic loading rates are suggested so that the time between filter cleanings may be increased. In this case, the initial capital costs will be higher, but the annual operating costs will be significantly lower. This is especially true for cold weather

FIGURE 5-1

LENGTH OF FILTER RUN AS A FUNCTION OF DAILY MASS LOADING FOR 0.17 mm EFFECTIVE SIZE SAND (61)

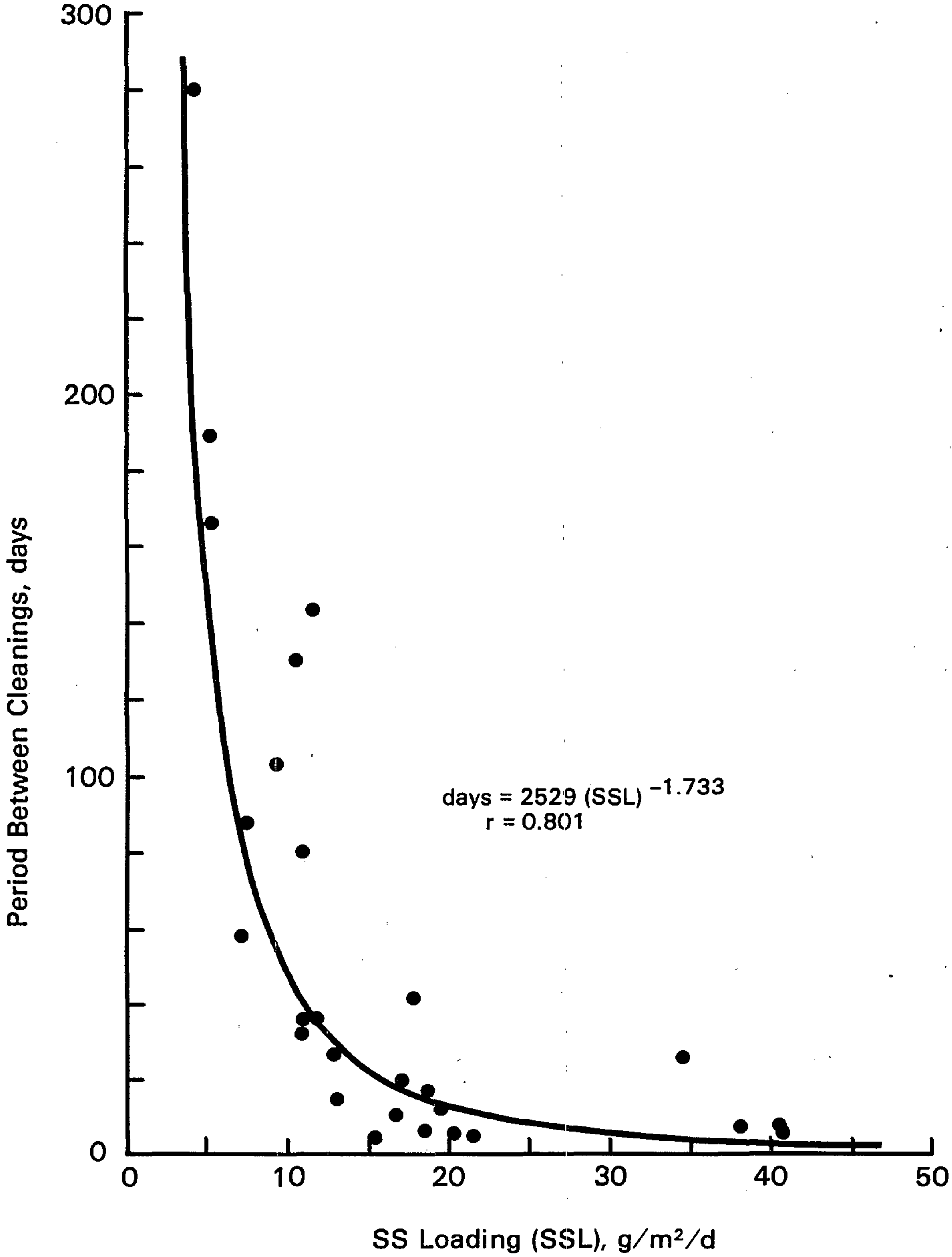


FIGURE 5-2

LENGTH OF FILTER RUN AS A FUNCTION OF DAILY MASS LOADING
FOR 0.4 mm EFFECTIVE SIZE SAND (61)

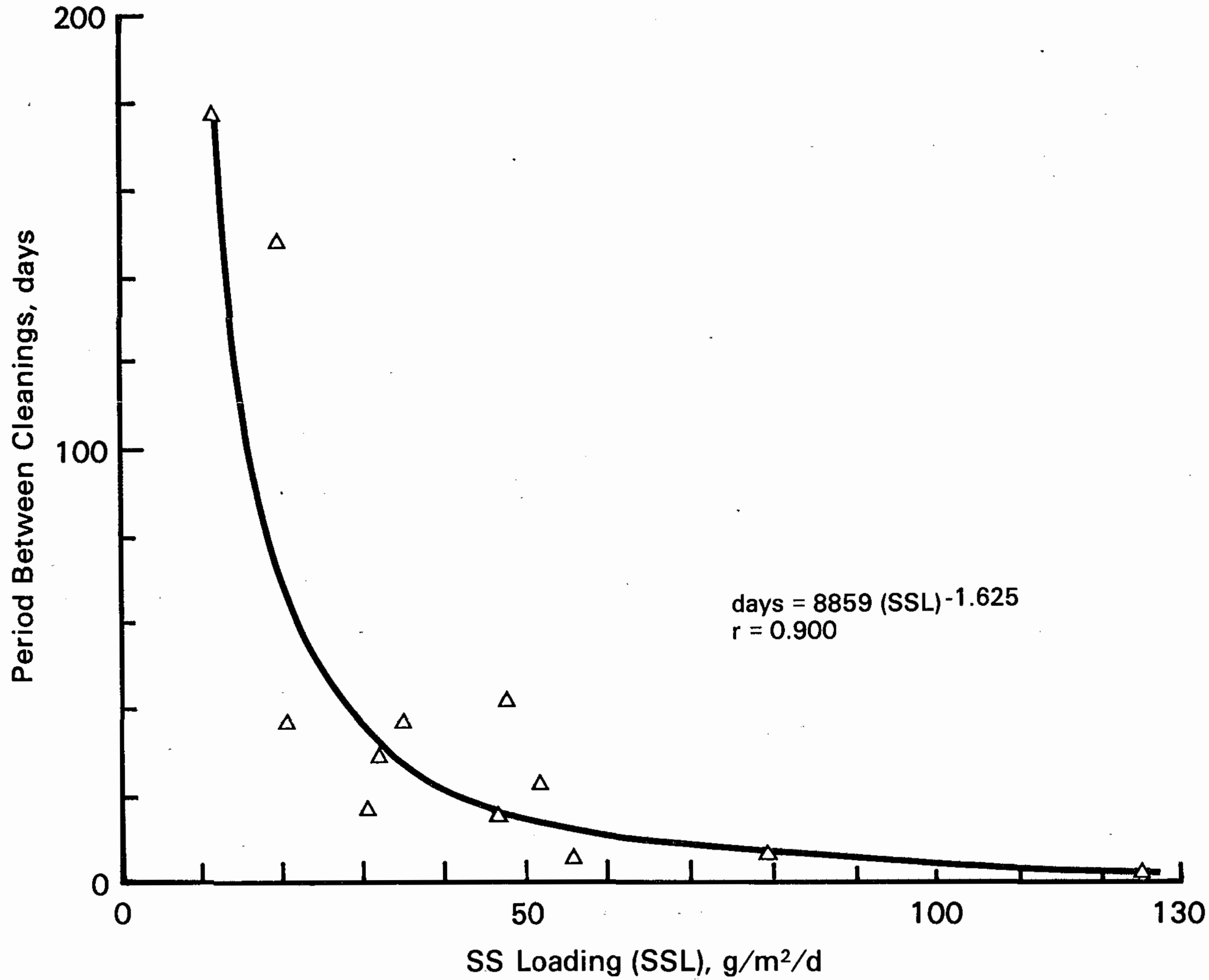


FIGURE 5-3

LENGTH OF FILTER RUN AS A FUNCTION OF DAILY MASS LOADING FOR 0.68 mm EFFECTIVE SIZE SAND (61)

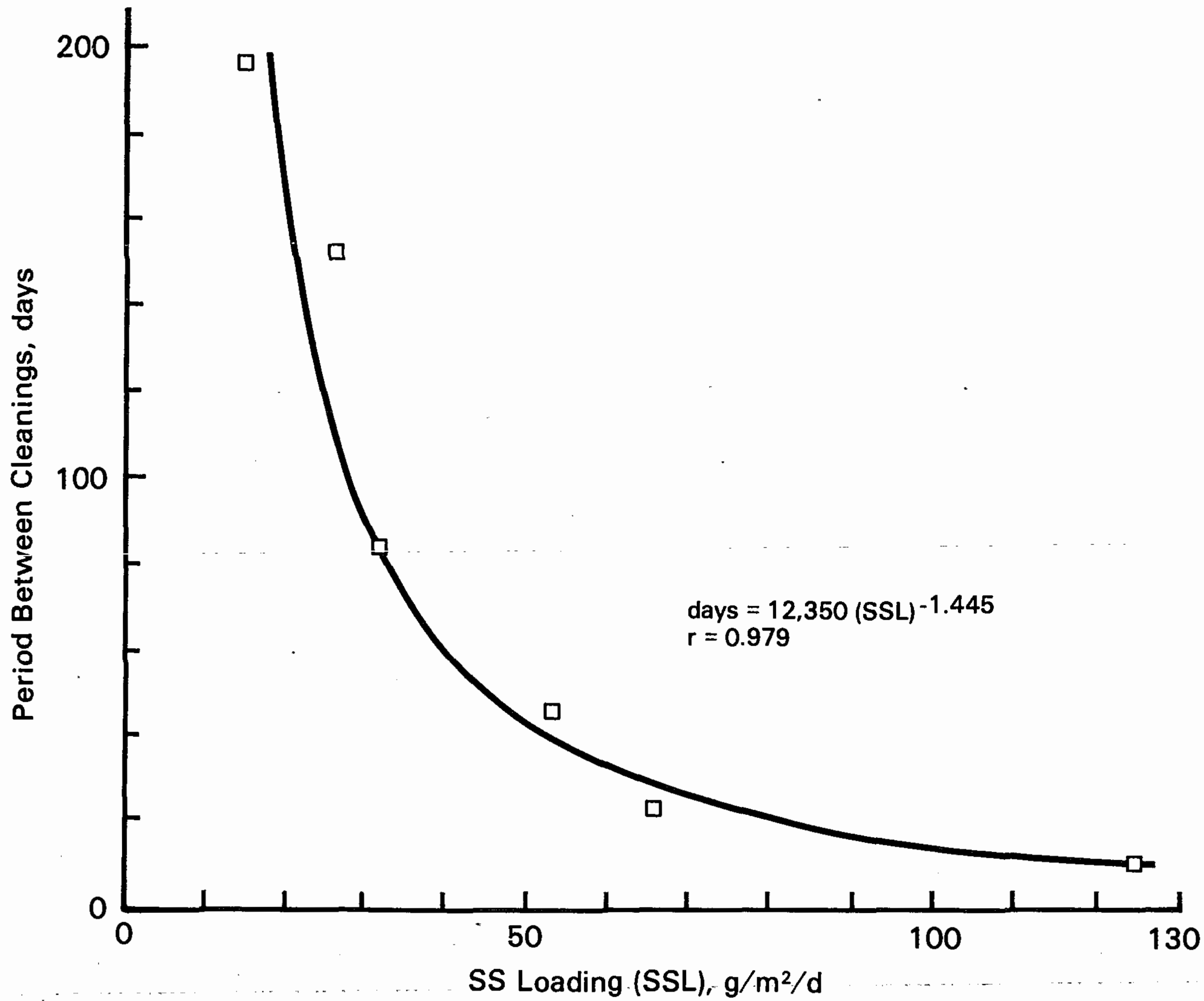
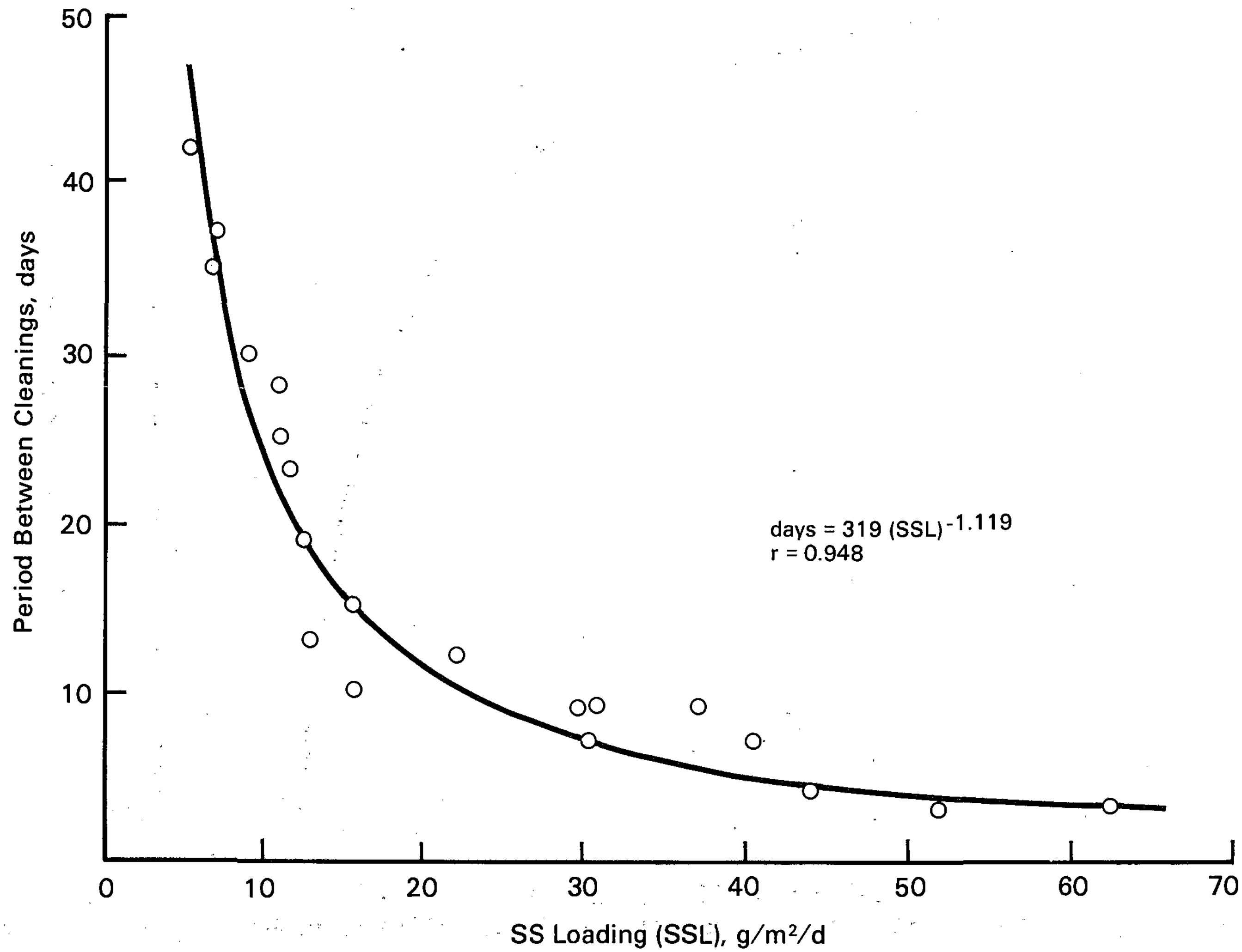


FIGURE 5-4

LENGTH OF FILTER RUN AS A FUNCTION OF DAILY MASS LOADING FOR POND EFFLUENTS HAVING CALCIUM CARBONATE PRECIPITATION PROBLEMS (0.17 mm EFFECTIVE SIZE SAND)



climates. The optimum design in cold weather climates would allow for long filter run times during the freezing months to eliminate filter scraping during this period of the year. A single-stage filter at Logan, UT, with a hydraulic loading rate of 0.19 $m^3/m^2/d$ (0.2 mgad) operated for 189 days during the winter season before scraping was necessary (45).

b. Series Operation

Series intermittent sand filters have been used only on a pilot-scale. Details of this experience can be obtained elsewhere (48).

c. Seasonal Variations

Because length of filter runs are affected by a combination of hydraulic loading rate and influent SS concentration, seasonal variations in hydraulic loading rates may be advantageous. Higher hydraulic loading rates may be employed during periods of low influent SS concentration. However, filter design must be based on the minimum hydraulic loading rate employed during the year.

5.3.1.5 Filter Size and Shape

The total filter area required for a single-stage intermittent sand filtration system is obtained by dividing the anticipated influent flowrate by the hydraulic loading rate selected for the system. For a series intermittent sand filtration system, the total area thus obtained must be supplied by each stage of the filtration system. Thus, if the same hydraulic loading rate is specified for a single-stage system and a three-stage series system treating the same influent flowrate, the total area required for the three-stage series system would be three times greater than the total area required for the single-stage system. However, in practice, the same hydraulic loading rates would not be employed to design both filter systems.

Depending upon the work schedule during cleaning operations, a filter may be totally out of service for several days. Thus, at least one spare filter should be included in the system to accommodate the cleaning schedule. Alternately, sufficient holding capacity could be included in the pond systems to allow storage of filter influent during filter cleaning operations. In any event, provisions must be made to accommodate down time during the cleaning operation. No system should have less than two filters, and three filters are preferred.

The exact size of an intermittent sand filter will depend on the influent flowrate and the hydraulic loading rate. However, if it is anticipated that sand scraping and filter cleaning operations are to be achieved by mechanical means, the filters should be large enough to allow relatively easy movement of machinery on the filter surface. For very small systems, sand scraping may be accomplished by hand. In such systems the maximum surface area per filter should not exceed approximately 90 m^2 (1,000 ft^2). For mechanically scraped filters, Huisman and Wood (62) recommended that it not exceed approximately 5,000 m^2 (55,000 ft^2).

The design of odd or random shaped filters to make the best use of available land may benefit the initial capital cost of construction; however, if mechanical cleaning is anticipated, equal size rectangular beds are preferred (59). Equal sized beds allow the alternation of loading between filters with a minimum of upset to the total system operation. In addition, cleaning and operational procedures may be standardized with equal sized filters.

5.3.1.6 Sand

Selected sand is generally used as a filter media; however, other granular substances, such as crushed coal and burnt rice husks, have been used when suitable sand was not available (62). The use of materials other than sand should be carefully evaluated in a pilot-scale filter before being used in a prototype intermittent sand filtration system.

Filter sands are generally described by their effective size (e.s.) and uniformity coefficient (u). The e.s. is the 10 percentile size, such that 10 percent of the filter sand by weight is less than that size. The uniformity coefficient is the ratio of the 60 percentile size to the 10 percentile size. An example procedure for determining e.s. and u for a specific filter sand follows:

Problem: For a sand with the characteristics shown in Table 5-3, determine effective size (e.s.) and uniformity coefficient (u).

TABLE 5-3

SIEVE ANALYSIS OF FILTER SAND

U.S. Sieve Designation No.	Size of Sieve Opening mm	Percent Passing
3/8 in	9.53	100
4	4.76	95
8	2.38	66
16	1.19	41
30	0.59	20
50	0.297	6
100	0.149	1

Solution:

1. Construct a plot of sand grain size (size of sieve opening) versus sand size distribution (percent passing) as shown in Figure 5-5.

2. Determine the 10 percentile size, P_{10}, by reading the 10 percentile line intersection on the curve in Figure 5-5.

$$P_{10} = 0.38 \text{ mm}$$

3. By definition, P_{10} is equivalent to e.s.

$$\text{e.s.} = P_{10} = 0.38 \text{ mm}$$

4. Determine the 60 percentile size, P_{60}, by reading the 60 percentile line intersection on the curve in Figure 5-5.

$$P_{60} = 2.00 \text{ mm}$$

5. By definition, u is P_{60} divided by P_{10}.

$$u = P_{60}/P_{10}$$

$$= 2.00 \text{ mm}/0.38 \text{ mm}$$

$$= 5.22$$

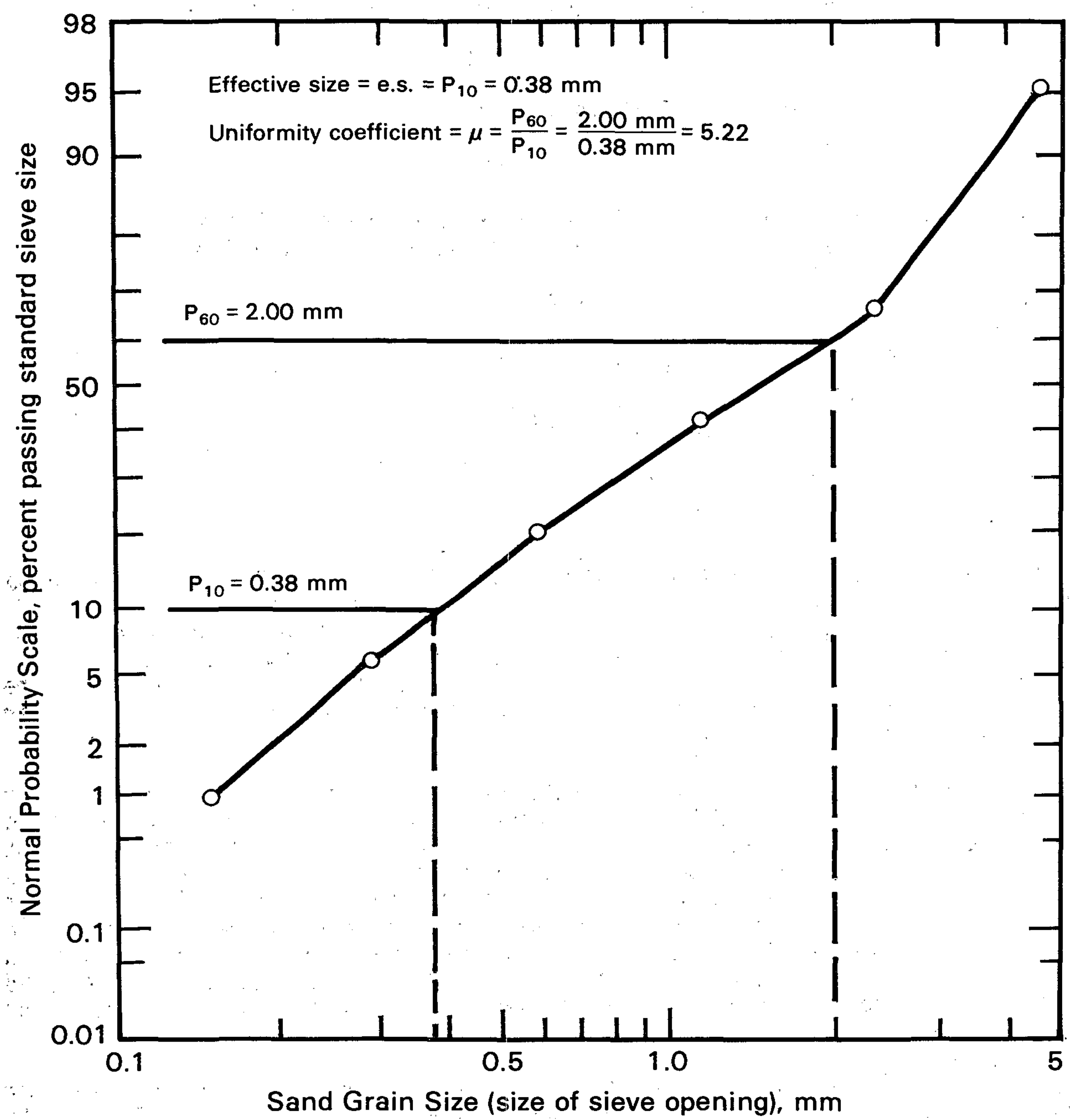
Filter sand of the required e.s. and u may be produced from stock sands by removing a given fraction that is either too large or too small. Alternately, the specified sand may be produced by mixing two sands with different characteristics. Mixing of the sands must be carried out very thoroughly, preferably in a concrete mixer. The procedures for mixing and the calculations for determining the proper portions for producing a given filter sand are described elsewhere (62)(63).

The filter may be clean river, beach, or bank sand with either sharp or rounded grains. It should be free from clay, dust, dirt, and organic impurities. It is desirable that the filter sand be washed prior to placement in the filter bed. If the sand has not been washed prior to installation, a poor quality filter effluent will result during the first few weeks of startup. The deterioration in quality is caused by washing of fine inorganic and organic material from the sand. However, once this material has been washed from the sand, a high quality effluent can be expected. The sand grains should be of a hard material which will not break down with wear and contact with water.

Experience indicates that, generally, pit run concrete sand is suitable for use in intermittent sand filters, provided the e.s. and u are suitable. Costs may be reduced substantially if a sand source can be located which does not require screening or washing. It is strongly recommended that a natural or pit run sand not requiring specialized grading be employed where possible.

FIGURE 5-5

SAND GRAIN SIZE vs SAND SIZE DISTRIBUTION



a. Single-Stage

Filter sand for a single-stage intermittent sand filter should have an e.s. ranging from 0.20 to 0.30 mm, a uniformity coefficient of less than 7.0, and less than 1 percent of the sand smaller than 0.1 mm. A higher quality effluent will be produced from filter sand with an e.s. near 0.20 mm while a slightly poorer quality effluent will result from using a filter sand with an e.s. near 0.30 mm. Most of the recent research on intermittent sand filtration has been conducted using a 0.17 mm e.s. sand (44-47)(60).

Previous reports (44)(45)(63) have recommended u values of 1.70 to 3.27, and u for slow sand filters for potable water treatment of less than 2.0 are suggested by Huisman and Wood (62). However, the recent work with polishing pond effluents utilized filter sand with a u of 9.74 (45-47). Hill et al. (48) employed filter sands with u values ranging from less than 2.0 to 9.74 with little effect on effluent quality.

It appears that the u has little effect on the quality of effluent produced from an intermittent sand filter. However, u values greater than 7.0 should be avoided. In general, u values ranging between 1.5 and 7.0 are acceptable.

b. Series Operation

In a series operation, the filter sand e.s. should decrease with each succeeding stage of the filter. Very little data exist to determine exact filter sand sizes for series operation. Hill et al. (48) reported using a 0.72 mm e.s. sand in the first-stage filter, a 0.40 mm e.s. sand in the second-stage filter, and a 0.17 mm e.s. sand in the third-stage filter. Based on these data for a three-stage series operation, the range of e.s. values for the first-stage filter should be 0.65 to 0.75 mm, the range for the intermediate stage 0.35 to 0.45 mm, and the range for the final stage 0.20 to 0.30 mm.

A careful pilot-scale study should be conducted before determining the filter sand e.s. for a two-stage series filter operation. The u values for sands used in series filter operations should be similar to those used in single-stage intermittent sand filtration.

c. Use of Highway Sand

Many highway specifications require that fine aggregate for concrete conform to the requirements of the American Association of State Highway and Transportation Officials (AASHO M6) (64). Such sand is applicable in intermittent sand filters used to polish pond effluents. The AASHO M6 gradation requirements are shown in Table 5-4.

TABLE 5-4

GRADATION REQUIREMENTS FOR FINE AGGREGATE
IN THE AASHO M6 SPECIFICATION (64)

U.S. Sieve Designation No.	Percent Passing
3/8 in	100
4	95-100
16	45-80
50	10-30
100	2-10

The sand size distribution of this sand at the specific upper and lower limits of specification are shown in Figure 5-6. Analysis of this figure indicates that the e.s. of this sand will range from 0.15 to 0.30 mm with u ranging from 4.23 to 5.39.

As can be seen in Figure 5-6, the No. 50 and No. 100 sieve are the critical points of gradation. The 10 percentile sand size must lie between these two sieve sizes because the e.s. is determined by the 10 percentile size sand. Generally, a filter sand with an e.s. of between 0.20 and 0.30 mm should be used for single stage and the final stage in series intermittent sand filters. The restrictions on u for sand employed in intermittent sand filtration are not severe. Thus, the limit on gradations for the No. 4 and No. 16 sieves are not exceptionally critical. However, 100 percent of the sand should pass a 3/8-inch sieve.

5.3.1.7 Filter Bed

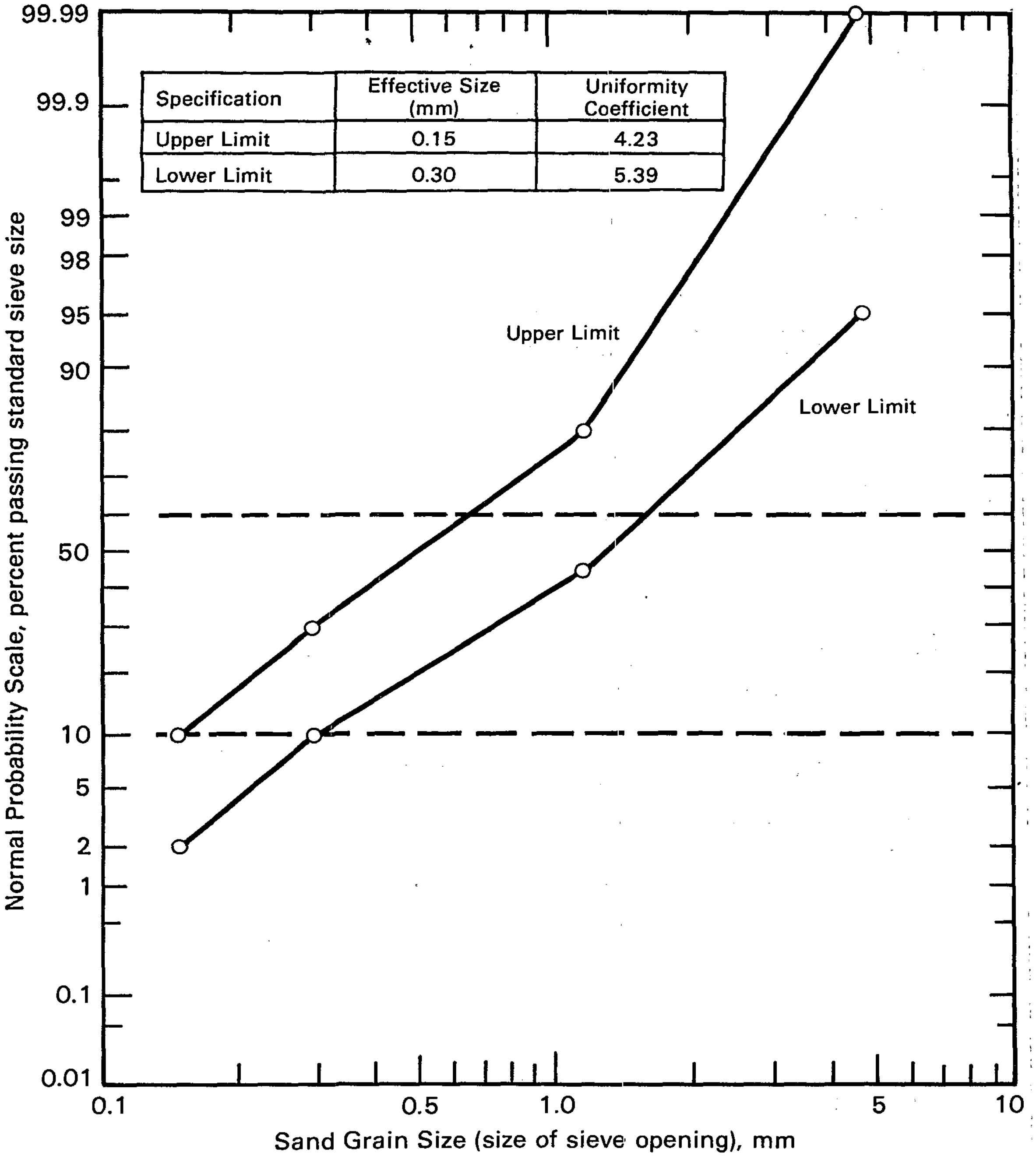
The filter bed consists of the filter sand and the gravel layer between the filter sand and the underdrain system. A cross section of a typical intermittent sand filter is shown in Figure 5-7.

a. Filter Sand Bed

In general, the filter sand should conform to the e.s. and u criteria outlined above. The sand depth should be sufficient to produce a high quality effluent and also provide a sufficient reserve to allow for several cleaning cycles. It has been reported that at least 45 cm (18 in) of filter sand are required to produce an adequate quality effluent (44). Huisman and Wood (62) recommend at least 70 cm (28 in) of filter sand be provided for slow sand filters

FIGURE 5-6

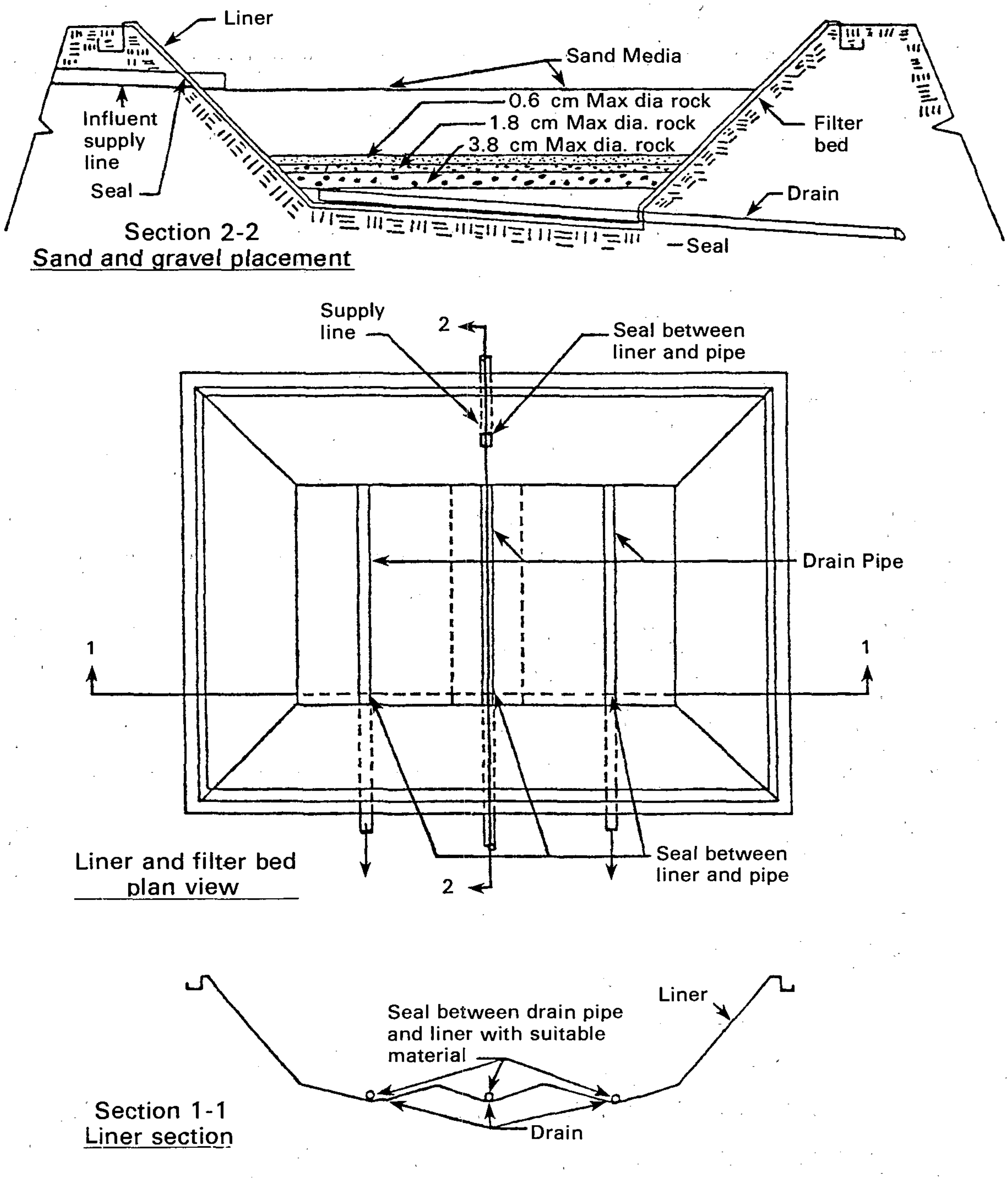
SAND GRAIN SIZE vs SAND SIZE DISTRIBUTION FOR
AASHO M6 SPECIFICATIONS



Specification	Effective Size (mm)	Uniformity Coefficient
Upper Limit	0.15	4.23
Lower Limit	0.30	5.39

FIGURE 5-7

CROSS SECTION OF A TYPICAL INTERMITTENT SAND FILTER



treating potable water. Research studies using a minimum of 60 cm (24 in) of filter sand have produced a high quality effluent (45-48)(60). Based upon the above data, it appears that an intermittent sand filter should not be operated with less than 45 cm (18 in) of filter sand.

In addition, sufficient sand for at least one year of cleaning cycles should be provided. Approximately 2.5 to 5 cm (1 to 2 in) of sand are removed during each cleaning cycle. If the anticipated length of filter run is 30 days, at least an additional 30 cm (12 in) of filter sand should be provided. In general, the total initial depth of filter sand employed on intermittent sand filters is 90 cm (36 in).

b. Gravel Bed

In general, a graded gravel layer 30 to 45 cm (12 to 18 in) in depth separates the filter sand from the underdrain system. If the underdrain system consists of perforated pipe as opposed to a conventional tiled or block underdrain system, the gravel will completely enclose the drain pipe. The gravel system is built up of various layers, ranging from fine at the top to coarse at the bottom. The layers are designed to prevent filter sand from entering and plugging the underdrain system. In general, the bottom gravel layer should consist of particles with an e.s. at least four times greater than the openings into the underdrain system. Each successive layer should be graded so that its e.s. is not more than four times smaller than that of the layer immediately below.

The research on polishing pond effluents at Logan, UT, was conducted with a gravel bed 30 cm (12 in) deep composed of three 10-cm (4-in) layers (45-47). The bottom layer consisted of gravel ranging from 1.9 to 3.8 cm (0.75 to 1.5 in) in diameter, the middle layer consisted of 1.3 to 1.9 cm (0.5 to 0.75 in) diameter gravel, and the top layer consisted of 0.32 to 0.64 cm (0.12 to 0.25 in) diameter gravel. This arrangement has proven to be satisfactory.

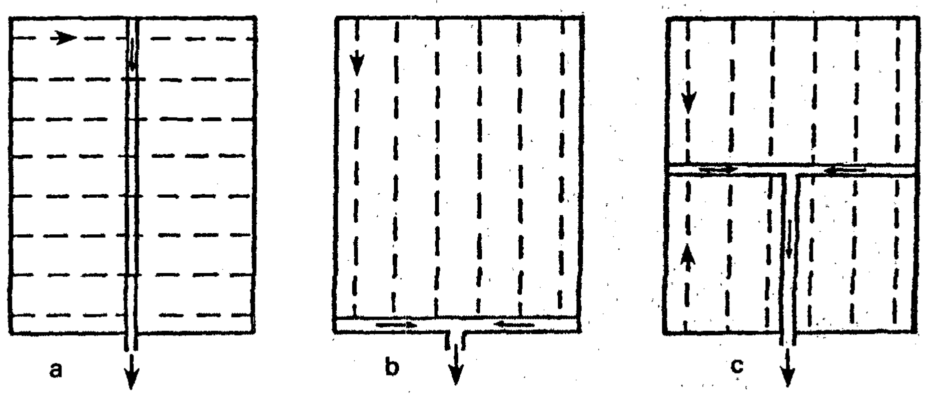
c. Underdrain Systems

The underdrain system may consist of porous or perforated unglazed drainage tiles, glazed pipes laid with open joints, or perforated asbestos or polyvinylchloride (PVC) pipe (62). Experience has indicated that a corrugated, perforated PVC pipe (similar to that used for irrigation drainage) provides an adequate drain system for minimum cost. If drain pipes are used, they should be placed within the bottom gravel layer of the filter to collect the flow as it infiltrates through the sand. Filter bottoms should slope toward the drainage pipe for efficient collection of effluent.

Typical drain schemes are shown in Figure 5-8. The drain pipes should be laid with sufficient slope to produce "scour velocity" in the pipes under average flow conditions. This will allow the drain system to be self-cleaning and thus reduce maintenance costs.

FIGURE 5-8

COMMON ARRANGEMENTS FOR UNDERDRAIN SYSTEMS (62)



The drains should be designed so that they are fully exposed to the air. It is imperative that air be able to circulate through the drain system into the sand filter bed. Intermittent sand filters are an aerobic system and thus must have an adequate air circulation if they are to perform properly. Outlet drains should be exposed to the atmosphere and not submerged.

5.3.1.8 Influent System

The influent system may be gravity flow, or utilize pumps or automatic dosing siphons depending on the configuration of operation and the topography of the site. When a relatively large amount of slope is not associated with the filter site, series intermittent sand filtration operations will require a pump lift station between filters to lift the effluent from one filter to the influent level of the next filter in the series.

The influent system should be designed with sufficient capacity to enable the total daily hydraulic load to be applied to the filter in less than 6 hr. For filters less than 90 m^2 (1,000 ft^2) in area, the daily hydraulic load may be applied to the filter in less than 1 hr for a relatively small investment. For larger filter systems, the dosing time should be accomplished in less than 6 hr. This method of operation allows the maximum head buildup on the filter and also, because the influent will drain through the filter quickly, the maximum bed aeration time is achieved. Influent velocities should be sufficient to prevent settling of solids in the lines.

The influent distribution system need not be complicated or elaborate. Even distribution of the influent across the filter surface will be accomplished by the water buildup on the filter caused by the short dosing time (i.e., less than 6 hr). Water may not be distributed evenly across the entire filter at first, but within a short time after loading begins the water will be standing several centimeters deep over the entire filter surface.

Simple channels which overflow at regular intervals across the filter bed will provide adequate distribution of the influent. Discharge velocities from these channels onto the filter surface should be small enough to prevent serious sand erosion. Splash pads may be necessary in some cases to reduce sand erosion near inlet structures. In addition, inlet channels should be equipped with drains or "weep holes" so that they may drain completely dry during periods of freezing temperatures. This should prevent the buildup of ice within the influent channels.

The influent system should be fully automated so that pumps, siphons, or transfer structures may be activated routinely without continual personal supervision. Provisions should be made so that the system can be operated either during day or night. Flexibility should be designed into the system so that each filter may operate independently. Series intermittent filtration influent systems should be constructed so that operation can be either in series or in parallel as a single-stage filtration system. In addition, manual overrides should be provided for all systems in case of power failures. Alternatively, an auxiliary power source should be available. Spare pumps and metering systems should be provided. In general, the same degrees of flexibility should be provided for a pond system as that found in a conventional treatment plant.

5.3.1.9 Filter Walls

Filter walls may be constructed as earthen embankments or from conventional building materials such as concrete, steel, or wood. In general, earthen embankments are much more economical, especially for larger filters, and are recommended for most designs. Steel or concrete filter walls may be applicable for small filters where embankment materials are not readily available or space is limiting.

When materials other than earthen embankment are employed for filter wall construction, care must be exercised to prevent "short-circuiting," the downward percolation of water along the inner wall face without passing through the filter bed. Short-circuiting may cause deterioration of effluent quality and is a particular problem in small sand filters. Structural precautions should be taken to guard against it. It is no problem with sloping walls or earthen wall embankments because the sand tends to settle tightly against these types of walls. However, with smooth vertical walls, it may be necessary to incorporate devices such as built-in grooves or artificial roughening of the internal surface. The most effective precaution is to give the walls a slight outward batter, so as to obtain the advantages of a sloping wall (62).

Earthen filter embankment construction should be similar to that used in a normal well-designed pond system. Embankments are usually designed with side slopes from 6:1 to 2:1 with 3:1 being the most common. Embankment top width should be at least 3 m (10 ft) and provide a 30-cm (1-ft) thick all-weather gravel road. Road surfaces should be crowned to assure rainwater runoff and minimum erosion.

The interior embankment should be impervious to prevent both exfiltration of filter influent and infiltration of seepage groundwater. Most state regulatory agencies have a standard for maximum seepage losses. Figure 5-7 illustrates the use of a liner to create an impervious embankment. Such liners are only economically feasible on relatively small intermittent sand filtration systems. In general, an impervious clay layer or similar material used to seal the pond system would be sufficient to seal the filter embankment.

Interior slopes should also be designed to prevent erosion due to wave action. Erosion protection can be provided by cobbles, broken or cast-inplace concrete, wooden bulkheads, or asphalt strips. Emphasis should be placed on shoreline control and reduction of aquatic weed growths. In addition, each filter should be provided with a ramp for easy access and routine maintenance of the system. The ramp can be used for both entry of cleaning equipment and as a boat ramp.

Embankments should provide at least 30 cm (1 ft) of head on the filter and 0.5 to 1 m (1.6 to 3.3 ft) of freeboard to prevent wave action from washing over the dike.

5.3.1.10 Sand Cleaning

An intermittent sand filter is considered to be plugged when the amount of water applied to the filter will not percolate through the sand bed before the next dose is applied. When the filter is plugged, it is taken out of service and the top 2.5 to 5 cm (1 to 2 in) of sand are removed or scraped from the filter surface.

Huisman and Wood (62) present a review of the current practice of sand washing associated with slow sand filters employed in potable water treatment. The design engineer should become thoroughly familiar with the current practice of sand reclamation before proceeding with the design of intermittent sand filters. Sand from an intermittent sand filter may be washed by conventional means, used as a soil conditioner, or disposed of in a landfill. In most cases, economic considerations will dictate that the sand be cleaned and reused rather than discarded. A typical sand washing device is shown in Figure 5-9. Basically, these sand washing devices consist of an upflow clarifier with sufficient velocity for removal of organic matter, without washing away the sand. The organic matter washed from the sand may be recycled back through the pond system. Once the sand has been cleaned, it can be stockpiled and eventually recycled to the filter.

Whether to dispose of or reuse the spent filter sand is largely dependent on the local availability of the filter sand. When sand costs are high, the removed sand should be stockpiled, washed, and recycled. Storage of the sand, in layers approximately 30 cm (1 ft) in depth and washing with 20 cm (8 in) or more of clean water, has successfully refurbished used sand on an experimental basis (65). Consideration of this approach appears particularly attractive in wet climates. It is also possible that filter effluent could be used to clean the sand by this technique.

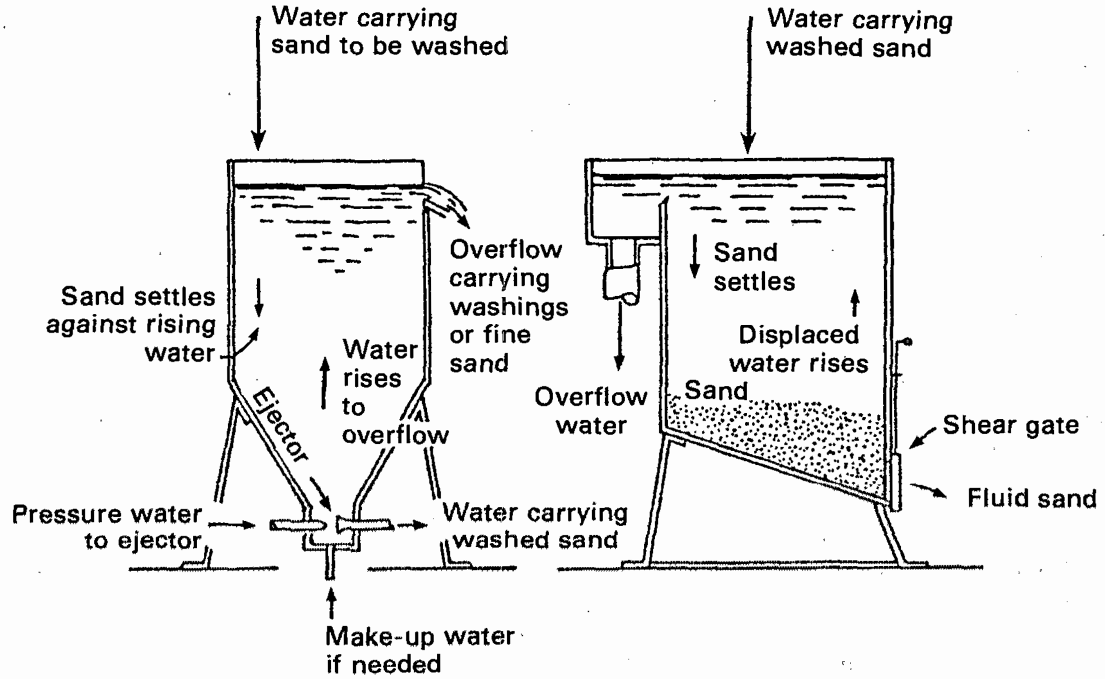
The sand washing equipment should be sized to accommodate the sand washing over several days at a time. Thus, the equipment may be smaller than required for immediate sand washing.

The use of the spent sand as a soil conditioner has been investigated on a limited basis (65). The sand is rich in nutrients and organic matter, and appears to be a good conditioner, especially for clay soils. If the pond effluent is high in heavy metals concentrations, use of the spent filter sand as a soil conditioner may be restricted since these metals may precipitate from the sand particles.

The disposal of the spent filter sand in a land disposal site may be economically feasible for very small sand filters where replacement filter sand is readily available. However, a careful economic evaluation should be conducted before complete sand disposal is practiced. In addition, there may

FIGURE 5-9

TYPICAL UPFLOW SAND WASHER AND SAND SEPERATOR UTILIZED IN WASHING SLOW AND INTERMITTENT SAND FILTER SAND (63)



(A)
Sand washer

(B)
Sand separator

be regulatory agency restrictions against placing the spent filter sand in landfills due to possible leaching of heavy metals and other toxic materials.

The filter media and the filtered matter provide an excellent environment for weeds and grass to grow. Weed control during the growing season is achieved by complete weed removal using manual labor or with mechanical raking devices. The best method of weed control is continuous monitoring and removal of early growth.

When a filter is observed to be plugged or approaching plugged conditions, it is necessary to rejuvenate the filter surface. Two approaches are available. The first method consists of raking the media surface and breaking the surface mat of filtered matter. Raking makes the cleaning process more economical by obtaining optimum use of the media surface prior to removal. Raking can be accomplished manually with a garden rake or with a tractor and a landscape rake. A maximum of two rakings before cleaning is recommended.

When the raking process no longer rejuvenates the filter surface due to the accumulation of the filtered matter in the top layer of the media, removal of the solids laden layer is necessary. The removal process can be performed manually or with mechanical devices. A four-wheel-drive garden size tractor equipped with a hydraulically operated scraper, bucket loading device, and either flotation-type tires or dual rear tires works well (59). A small floating dredge has been proposed as a means of cleaning a large [4-ha (10-ac)] system in Wyoming (66). Heavy equipment such as road graders or heavy front loaders should not be used on the filter.

5.3.1.11 Winter Operations

Winter operations are basically the same as summer operations except that cleaning of the filters during the winter is far more difficult. The cold season should be started with clean filters and in most instances the filters will operate through the cold weather without a cleaning being required. The system must be designed to prevent the accumulation of water in the filter underdrain to a depth near the media surface where freezing can occur.

Operation of experimental intermittent sand filters at Logan, UT, during freezing and sub-zero temperatures was investigated by Harris et al. (45-47). Experience with these experimental and full-scale systems indicates that the systems operated without any preparation of the sand surface. If severe water conditions are expected, the most economical method for winter operation appears to be the "ridge and furrow" technique.

The ridge and furrow technique requires that the sand filter surface be plowed into small ridges and furrows. The ridges are spaced approximately 0.6 to 1.0 m (24 to 40 in) apart with 30 to 45 cm (12 to 18 in) deep furrows in between each ridge. The basic idea of the ridge and furrow technique is to allow the formation of a floating ice cover which will settle on the peaks of the ridges as the water percolates through the sand filter bed. The furrows allow the influent water to run under the settled ice cover during the filter hydraulic loading period and thus float the ice cover. Occasionally, the ice cover may break up due to its own weight as it rests upon the ridges. This also allows the influent water to infiltrate through the sand surface.

For very small filters it may be economically feasible to construct an insulated cover over the filter for winter operation. With such a covered structure, it might also be feasible to provide auxiliary heat to prevent freezing. In severe climates with long freezing periods, insulated filter covers may be essential for winter operation.

Covered filters also prevent algal growth on the filter surface during summer periods. Thus, the length of filter runs may be substantially increased.

5.3.1.12 Operational Modes

Length of filter runs may be increased if the filter influent SS are low. It may be advantageous, therefore, to hold pond effluents during high algal periods and discharge during periods of low algal growth (i.e., early spring and late fall). These periods may not result in a high quality of pond effluent in terms of BOD_5, but the filter is capable of significantly reducing the BOD_5. Loading the filters at night rather than during the day has also increased length of filter runs. This is due to the reduction of algal growth on the filter itself during dark hours.

5.3.1.13 Summary of Design Considerations

A summary of intermittent sand filter criteria is presented in Table 5-5.

5.3.1.14 Typical Design of Intermittent Sand Filter

a. Assumptions

1. Design Flow = 378.5 m^3/d (0.1 mgd).

2. Hydraulic Loading Rate (HLR) = 0.29 $m^3/m^2/d$ (0.3 mgad).

3. Minimum Number of Filters = 2 (Table 5-5).

4. Designed to minimize operation and maintenance.

5. Gravity flow.

6. Topography and location are satisfactory.

TABLE 5-5

SUMMARY OF INTERMITTENT SAND FILTER DESIGN CRITERIA

Design Topic	Typical Description
Hydraulic Loading Rate	Equal to or less than 0.47 $m^3/m^2/d$ using two or more equal dosings per day.
Filter Size/Number	Minimum of two filter units. Area of individual filters $\leq$0.4 ha.
Filter Shape	Dependent upon site plan and topography with rectangular shape desirable to improve distribution of wastewater.
Depth of Filter Media	Large gravel-minimum cover of 10 cm (leveled), medium gravel - 10 cm, pea gravel - 10 cm , filter sand - 0.6-1.0 m.
Size of Filter Media and Underdrain Media	Large gravel (avg. dia. = 3.3 cm), medium gravel (avg. dia. = 1.9 cm), pea gravel (avg. dia. = 0.64 cm), sand (0.15 mm to 0.30 mm e.s., u <7).
Filter Containment	Compacted earthen bank of reinforced concrete; freeboard $\geq$0.5 m.
Influent Distribution	Dosing basin with siphon or electrically actuated valves with timer control and piping to gravel splash pads. Splash pad gravel should be 3.8-7.6 cm in diameter and surface area and depth should be 1.2 m^2 and 25 cm.
Underdrain System	Network of clay tile or perforated PVC pipe at a slope of 0.025 percent serve as laterals. Pipes are placed in sloped ditches and attached to larger drain manifolds. Minimum lateral size is 15 cm in diameter and manifold should be adequate to transport design flowrate at a velocity of 1.0-1.2 m/s when flowing full. Maximum spacing of laterals is 1.4 m.
Maintenance Considerations	Grass encroachment, rodent activity, serviceability, access to filter by cleaning devices.
Maintenance Required	Removal of vegetation on filter surface. Raking and cleaning of top 2-5 cm of filter sand when plugged.
Cleaning Frequency	Dependent on hydraulic loading rate and the SS concentration in the applied water (1 month to >1 year).
Method of Cleaning	Raking maximizes the efficiency of the cleaning by fully utilizing the top layer of sand. Manual or mechanical equipment cleaning can be used.

7. Adequate land is available at reasonable cost.

8. Filter sand is locally available.

9. Filters are considered plugged when, at the time of dosing, the water from the previous dose has not dropped below the filter surface.

b. Calculate Dimensions of Filters

$$\text{Area of Each Filter} = \text{design flow/HLR}$$

$$= (378.5\ m^3/d)/(0.29\ m^3/m^2/d)$$

$$= 1{,}357\ m^2\ (14{,}800\ ft^2)$$

$$\text{Letting } L = 2W,\ \text{Area} = 2W^2$$

$$W = (\text{Area}/2)^{0.5}$$

$$= (1{,}357/2)^{0.5}$$

$$= 26\ m\ (86\ ft)$$

$$L = 2W$$

$$= 2(26)$$

$$= 52\ m\ (172\ ft)$$

Construct two filters 26 m x 52 m (86 ft x 172 ft) side-by-side as shown in Figure 5-10.

c. Influent Distribution System

Assumptions:

1. Use of dosing basin with gravity feed to the filters.

2. Loading sequence will deliver one-half the daily flowrate to each filter unit per day in two equal doses.

3. Loading system will consist of two electrically activated valves that are operated alternately by a simple electronic control system triggered by a float switch or two alternating dosing siphons.

4. Pipe sizes are selected to avoid clogging and to make cleaning convenient. Hydraulics do not control.

Dosing Basin Size:

$$\text{Design Flow Rate} = 378.5\ m^3/d/2\ \text{filters}$$

$$= 189.3\ m^3/d\ (0.05\ \text{mgd})$$

$$\text{Dosing Basin Volume} = 189.3\ m^3/d/2\ \text{doses}/d$$

$$= 95\ m^3\ (25{,}000\ \text{gal})$$

Use a square shape and a water depth of 1.0 m (3.3 ft) to minimize velocity in distribution system. Use 0.3 m (1 ft) freeboard and install overflow pipe. Total depth = 1.3 m (4.3 ft).

$$\text{Dosing Basin Area} = \text{volume/depth}$$

$$= 95\ m^3/1.0\ m$$

$$= 95\ m^2\ (1030\ ft^2)$$

$$\text{Dosing Basin Width} = (95\ m^2)^{0.5}$$

$$= 9.75\ m\ (32.2\ ft)$$

Dosing Basin Size = 9.75 m (32.2 ft) square x 1.3 m (4.3 ft) deep

Distribution manifold from the two valves leading to the individual filters would be 20-cm (8-in) diameter pipe. Each of the outlets from the manifold will serve 6 m (20 ft) of the long side of the filter unit. The manifold outlets will discharge onto splash pads constructed of gravel 3.8 to 7.6 cm (1.5 to 3 in) in diameter placed in a 75-cm (2.5-ft) square configuration at each outlet opening.

d. Filter Containment and Filter Underdrain System (See Figure 5-10)

Use a reinforced concrete retaining structure or a 20-mil plastic liner to prevent infiltration and exfiltration to adjacent groundwater.

Slopes of filter bottom are dependent on drain pipe configuration using 0.025 percent slope with lateral collection lines 4.6 m (15 ft) on center.

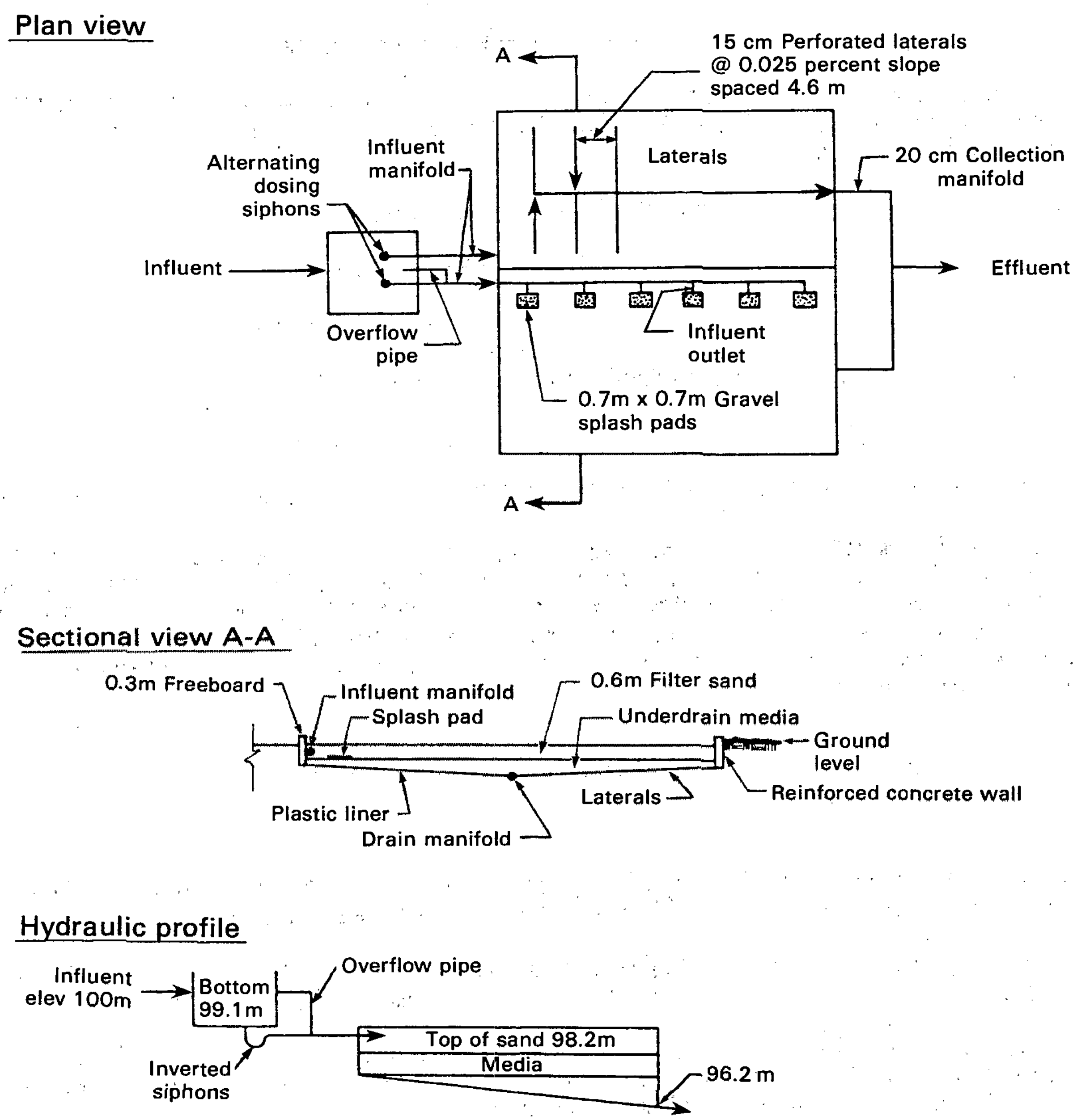
Utilization of 15-cm (6-in) diameter perforated PCV pipe as collecting laterals and 20-cm (8-in) diameter collection manifolds will provide adequate hydraulic capacity and ease of maintenance.

Minimum Freeboard Required:

$$\text{Depth} = \text{volume/area} = 95\ m^3/1{,}357\ m^2 = 7.0\ cm\ (2.8\ in)$$

FIGURE 5-10

PLAN VIEW, CROSS SECTIONAL VIEW, AND HYDRAULIC PROFILE FOR INTERMITTENT SAND FILTER



Minimum freeboard required to accommodate wastewater when filter is plugged at time of application of the dose is 7.0 cm (2.8 in). However, with infrequent inspection by an operator it is recommended that a safety factor be specified and the value of 30 cm (1 ft) mentioned above be used.

5.3.1.15 Summary of Design Criteria for Existing and Planned Filters

A summary of the design criteria and costs associated with existing and proposed intermittent sand filters used to upgrade pond effluent is presented in Table 5-6.

5.3.2 Rock Filtration

A rock filter operates by allowing pond effluent to travel through a submerged porous rock bed, causing algae to settle out on the rock surface and into the void space. The accumulated algae are then biologically degraded. Algae removal with this system filter has been studied extensively at Eudora, KS, beginning in 1970 (67)(68).

Two experimental rock filters at Eudora used a submerged rock depth of 1.5 m (5 ft) and rock of 1.3 cm (0.5 in) in one and 2.5 cm (1.0 in) in the other filter (68). Influent to the filter submerges the rock bed, which is contained in a diked area, and effluent is drawn off at the bottom of the bed. The period of peak efficiency of the rock filter is in the summer and early fall and hydraulic loading can be increased in this period. The filters at Eudora were operated at loading rates up to 1.2 $m^3/m^3/d$ (9 gpd/ft^3) in the summer and this loading was decreased to 0.4 $m^3/m^3/d$ (3 gpd/ft^3) in the winter and spring. Tests on pond effluent having a BOD_5 of 10 to 35 mg/l and SS level of 40 to 70 mg/l showed that the rock filter reduced BOD_5 by only a relatively small amount (however, the final concentration was always below 30 mg/l) and would reduce SS to 20 to 40 mg/l (Figures 5-11 and 5-12). It was concluded that the rock filter could be operated to meet effluent requirements of 30 mg/l BOD_5 and it was doubtful that the filter could consistently reduce the SS to 30 mg/l. It was postulated that the filter would not become plugged for more than 20 years. One drawback is the production of hydrogen sulfide during the summer and early fall when the filter becomes anaerobic. Aeration of the effluent would be required prior to discharge.

A rock filter was constructed at California, MO, in 1974 to upgrade an existing pond (67). This was placed along one side of a tertiary pond as illustrated in Figure 5-13. The rock filter was designed for a hydraulic loading rate of 0.4 $m^3/m^3/d$ (3 gpd/ft^3). In 1975, a 757 m^3/d (0.2 mgd) rock filter was constructed in Veneta, OR. In 1977 and 1978, extensive monitoring programs were conducted by Oregon State University to determine removal mechanisms and efficiency of this filter (69)(70). A schematic of the Veneta system is presented in Figure 5-14.

TABLE 5-6

SUMMARY OF DESIGN CRITERIA AND COSTS FOR EXISTING AND PLANNED INTERMITTENT SAND FILTERS USED TO UPGRADE POND EFFLUENT (59)

Location	Design Flow	Pond Type[a]	Retention Time	Filters		Loading Rate	e.s.	u	Sand Depth	Filter Costs			
				Number	Size					Capital	O&M	Total	Year
	l/s		days		ha	$m^3/m^2/d$	mm		m	$/m³ flow			
Covello, CA	3.5	F	49	4	0.048	0.48	0.6-0.7	NA	0.6	0.11	--	--	1977
Mt. Shasta, CA	0.7L[b]	A	20L[b]	3	0.405	0.67	0.37	5.1	0.6	0.05	0.01	0.06	1976
Tomales, CA	1.8	A	29	2	0.012	0.55	0.15-0.30	1.5-2.5	0.9	0.07	--	--	1978
Cimarron, NM	6.6	F	55	2	0.04	0.76	NA	NA	0.6	--	--	--	1975
Cuba, NM	6.1	A&F	20	4	0.02	0.57	NA	NA	0.6	0.04[c]	--	--	1976
Moriarty, NM	8.8	A&F	20	8	0.028	0.57	0.2	4.1	0.6	0.03	0.01	0.04	1976
Portalles, NM	87.8	A&F	20	3	0.405	1.91	0.4	3.2	0.8	--	--	--	1976
Roy, NM	2.5	F	60	2	0.028	0.72	0.4	3.2	0.6	0.01	--	--	1976
Adel, GA	48.3	A	30	2	0.405	0.48	0.25	3.4	1.2	0.02	--	--	1978
Ailey, GA	3.5	F	70	2	0.06	0.38	0.25	3.3	0.8	0.05	0.01	0.06	1976
Cummings, GA	8.8	A&F	36	4	0.06	0.29	0.25-0.80	<4	0.7	0.03	--	--	1978
Douglas County, GA													
School	0.6	F	5	2	0.008	0.29	0.3	3.67	0.8	--	--	--	1976
Nursing Home	1.5	F	45	2	0.012	0.48	0.35-0.75	<3.5	0.8	0.04	--	--	1978
Shellman, GA	6.6	F	55	4	0.032	0.48	0.35-0.75	<3.5	0.9	0.01	--	--	1979
Stone Mountain, GA	0.9	F	1	1	0.081	0.14	0.45-0.55	±1.5	0.8	--	0.03		1976
Huntington, UT	13.2	F	214	3	0.271	0.19	0.2-0.25	<3	1.2	0.04	--	--	1976

[a]A = Aerated.

F = Facultative.

[b]L = Dry Weather Flow.

[c]Twenty percent of total capital cost.

FIGURE 5-11

BIOCHEMICAL OXYGEN DEMAND (BOD_5) PERFORMANCE OF LARGE ROCK FILTER AT EUDORA, KANSAS (68)

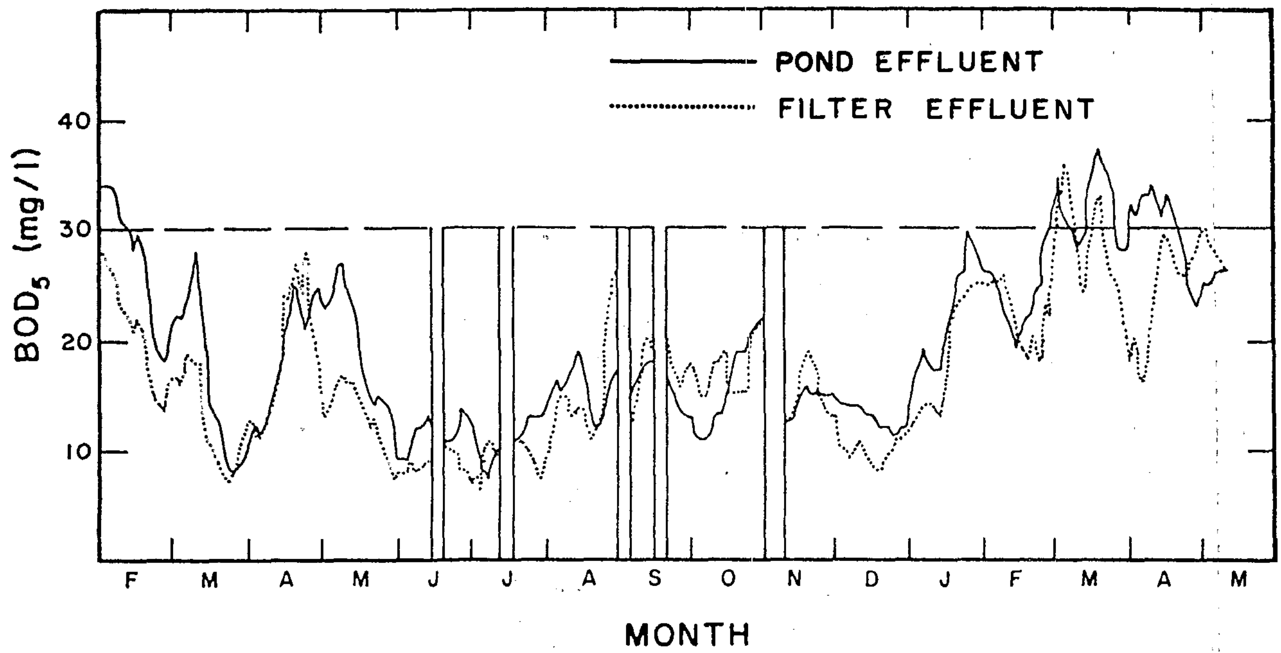


FIGURE 5-12

SUSPENDED SOLIDS PERFORMANCE OF LARGE ROCK FILTER AT EUDORA, KANSAS (68)

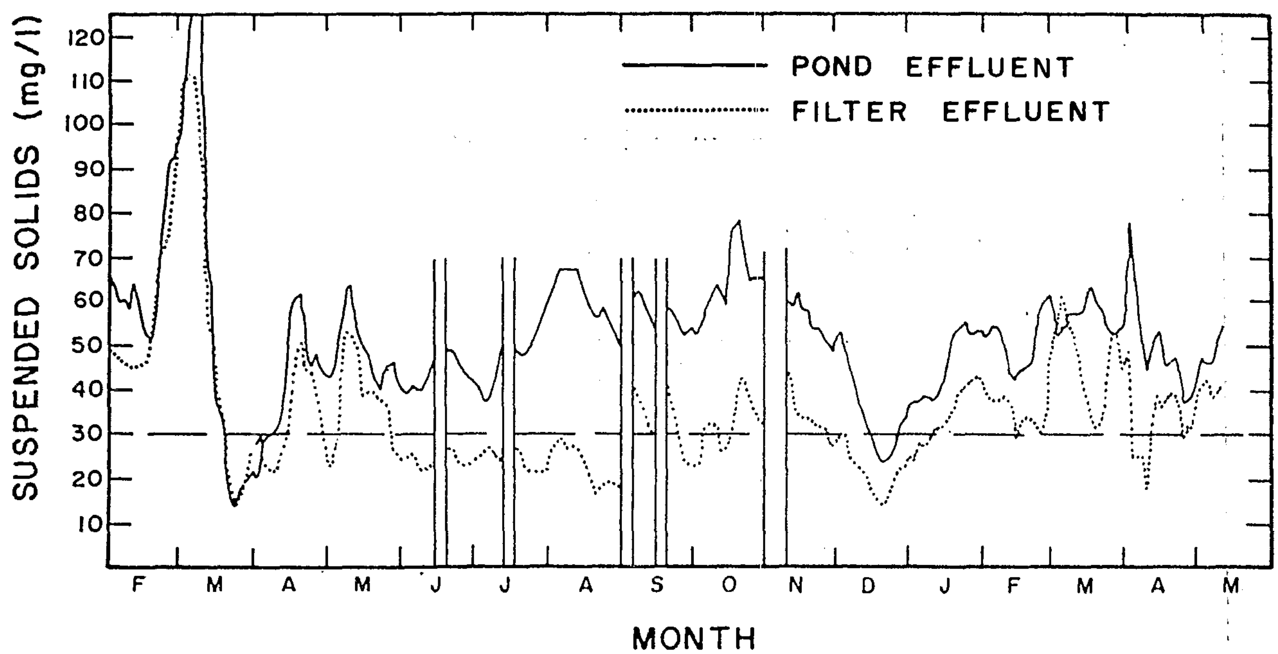
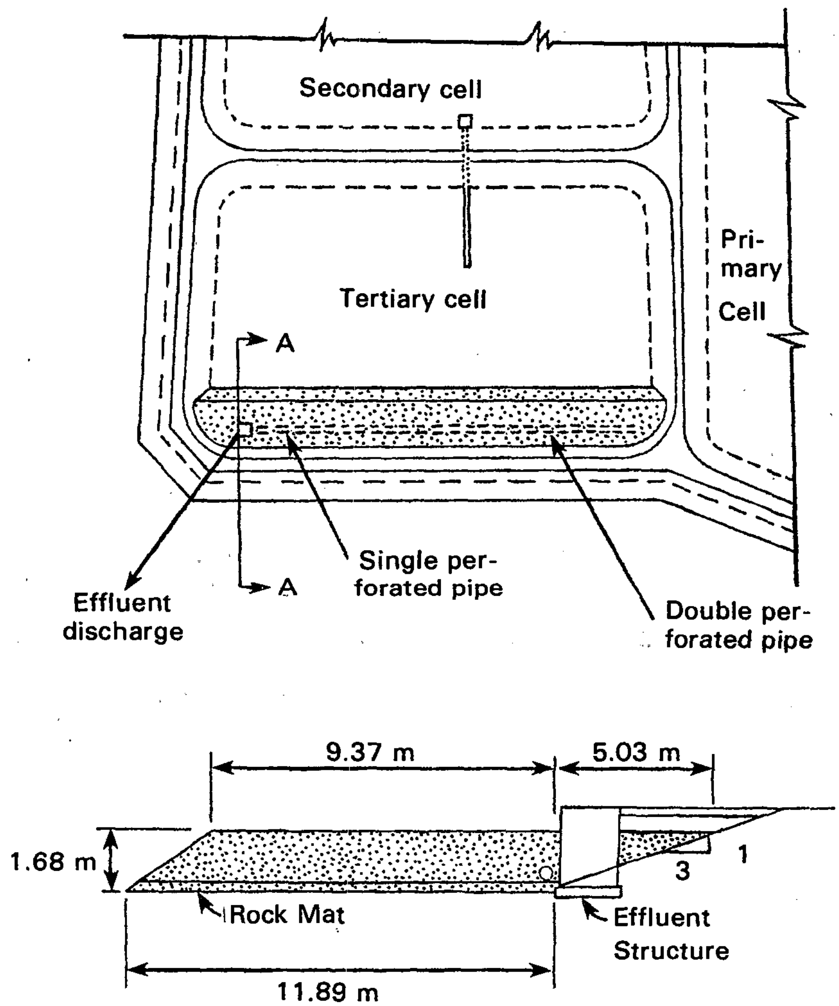


FIGURE 5-13

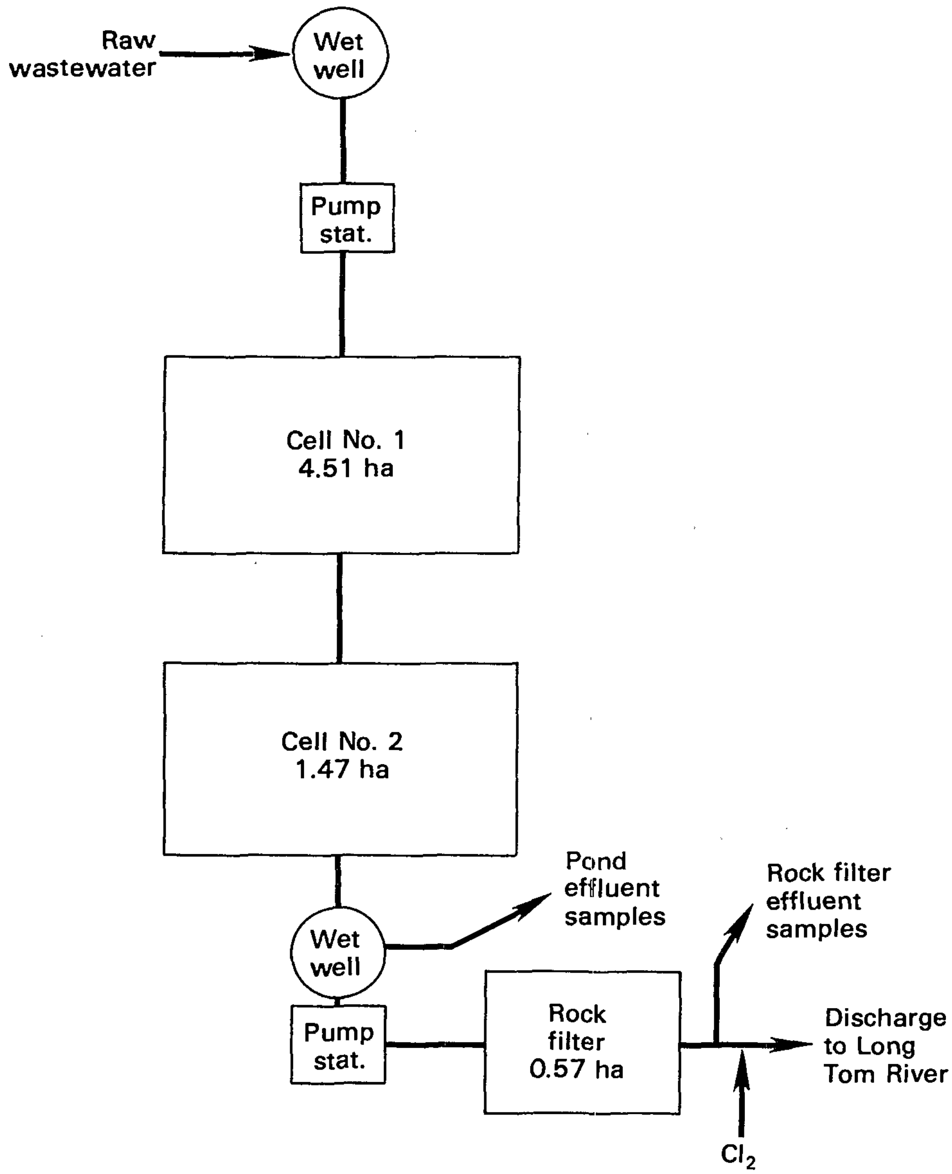
ROCK FILTER INSTALLATION AT CALIFORNIA, MISSOURI (67)



Section A-A

FIGURE 5-14

SCHEMATIC FLOW DIAGRAM OF VENETA, OREGON WASTEWATER TREATMENT SYSTEM (69)



A performance evaluation of the California, MO, rock filter was conducted during March and April 1975 (67). The results of that evaluation indicated that the actual average hydraulic load on the filter was 0.25 $m^3/m^3/d$ (1.9 gpd/ft^3). A summary of the rock filter performance is reported in Table 5-7. During the evaluation period, the rock filter average effluent BOD_5 concentration was only 21 mg/l. The rock filter average influent SS concentration was 69 mg/l while the rock filter average effluent SS concentration was 22 mg/l.

TABLE 5-7

PERFORMANCE OF ROCK FILTER AT CALIFORNIA, MISSOURI (67)

	BOD_5		SS		DO	
	Influent	Effluent	Influent	Effluent	Influent	Effluent
	mg/l		mg/l		mg/l	
3/5/75	15	8	35	24	19	6.6
3/13/75	14	6	64	26	16.8	7.9
3/19/75	19	7	80	24	--	12.9
3/26/75	25	13	94	20	12.9	5.2
4/2/75	30	15	74	16	13.3	4.6
Average	21	12	69	22	16	7.4

A routine monitoring program for the California, MO, rock filter was initiated by the Missouri Department of Natural Resources (71) in March 1975, and the results of this work are summarized in Figure 5-15. All solids determinations were made on grab samples. The performance of the rock filter was sporadic and failed to meet the federal discharge standard of 30 mg/l of SS in the effluent on 11 of the 19 sampling dates.

The Veneta rock filter (Figure 5-16) is of a different design (69)(70). Influent enters the rock filter through a pipe laid on the bottom and running through the center of the filter. The water rises through 2 m (6.6 ft) of rock, 7.5 to 20 cm (3 to 8 in) in diameter, and is collected in effluent weirs on the sides of the rock filter. The Veneta rock filter can consistently meet daily maximum effluent limits of 20 mg/l BOD_5 and 20 mg/l SS for hydraulic loadings of 0.3 $m^3/m^3/d$ (2.2 gpd/ft^3) (Figure 5-17). The relationship between the SS removal and the hydraulic loading rate of the Veneta facility is shown in Figure 5-18.

Principal advantages of the rock filter are its relativley low construction cost and simple operation. Odor problems can occur, and the design life for the filters and cleaning procedures have not yet been firmly established.

FIGURE 5-15

PERFORMANCE OF CALIFORNIA, MISSOURI ROCK FILTER
TREATING POND EFFLUENT (71)

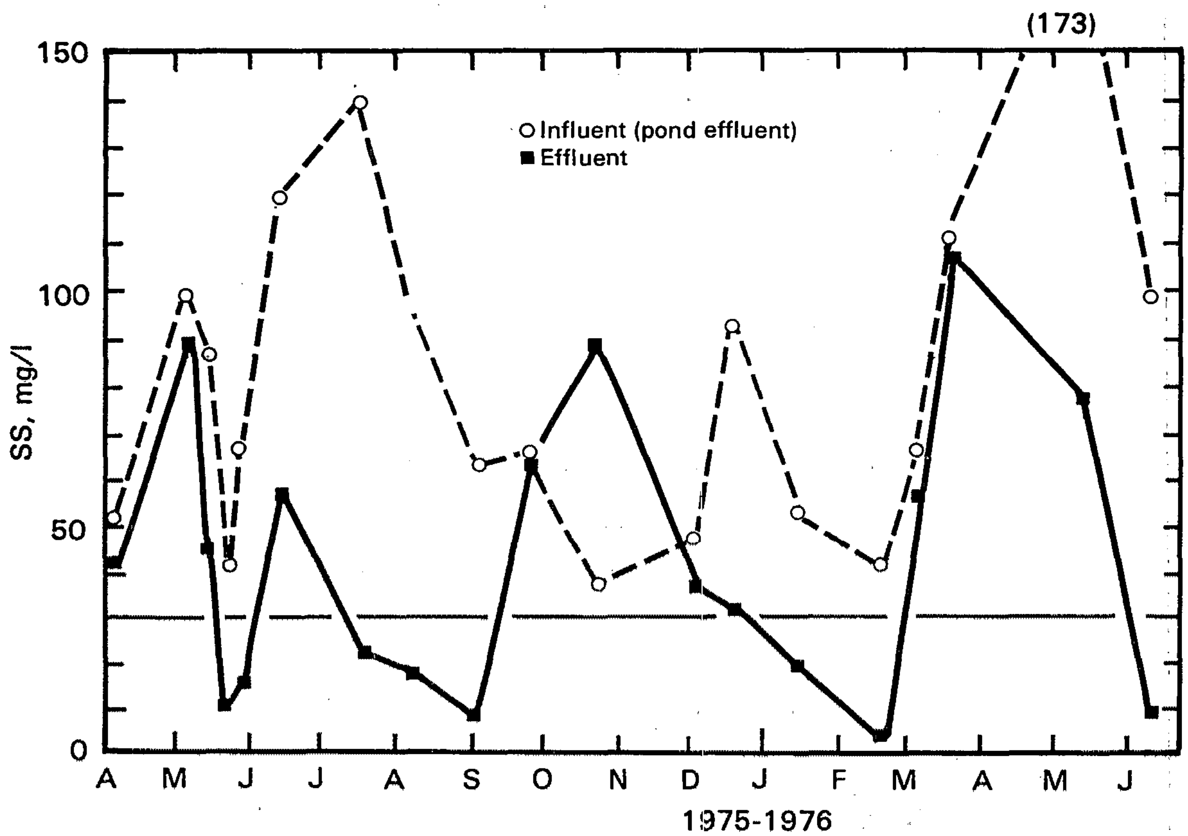


FIGURE 5-16

VENETA, OREGON ROCK FILTER (69)

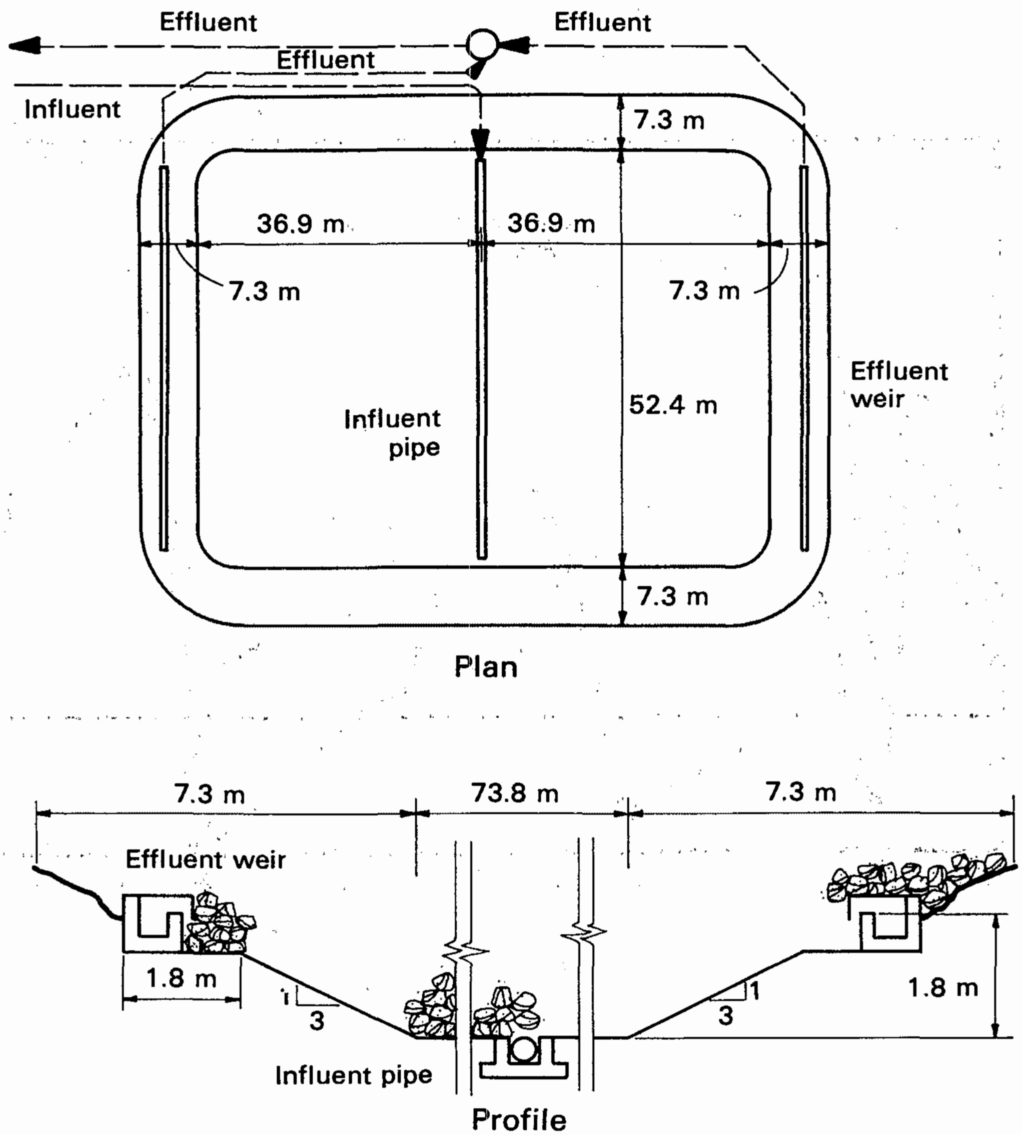


FIGURE 5-17

PERFORMANCE OF VENETA, OREGON ROCK FILTER (69)

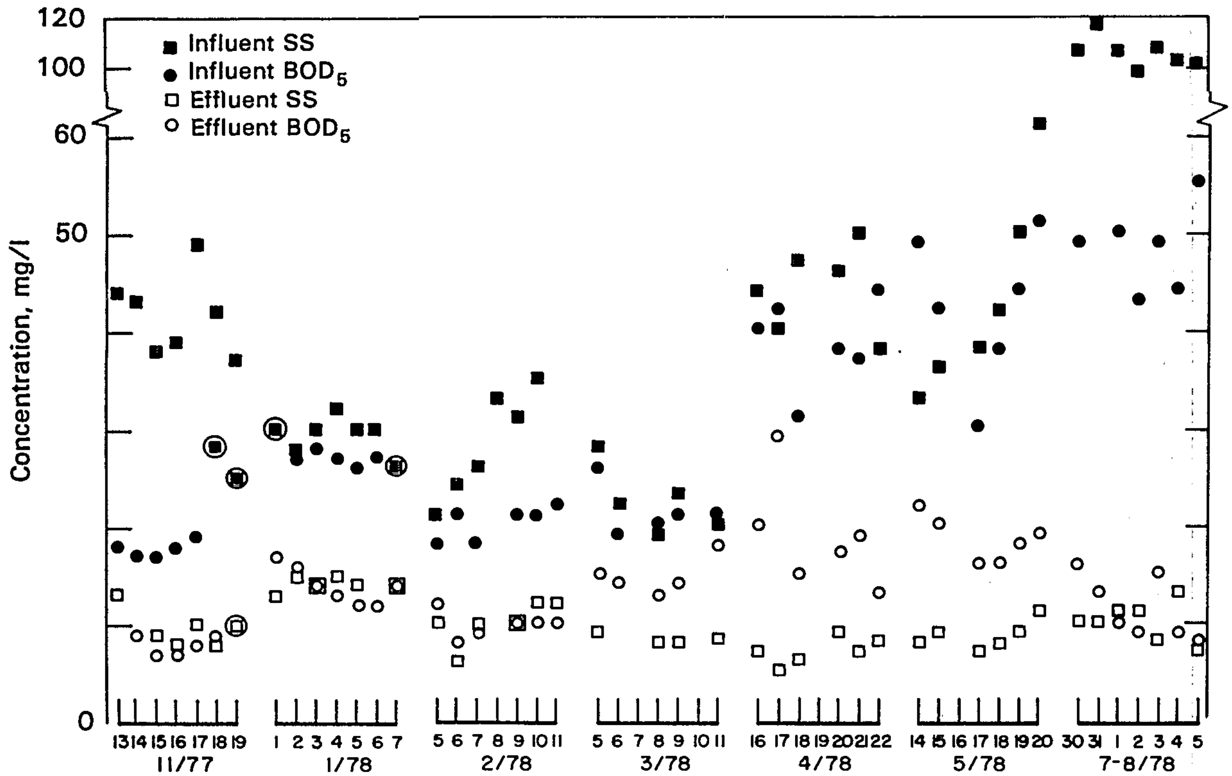
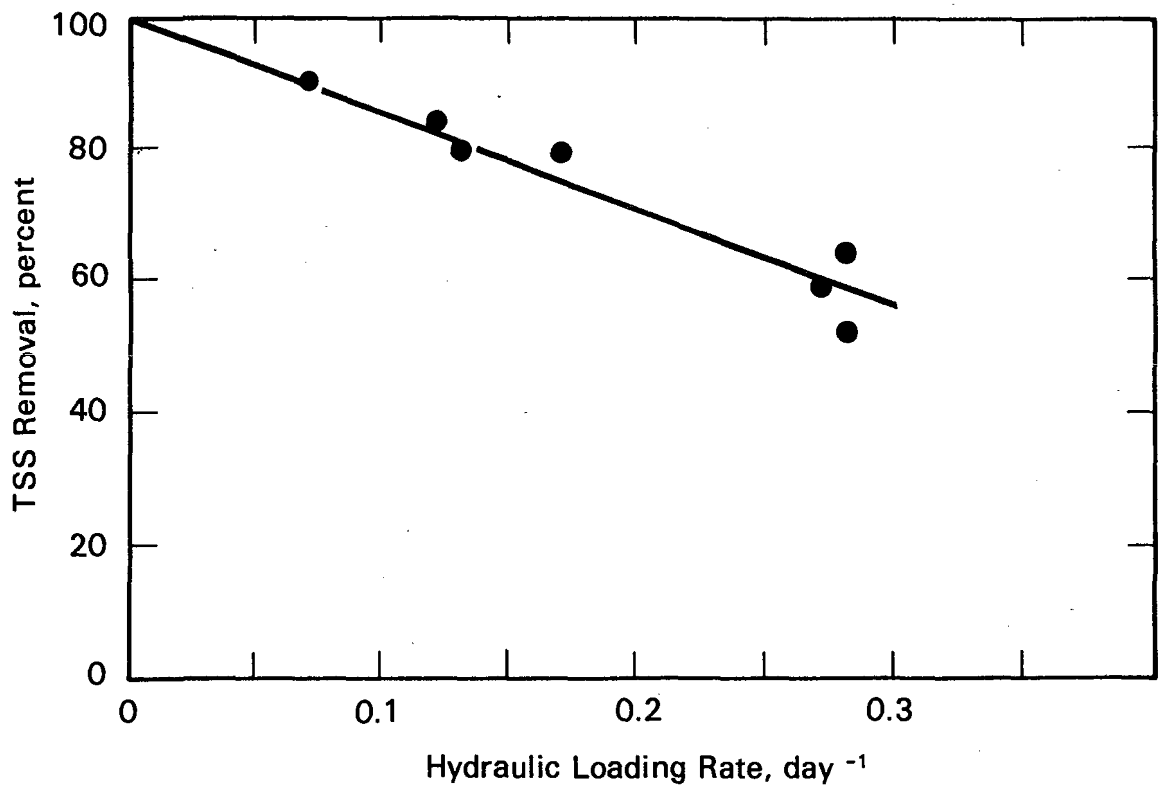


FIGURE 5-18

SS REMOVAL vs HYDRAULIC LOADING RATE AT VENETA, OREGON (69)



5.3.3 Microstrainers

Early experiments with microstrainers to remove algae from pond effluents were largely unsuccessful (72-75). This was generally attributed to the algae being smaller than the mesh size of the microstrainers tested.

Envirex, Inc. has tested a one-micron polyester mesh microstrainer for algae removal from pond effluents. A portable pilot unit was used at 10 locations in 1977 and 1978 (76). The results of seven of these tests are summarized in Table 5-8. Algae removal was best for the larger species of algae; smaller species of algae, such as Chlorella, were removed only when a thin algal mat was maintained on the screen. Slime growth on the screen can be controlled by chlorine without harming the polyester mesh. SS and BOD_5 levels below 30 mg/l were consistently achieved during the test periods.

Unsuccessful tests of one-micron and six-micron screens were reported by Union Carbide (77). Algae samples taken several days before and two weeks after the tests indicated that predominant algae species removed were Aphanizomenon and Lyngbya. These algae both have a width of one to two microns. Heavy rainfall accompanied by a large amount of colloidal material in the pond occurred during the tests and there is some question as to how representative the SS content of the pond was at that time. Addition of polymer in relatively large doses (23 to 65 mg/l) was necessary to produce an effluent SS <30 mg/l. This experience emphasizes the need to identify the algae species to be removed and conduct pilot studies under representative conditions.

The tests represented in Table 5-8 are of relatively short duration. Microstraining performance and reliability at pond sites over an extended period of time has yet to be proved.

The first full-scale microstrainer application to pond effluent, a 7,200 m^3/d (1.9 mgd) unit, was placed in operation at Camden, SC, in December 1981 (78)(79). Typical design criteria include surface loading rates of 90 to 120 $m^3/m^2/d$ (1.5 to 2.0 gpm/ft^2) and head losses up to 60 cm (2 ft) (76). Other process variables include backwash rate and pressure, and drum speed, which are normally determined depending upon influent quality and desired effluent quality. The service life of the screen is reported to be longer than five years; however, actual field experience on the one-micron screen is limited. For a 3 m by 3 m (10 ft x 10 ft) unit it would take 12 to 16 man-hours to replace all the screens (77). The microstrainer portion of the plant at Camden cost $1.5 million and is estimated to cost $71,000/year to operate (1979 US $). Limited performance data are available, but the results indicate that the system can meet an effluent standard of 30 mg/l BOD_5 and SS (78).

5.3.4 High Rate Filtration

Conventional rapid sand or multimedia filters have been used both for direct filtration of algae-laden waters and for polishing filtration, which follows coagulation-clarification.

TABLE 5-8

SUMMARY OF PERFORMANCE OF 1-micron MICROSTRAINER PILOT PLANT TESTS (76)

Test Site	Length of Test hr	Lagoon Effluent SS mg/l	Lagoon Effluent BOD_5 mg/l	Microscreen Effluent SS mg/l	Microscreen Effluent BOD_5 mg/l
Adel, Georgia	60	69	72	9	9
Owasso, Oklahoma	60	58	30	15	15
Greenville, Alabama	50	44	45	12	14
Camden, South Carolina	22	126	38	19	23
Gering, Nebraska	90	44	--	13	--
Blue Springs, Missouri	90	64	32	22	16
Cummings, Georgia	100	26	--	6	--

5.3.4.1 Direct Filtration

Experiments with direct sand filtration have generally resulted in poor SS removals, as indicated in Table 5-9. Without coagulation, algae have a low affinity for sand; furthermore, green algae are too small to be efficiently removed by straining. The larger diatoms can be removed effectively, but special precautions must be taken in media design to ensure that the filter does not become rapidly clogged.

At least one investigation with direct filtration has proved successful. Wilkinson tested two types of mixed media filters for removing algae from stabilization pond effluent at the Exxon Baytown Refinery (80). A 45-cm (1.5-ft) diameter downflow deep bed was constructed using 45 cm (1.5 ft) of gravel support, 60 cm (2 ft) of 0.6 to 0.75 mm sand, and 75 cm (2.5 ft) of 0.7 to 0.9 mm anthracite coal. A downflow 43-cm (17-in) diameter Neptune-Microfloc shallow bed filter pilot unit was also used. It contained 30 cm (1 ft) of gravel support, 7.6 cm (3 in) rough garnet, 11 cm (4.5 in) of 0.33 mm garnet, 23 cm (9 in) of 0.4 to 0.6 mm sand, and 57 cm (22.5 in) of 1.0 to 1.1 mm anthracite coal. Side-by-side comparisons indicated no run length advantages for deep bed filtration. Acceptable filter operation for both filters with influent SS in the range of 20 to 60 mg/l was obtained using alum with 0.5 to 1.0 mg/l anionic or 1.0 to 2.0 mg/l cationic polyelectrolyte. Alum dosages of 25 mg/l produced only marginal improvement, while 40 to 50 mg/l provided good performance. The hydraulic loading rate was 175 to 300 $m^3/m^2/d$ (3 to 5 gpm/ft^2). At a constant SS feed of 30 mg/l, tests with the Neptune Microfloc filter indicated that the percent backwash flow increased from 8 percent at 235 $m^3/m^2/d$ (4 gpm/ft^2) to 11.8 percent at 350 $m^3/m^2/d$ (6 gpm/ft^2), and to 17.5 percent at 470 $m^3/m^2/d$ (8 gpm/ft^2). In general, work to date indicates that direct filtration of oxidation pond effluent is impractical unless algae concentrations are low.

5.3.4.2 Polishing Filtration

Use of a rapid sand or multimedia filter system to reduce SS concentrations following coagulation-clarification is very effective, achieving final effluent SS levels less than 10 mg/l and turbidities less than 4.0 JTUs (86). Diatomaceous earth filters also work efficiently, but filter cycles may be short because of filter binding by algae and other particulate matter. This results in excessive diatomaceous earth use and high operating costs.

Baumann and Cleasby (87) have shown that, while there are many similarities between water filtration (for which the most information is available) and wastewater filtration, there are also differences that must be properly accounted for in design. In particular, the quantity of solids in wastewater is generally higher and the characteristics much more variable than for water. Furthermore, filter effluent turbidities and SS concentrations will generally be much lower for water treatment applications. Therefore, direct application of designs developed for water treatment plants may result in less than optimum operation and performance in wastewater treatment.

TABLE 5-9

PERFORMANCE SUMMARY OF DIRECT FILTRATION WITH RAPID SAND FILTERS

Coagulant and Dose	Filter Loading	Filter Depth	Sand Size	Finding	Reference
mg/l	$m^3/m^2/d$	cm	mm		
None	11.7-117	61	d_{50} = 0.32	Removal declines to 21-45 % after 15 hr	81[a]
Fe: 7	123	61	d_{50} = 0.40	50 percent algae removal	
None	28.8	--[b]	d_{50} = 0.75	22 percent algae removal	82[a]
None	28.8	--[b]	d_{50} = 0.29	34 percent algae removal	
None	111	--[b]	d_{50} = 0.75	10 percent algae removal	
None	111	--[b]	d_{50} = 0.29	2 percent algae removal	
None	117	61	d_{50} = 0.71	pH 2.5, 90 percent algae removal pH 8.9, 14 percent removal	83[a]
None	64.5	28	d_{10} = 0.55	0-76 percent SS removal	75[c]
None	29.3-58.7	131	d_{50} = 0.22	20 to 45 percent SS removal	84[d]
None	29.3-176	61	d_{10} = 0.22 and 0.5	22 to 66 percent SS removal	85[d]

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

It is essential for filter runs of reasonable length that the filter remove solids throughout the entire depth of media (deep-bed filtration) and not mainly at the filter surface. Deep-bed filters can be designed by using high filtering velocities, up to 350 $m^3/m^2/d$ (6 gpm/ft^2) which permit deeper penetration of the solids into the filter, and by allowing the water to pass through a coarse-to-fine media gradation. It is advantageous in wastewater filtration to use a greater depth of filter media, 150 to 175 cm (60 to 70 in), than in water filtration, 75 to 130 cm (30 to 50 in), to allow for greater floc storage in the filter.

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

The ability of mixed media beds to capture large particles in the top layer of the bed and small particles in the lower region, allowing for greater penetration of the suspended matter throughout the filter with subsequent lower head losses and long filter runs, has been demonstrated by several researchers.

A dual-media filter consisting of 120 cm (48 in) of anthracite coal (2.4-4.8 mm) and 45 cm (18 in) of sand (0.8-1.0 mm) was used for polishing flotation-tank type effluent in a pilot study at Sunnyvale, CA (86). The loading rate was 330 $m^3/m^2/d$ (5.6 gpm/ft^2). Figure 5-19 shows effluent turbidity as a function of filter-run duration. Solids breakthrough occurred after 10 hours. Figure 5-20 shows development of the head loss profile with time. The uniform head loss increase at all depths indicates that the filter has removed solids uniformly throughout the filter depth. This factor is important in optimizing filter runs.

The average SS removal performance was 89 percent (3.0 to 6.0 mg/l effluent) using influent with concentration of 32 to 62 mg/l. Generally one-half to two-thirds of the effluent SS was volatile solids.

A similar type of filter was used by the Napa-American Canyon (California) Wastewater Management Authority to polish effluent from the coagulation-sedimentation process (88). Filter media was 50 cm (20 in) of 1.0-1.2 mm anthracite coal, 45 cm (18 in) of 0.4-0.5 mm sand, and 15 cm (6 in) of silica gravel support material. The plant used six filters with a combined capacity of 58,300 m^3/d (15.3 mgd). Filter loading rate was 290 $m^3/m^2/d$ (5 gpm/ft^2). Table 5-10 presents a summary of the data from the treatment plant for the first nine months of operation. BOD_5 removal data through the filter were not available, but effluent BOD_5 values were reported. There is consistent removal of SS.

FIGURE 5-19

DUAL-MEDIA FILTER EFFLUENT TURBIDITY PROFILE (86)

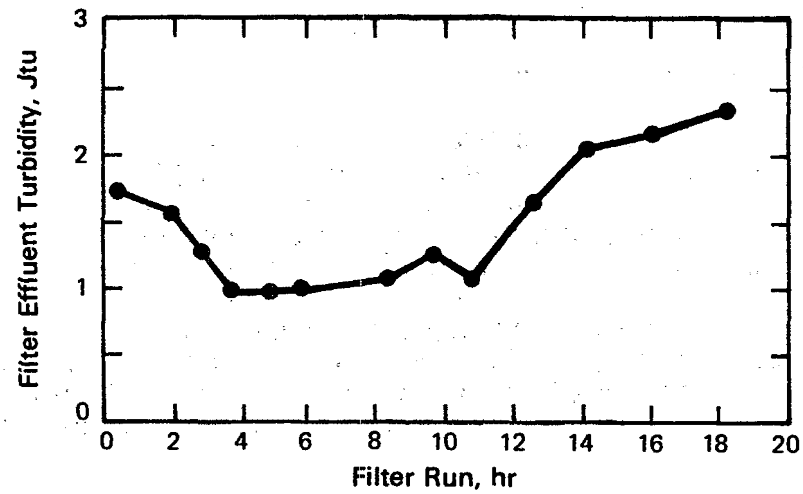


FIGURE 5-20

DUAL-MEDIA FILTER HEADLOSS PROFILE (86)

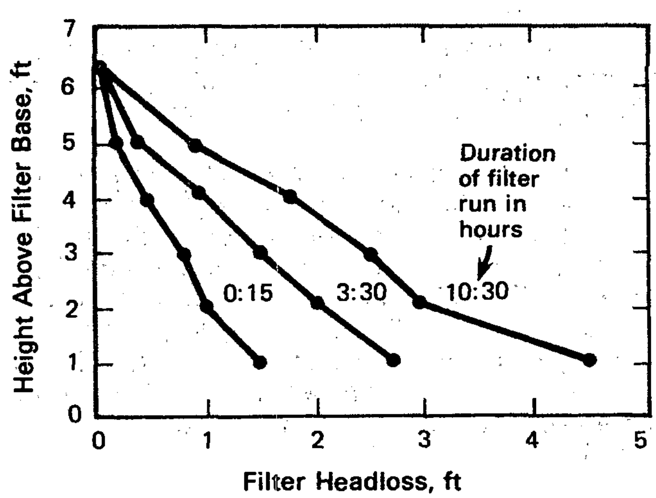


TABLE 5-10

PERFORMANCE OF THE NAPA-AMERICAN CANYON WASTEWATER MANAGEMENT AUTHORITY DUAL-MEDIA FILTERS (88)

Month	BOD_5[a] Effluent mg/l	SS Influent mg/l	SS Effluent mg/l	SS Removal percent
1978				
October	5.5	14	6.6	53
November	4.4	22	9.1	59
December	3.8	25	9.2	63
1979				
January	4.3	22	6.1	72
February	5.2	23	4.3	81
March	7.3	13	3.3	75
April	3.2	14	4.2	70
May	5.7	19	8.1	57
June	7.0[b]	19	7.4	61
Average	5.2	19	6.5	66

[a]Filter influent BOD_5 not monitored.
[b]First 11 days of month, plant does not operate in summer.

The importance of grain size selection in producing long filter runs was demonstrated by Hutchinson, et al. (89). It was found that diatoms were clogging the upper layers of mixed media filters. By increasing the effective grain size of the upper coal layer from 0.9 to 1.5 mm, while keeping the lower sand layer at an effective grain size of 0.5 mm, the length of filter run was increased from 5-12 hr to 12-20 hr. This increase was attributed to the ability of the new coal layer to capture the diatoms in a more uniform fashion throughout its depth. The quality of filter effluent was not diminished because the sand layer retained the ability to capture the smaller suspended matter.

Design procedures for effluent filtration are described in detail elsewhere (87).

5.4 Coagulation-Clarification Processes

5.4.1 Introduction

Coagulation followed by sedimentation has been applied extensively for the removal of suspended and colloidal materials from water. Lime, alum, and ferric salts are the most commonly used coagulating agents. Each of these chemicals, alone or in combination with others, may be the most appropriate under particular circumstances. The coagulant chosen will depend on pond effluent quality, the type and concentration of predominant algae, process considerations, and total cost (including sludge disposal). Procedures leading to coagulant selection include jar tests, pilot tests, and engineering feasibility studies.

5.4.2 Coagulation-Sedimentation

Although sedimentation has been used to clarify many types of wastewater, it cannot by itself be used for algae removal. Chemical coagulants must first be added to destabilize the algae. The algae-coagulant particles must then be aggregated to form flocs large enough to settle and be removed in a sedimentation tank. Thus, the sedimentation process involves three stages: 1) chemical coagulation, 2) flocculation, and 3) settling.

A number of investigators have obtained high algae removals using the coagulation-flocculation-settling sequence. Representative performance data are shown in Table 5-11. Overflow rates for conventional sedimentation processes have been 12 to 50 $m^3/m^2/d$ (0.2 to 0.8 gpm/ft^2) with hydraulic detention times of three to four hours. Flocculation tank design criteria that were found to be adequate were detention times of 25 min with a G value of 36 to 51 sec (81). Underflow total solids have generally been in the range of 1.0 to 1.5 percent when alum or iron is used.

Table 5-12 presents monthly averages of daily data collected at the Napa, CA, algae removal plant during its first nine months of operation. This facility used lime coagulation-flocculation followed by settling and dual media filtration. The effluent COD and BOD_5 removal percentages apply to the entire algae removal plant whereas the SS data are for only the coagulation-flocculation-settling process. Influent SS varied throughout the year but a consistent SS effluent quality from the clarifiers was maintained by adjusting the chemical dosage.

As shown in Table 5-11, most applications have involved alum or lime. In using these coagulants, pH control is important. Golueke and Oswald (72) found that pH for flocculation with alum was in the range of 6.3 to 6.8. This pH range applies whether alum dosages are relatively low, as with the Golueke and Oswald studies (100 mg/l), or relatively high, as with the studies at Lancaster (240-360 mg/l). When lime is added, the major effect is to raise the pH to about 11 where a magnesium hydroxide [$Mg(OH)_2$] precipitate forms,

TABLE 5-11

SUMMARY OF COAGULATION-FLOCCULATION-SETTLING PERFORMANCE

Location	Coagulant	Dose	Overflow Rate	Detention Time	BOD_5 Influent	BOD_5 Effluent	BOD_5 Removal	SS Influent	SS Effluent	SS Removal
		mg/l	$m^3/m^2/d$	min	mg/l	mg/l	percent	mg/l	mg/l	percent
Windhoek, South Africa (24)	Alum[a]	216-300	15.8	200	27.3	9.5	95	85	17	80
	Lime[b]	300-400[c]	15.8	200	27.3	3.5	87	85	8	92
Richmond, California (72)	Alum	100	45.8	150	23.0	1.0	96	199	13	93
Napa, California Pilot Plant (88)	Lime	200[d]	--e	--e	30.0	3.6	88	102	23	79
	Alum	45								
Napa, California Prototype[f] (88)	Lime	200-300	37.6	173	37.0	5.2[g]	85[g]	96	19	74

[a]As $Al_2(SO_4)_3 \cdot 14.3H_2O$ (molecular weight - 600).
[b]As CaO.
[c]pH 10.7.
[d]pH 10.8.
[e]Not available.
[f]Average of 9 months.
[g]Effluent from dual-media filters.

TABLE 5-12

PERFORMANCE OF THE NAPA-AMERICAN CANYON WASTEWATER MANAGEMENT AUTHORITY ALGAE REMOVAL PLANT (88)

Month	BOD[a]			SS[b]		
	Influent mg/l	Filtered Effluent mg/l	Removal percent	Influent mg/l	Clarified Effluent mg/l	Removal percent
1978						
October	39	5.5	86	103	14	82
November	32	4.4	86	82	22	73
December	27	3.8	86	90	25	71
1979						
January	27	4.3	84	55	22	60
February	23	5.2	77	42	23	52
March	44	7.3	83	57	13	74
April	36	3.2	91	62	14	77
May	45	5.7	87	183	19	89
June[c]	61	7.0	89	194	19	90
Average	37	5.2	85	96	19	74

[a]Following dual-media polishing filtration.
[b]Ahead of dual-media polishing filtration.
[c]First 11 days of month, plant does not operate in summer.

attaches to algae cells, and causes their sedimentation. Required lime dosage will fluctuate daily, generally from 200 to 400 mg/l. Mean lime dosage at the Napa, CA, algae removal plant for the first six months of 1979 was 246 mg/l; monthly mean lime dosages during this period ranged from 200 to 300 mg/l (88).

Tests conducted by Al-Layla and Middlebrooks (91) found that the most significant variables, in order of importance, were (1) alum dosage, (2) temperature, (3) flocculation time, (4) paddle speed, and (5) settling time. They found that large water temperature differences, even during the day, could be important.

The Los Angeles County Sanitation Districts have the longest record of experience with a coagulation-flocculation-settling system at the Lancaster Tertiary Treatment Plant, constructed in 1970. The Lancaster plant utilizes alum coagulation, sedimentation, and dual-media gravity filtration. The system has consistently produced an effluent with turbidity below 1.7 JTUs.

In designing a coagulation-flocculation-settling facility, care should be taken to ensure that conditions promoting autoflocculation are not encouraged. Floating sludge in the sedimentation tank defeats the purpose of the sedimentation process. To prevent this effect, supersaturation should be relieved by preaeration before sedimentation, and photosynthesis in the sedimentation tank should be prevented by covering the tank surface.

5.4.3 Flotation

The flotation process involves the formation of the fine gas bubbles that become physically attached to the algal solids, causing them to float to the tank surface. Chemical coagulation results in the formation of a floc-bubble matrix that allows more efficient separation to take place in the flotation tank.

Two means are available for forming the fine bubbles used in the flotation process: autoflotation and dissolved-air flotation (DAF). Autoflotation results from the provision of a region of turbulence near the inlet of the flotation tank (which causes bubble formation from dissolved gases) and from oxygen supersaturation of the pond effluent. In DAF, a portion of the influent (or recycled effluent) is pumped to a pressure tank where the liquid is agitated with high pressure air to supersaturate the liquid. The pressurized stream is then mixed with the influent, the pressure is released, and fine bubbles are formed. These become attached to the coagulated algal cells. Table 5-13 presents a summary of operating and performance data on coagulation-flotation studies.

5.4.3.1 Autoflotation

Autoflotation is the natural flotation of algae brought about by gas supersaturation in stabilization ponds. Information on autoflotation has been

TABLE 5-13

SUMMARY OF TYPICAL COAGULATION-FLOTATION PERFORMANCE

Location	Coagulant	Dose	Overflow Rate	Detention Time	BOD$_5$ Influent	BOD$_5$ Effluent	BOD$_5$ Removal	SS Influent	SS Effluent	SS Removal
		mg/l	$m^3/m^2/d$	min	mg/l	mg/l	percent	mg/l	mg/l	percent
Autoflotation										
Windhoek, South Africa (91)	Alum	220 mg/l	205	8	12.1	2.8	77	--[a]	--[a]	--[a]
	CO_2	to pH 6.5	106	8	12.1	4.4	64	--[a]	--[a]	--[a]
Stockton, California (92)	CO_2 Alum Acid	to pH 6.3 200 mg/l to pH 6.5	117	22	--[a]	--[a]	--[a]	156	75	44
Dissolved Air Flotation										
Stockton, California (92)	Alum Acid	225 mg/l to pH 6.4	158	17[b]	46	5	89	104	20	81
Lubbock, Texas (93)	Lime	150 mg/l	--[a]	12[d]	280-450	0.3	>99	240-360	0-50	>79
El Dorado, Arkansas (94)	Alum	200 mg/l	235[c]	8[c]	93	<3	>97	450	36	92
Logan, Utah (95)	Alum	300 mg/l	76-141[e]	--[a]	--[a]	--[a]	--[a]	100	4	96
Sunnyvale, California (86)	Alum Acid	175 mg/l to pH 6.0-6.3	117[f]	11[f]	--[a]	--[a]	--[a]	150	30	80

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

developed at Windhoek, South Africa, and Stockton, CA (58)(92)(93)(95)(97). For autoflotation to be effective, the DO content of the pond must exceed about 13 to 15 mg/l and the pH must be greater than 11.

Autoflotation can perform well under the proper circumstances. The major disadvantage is dependence on the development of gas supersaturation within the oxidation pond. At Windhoek, the tertiary pond is supersaturated around the clock because of their light organic loading and the presence of favorable climatic conditions. At Stockton, the required degree of supersaturation was present only intermittently, and then for less than half the day. The Stockton pond BOD_5 loadings of 37 kg/ha/d (33 lb/ac/d) during the summer are closer to normal facultative pond loadings than those at Windhoek.

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

5.4.3.2 Dissolved Air Flotation (DAF)

The principal advantage of coagulation-DAF over coagulation-flocculation-sedimentation is the smaller tanks required. Flotation can be undertaken in shallow tanks with hydraulic residence times of 7 to 20 minutes, rather than the 3 to 4 hours required for deep sedimentation tanks. Overflow rates for flotation are higher, about 120 $m^3/m^2/d$ (2 gpm/ft^2) (excluding recycle) compared to 50 $m^3/m^2/d$ (0.8 gpm/ft^2) or less for conventional sedimentation tanks.

Sedimentation, however, does not require the air dissolution equipment of flotation, making it a simpler system to operate and maintain. This factor is especially important for small plants, and it was crucial in the selection of sedimentation over flotation for the Lancaster Tertiary Treatment Plant (74).

Another advantage of flotation over sedimentation is that a separate flocculation step is not required. In fact, a flocculation step after chemical addition and before introduction of the pressurized flow into the influent has been found to be detrimental (95)(98).

a. Optimization of DAF Operation

Ramirez et al. (99) used electrocoagulation prior to DAF and achieved good results. The electrocoagulation cell, a LectroClear system, supplies a current to the influent of 0.53 ampere-minute/l. The system operates on 24 volts and has a capacity of 3,000 amperes. This helps to destabilize the algae's negative charge, making chemical coagulation more effective.

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

b. Surface Loading Rates

Studies at Stockton and Sunnyvale, CA (86)(93)(97) and at Logan, UT (96) indicate that maximum surface loading rates generally vary from 120 to 160 $m^3/m^2/d$ (2 to 2.7 gpm/ft^2), including effluent recycle, where used) *depending on tank design. Stone et al. (86) found, in prior studies at* Sunnyvale, that loadings greater than 120 $m^3/m^2/d$ (2 gpm/ft^2) caused deteriorating performance. However, the flotation tank used in the study was of poor hydraulic design and it was concluded that higher loading rates might be used in prototype facilities. It was also concluded that influent pressurization produced better results than recycle pressurization and allowed use of smaller tanks as well. Bare et al. (96) found that 140 $m^3/m^2/d$ (2.4 gpm/ft^2) was optimum and Parker et al. (93) used 160 $m^3/m^2/d$ (2.7 gpm/ft^2) at Stockton, CA. Alum was the coagulant used in all cases.

c. Pressurization and Air/Solids Ratio

The air/solids ratio is defined as the weight of air bubbles added to the process divided by the weight of SS entering the tank. Values used generally range from 0.05 to 0.10 (93)(96). The air/solids ratio is dependent on influent solids concentration, pressure level used, and percentage of influent or recycled effluent pressurized. Pressurization levels used in DAF generally range from 1.7 to 5.4 atm. Pressure may be applied to all or a portion of the flotation-tank effluent, which is then recycled to the tank influent. The latter mode has traditionally been used for sludge thickening applications when the influent solids have been flocculated and pressurizing the influent might cause floc breakup.

d. pH Sensitivity of Metal Ion Flocculation

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

Figure 5-21 shows the effect of lowering the pH on effluent SS levels during pilot studies at Sunnyvale (86), using alum as the coagulant. It was concluded that not much could be gained by lowering the pH below 6.0, and that

FIGURE 5-21

EFFECT OF ALUM DOSE AND pH ON FLOTATION PERFORMANCE (86)

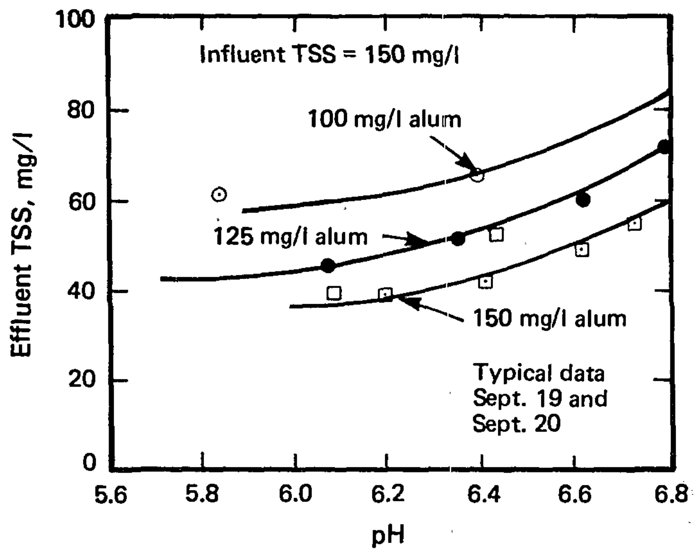
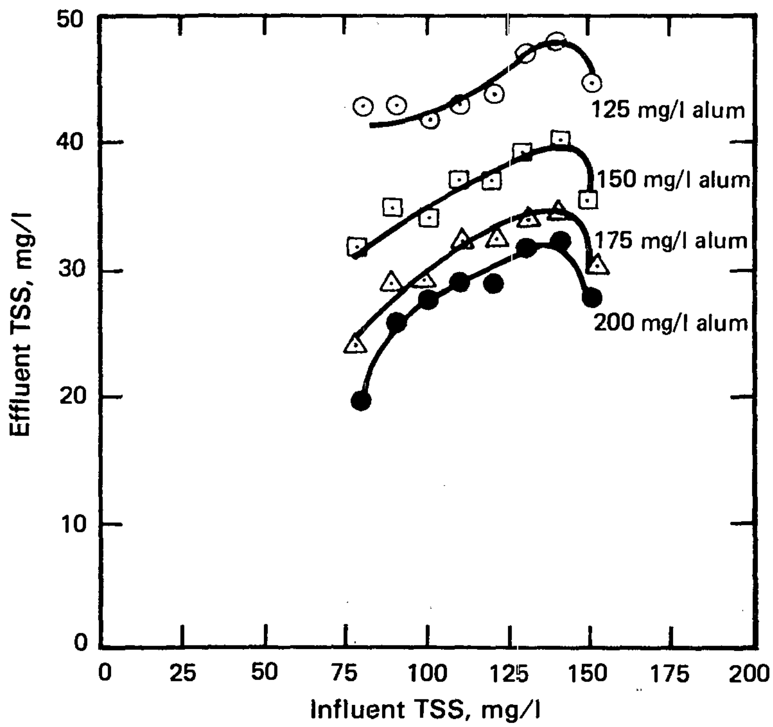


FIGURE 5-22

EFFECT OF ALUM DOSE AND INFLUENT SS ON FLOTATION PERFORMANCE (86)



the range of 6.0 to 6.3 could be used for optimum performance. Subsequent neutralization can be accomplished by adding caustic soda.

e. Alum Dose

Pilot studies at Stockton (93) and Sunnyvale (86) (Figure 5-22) show the effect of influent TSS and alum dose on effluent TSS concentrations. The results presented in Figure 5-22 show that influent TSS have a relatively minor effect on effluent quality. The benefit of increasing alum doses is most pronounced up to about 175 mg/l. Beyond that range, increased alum addition results in only marginal improvement in effluent TSS levels.

f. Physical Design

It was noted above that proprietary flotation tank designs do not possess certain features important in pilot- and full-scale studies of algae removal. Features incorporated in the flotation tank designs for Sunnyvale and Stockton are shown in Figure 5-23 and illustrate important design concepts.

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

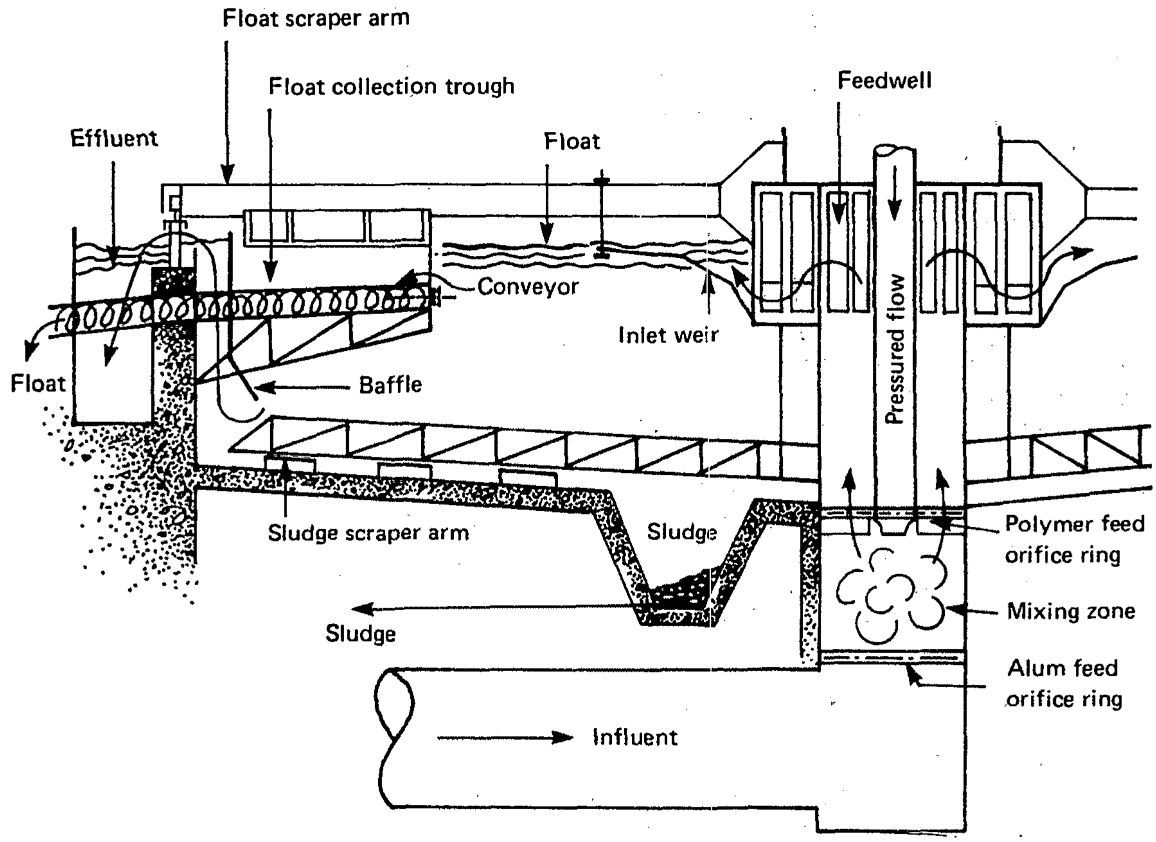
- The point of pressure release is in the feedwell. An orifice, rather than a valve, can be used on the pressurized line because the DAF tanks can operate at constant flow, using the ponds for flow equalization. In most proprietary designs, a valve is provided on the pressurized line at the outside tank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

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

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

FIGURE 5-23

CONCEPTUAL DESIGN OF DISSOLVED AIR FLOTATION TANK
APPLIED TO ALGAE REMOVAL



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

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

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

g. Float Concentration

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

Bare et al. (96) reported float concentrations of 1.0 to 1.3 percent with alum coagulation/DAF. Concentrations increased to about 2.0 percent when a second flotation was allowed to occur in the skimmings receiving tank. Stone et al. (86) reported float concentrations of 1.3 to 2.1 percent in the Sunnyvale, CA, studies with specific gravities of 0.45 to 0.55.

h. Solids Handling and Treatment

Satisfactory disposition must be made of the algal-chemical sludge generated by coagulation-clarification processes. Application of conventional solids-handling and treatment processes requires increased capital and operating expenses. This consideration was among those that led Middlebrooks et al. (58) to recommend against using coagulation-clarification processes for small plants.

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

Both centrifugation and vacuum filtration of unconditioned algal-alum sludge have produced marginal results because of dewatering difficulties and the need for using low process loading rates (72)(97). Heat treatment using the Porteous process at temperatures of 193 to 213 °C has been shown to improve subsequent vacuum filter yield and cake concentration to a limited extent. Filter yield was low and ranged from 4.4 to 12.2 $kg/m^2/h$ (0.9 to 2.5 $lb/ft^2/hr$). Cake concentrations during the study were 8.3 to 21.6 percent total solids, using raw sludge with a solids concentration of about four percent in the feed algal-alum sludge (97).

Use of Zimpro low-temperature oxidation, at temperatures of 180 to 220 °C, has resulted in vacuum filter cake concentrations of 15 to 19 percent total solids, at filter-yield rates of 3.3 to 14.9 $kg/m^2/h$ (0.7 to 3.0 lb ft^2/hr) (97).

Zimpro high-temperature oxidation, with temperatures ranging from 220 to 275 °C, was also investigated because it would lead directly to ultimate disposal of the sludge. Evaluation showed that cake concentrations and filter yield improvements were marginal, indicating that ultimate disposal should incorporate ponds. The high-oxidation process reduces VSS in the sludge by about 97 percent, which is important in producing a stable end product. Although some of the volatile solids are made soluble in the liquid, the final solids are stable and suitable for pond storage (96).

Only limited investigations have been made into the use of centrifugation for concentrating algal-chemical sludges. At Firebaugh, CA, a Bird solid-bowl centrifuge and a DeLaval yeast-type separator were used to dewater sludge (100). Both devices were considered failures, although the use of sludge conditioning aids, such as organic polymers, might be expected to improve their performance. A DeLaval self-cleaning basket machine, also tested, was able to concentrate a two to three percent feed to 10 percent total solids with a recovery of 98 percent.

Centrifugation has been used for lime classification of raw wastewater sludges (101)(102), but the only report on its use for algal-lime sludge did not present specific details (94).

Another process that has been investigated is a chemical-oxidation scheme, called Purifax, that employs chlorine as the oxidant. This process was capable of stabilzing the sludge, and yielded a product that could be dewatered on sand drying beds or in a pond; however, chlorine costs are relatively high (97).

Initial work on anaerobic digestion of algal-alum sludge, at the University of California, indicated that the process held little promise for future use (103). Volatile matter reduction was less than 44 percent, and the digested sludge was unstable and slow to dewater. Subsequent work has shown that algae can be anaerobically degraded successfully if they are killed before their introduction into the digester.

While these relatively complex processes have generally proved unsatisfactory, there is a comparatively simple, and potentiallly effective, solution to the solids-handling problem--return of the algal-alum sludge to the pond (104). When algae-alum sludge is returned to a pond, it must be distributed to reduce accumulation at a single point. Furthermore, when air is contained in the sludge float, procedures must be found to remove it before introducing the sludge into the pond, or floating sludge problems will result. Several methods have been investigated for breaking down collected float, including the use of high-shear pumps, pumps using a vacuum, high-shear mixers, and water sprays (100).

i. Coagulant Recovery

Because chemicals are used in large quantities for coagulation, their regeneration and reuse may be a way to reduce overall operating costs. Use of acid to reduce pH to about 2.5 can result in a 70 percent alum recovery (100). Because phosphorus is also released at low pH, acid recovery will be limited to those situations where phosphorus removal is not required.

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

5.5 Land Application

5.5.1 Introduction

Land application systems can be classified into one of three categories: overland flow (OF) (involving flow over grassed terraces); irrigation of crops by conventional methods, or slow-rate treatment (SR); and rapid infiltration (RI) systems. The following subsections discuss design criteria and performance of the three system types when used in conjunction with pond effluent. Comparison of typical characteristics and design criteria for land treatment processes are presented in Tables 5-14 and 5-15 (105).

5.5.2 Overland Flow

Overland flow is a land treatment process which can be used specifically on sites containing soils with limited permeability. In an OF process, wastewater is distributed along the top portion of sloped terraces and allowed to flow across a vegetated surface to runoff collection ditches. The wastewater is renovated as it flows in a thin film down over the sloping ground surface. Since the system does not rely on percolation into the soil, overland flow can be used on clay and silty type soils with low infiltration capacity.

The treatment mechanisms of an OF system are similar in some respects to most land treatment systems. Biological oxidation, sedimentation, and grass filtration are the primary removal mechanisms for organics and SS. Phosphorus and heavy metals are removed principally by adsorption, precipitation, and ion exchange in the soil. Some are also removed by plant uptake. Dissolved

TABLE 5-14

COMPARISON OF SITE CHARACTERISTICS FOR
LAND TREATMENT PROCESSES (105)

	Slow Rate	Rapid Infiltration	Overland Flow
Grade	Less than 20% on cultivated land; less than 40% on noncultivated land	Not critical; excessive grades require much earthwork	Finish slopes 2-8%[a]
Soil Permeability	Moderately slow to moderately rapid	Rapid (sands, sandy loams)	Slow (clays, silts, soils w/impermeable barriers)
Depth to Groundwater	0.6-1.0 m minimum[b]	0.3 m during flood cycle; 1.5-3.0 m during drying cycle	Not critical[c]
Climatic Restrictions	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often needed for cold weather treatment

[a]Steeper grades might be feasible at reduced hydraulic loadings.
[b]Underdrains can be used to maintain this level at sites with high groundwater table.
[c]Impact on groundwater should be considered for more permeable soils.

TABLE 5-15

COMPARISON OF TYPICAL DESIGN FEATURES FOR LAND TREATMENT PROCESSES (105)

	Slow Rate	Rapid Infiltration	Overland Flow
Application Techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface
Application Rate	0.5-6.0 m/yr	6-125 m/yr	3-20 m/yr
Field Area Required[b]	23-280 ha	3-23 ha	6.5-44 ha
Typ. Weekly Application Rate	1.3-10 cm	10-240 cm	6-40 cm[c]
Minimum Pre-application Treatment Provided in U.S.	Primary sedimentation[d]	Primary sedimentation	Grit removal and comminution[e]
Disposition of Applied Wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for Vegetation	Required	Optional	Required

[a]Includes ridge-and-furrow and border strip.
[b]Field area (not including buffer area, road, or ditches) for 3,785 m^3/d flow.
[c]Range includes raw wastewater to secondary effluent, higher rate for higher level of preapplication treatment.
[d]With restricted public access; crops not for direct human consumption.
[e]With restricted public access.

solids are removed by plant uptake, ion exchange, and through leaching. Algae removal by OF systems has been inconsistent and it should be evaluated carefully before using OF as a means of algae removal.

Nitrogen levels can also be significantly reduced by several soil and plant processes. An aerobic zone is maintained in the top layer of liquid flow where ammonia is oxidized into nitrite and then nitrate. Immediately below the soil surface, in the top few millimeters, an anaerobic zone is established where denitrifying bacteria convert the nitrates into nitrogen gas, resulting in high overall reduction in the total applied nitrogen. Plant uptake and volatilization under proper pH conditions are other important removal mechanisms, but permanent nitrogen removal by plant uptake is only possible if the cover crop is harvested and removed from the field.

The basic differences between the OF systems cited in Tables 5-16 through 5-18 are the application methods and rates. Two types of application can be used: (1) sprinkler, or; (2) gravity surface irrigation by means of grated pipe or bubbling orifice. The most common method is sprinkler, which ensures even distribution and can be automatically controlled.

5.5.3 Slow-Rate or Crop Irrigation

In humid climates, SR systems are generally designed for the maximum possible hydraulic loading to minimize land requirements. Systems of this type have been successfully designed for forests, pastures, forage grasses, corn, and other crop production. In arid climates, where water conservation is more critical, wastewater is often applied at rates that just equal the irrigation needs of the crop. Water rights must also be given careful consideration in arid climates prior to diverting pond effluent to another location for land treatment.

Design of SR systems for typical municipal effluents is usually based on either the limiting permeability of the in situ soils or on meeting nitrate requirements in the groundwater, if the site overlies a potable aquifer. The wastewater hydraulic loading rate is calculated for both conditions, and the limiting value then controls design (105).

Performance and design data for selected SR systems treating pond effluent are summarized in Tables 5-19 through 5-22.

5.5.4 Rapid Infiltration

The principal difference between RI and SR is that hydraulic loading rates are greater. Highly permeable soils must be available. Nitrogen removal mechanisms rely less on crop uptake and more on nitrification-denitrification within the soil. It has been suggested that such systems only be allowed when groundwater quality is either of no consequence or when the percolate can be controlled (106).

TABLE 5-16

SUMMARY OF BOD AND SS REMOVALS AT OVERLAND FLOW SYSTEMS TREATING POND EFFLUENTS[a] (105)

Location	Slope Length	Application Rate	Hydraulic Loading Rate	Application Period	Application Frequency	BOD Influent	BOD Effluent	SS Influent	SS Effluent
	m	m^3/m/hr	cm/d	hr/d	d/wk	mg/l			
Pauls Valley, OK	46	0.06	1.66	12	7	27.7	20.5	114	72.8
Utica, MS	46	0.032	1.27	18	5	22	3.5	30	5.5
		0.065	2.54	18	5	22	4.0	30	8.0
		0.049	2.54	24	7	22	5.5	30	13.0
		0.13	5.08	18	5	22	7.5	30	13.0
		0.10	1.27	6	5	22	8.6	30	6.4
Easley, SC	46	0.23	3.58	7	5	28	15	60	40

[a]Performance during warm season.

TABLE 5-17

SUMMARY OF NITROGEN AND PHOSPHORUS REMOVALS AT OVERLAND FLOW SYSTEMS TREATING POND EFFLUENTS[a] (105)

Location	Hydraulic Loading Rate	Total N		Ammonia-N		Nitrate-N		Total P	
		Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
	cm/d	mg/l							
Pauls Valley, OK	1.66	15.5	11.4	1.7	0.4	<0.1	0.2	6.3	5.1
Utica, MS	1.27	20.5	4.3	15.6	0.1	<1.0	1.0	10.3	4.9
	2.54	20.5	7.5	15.6	0.8	<1.0	2.6	10.3	6.1
	2.54	20.5	7.3	15.6	0.7	<1.0	3.1	10.3	5.9
	5.08	20.5	10.0	15.6	1.1	<1.0	4.8	10.3	8.2
	1.27	20.5	7.0	15.6	0.8	<1.0	3.2	10.3	7.1
Easley, SC	3.58	6.7	2.1	1.0	0.4	2.4	1.1	3.8	2.2

[a]Performance during warm season.

TABLE 5-18

REMOVAL OF HEAVY METALS AT DIFFERENT HYDRAULIC RATES AT UTICA, MISSISSIPPI (105)

Hydraulic Loading Rate	Runoff Concentration				Removal Efficiency			
	Cadmium	Nickel	Copper	Zinc	Cadmium	Nickel	Copper	Zinc
cm/d	mg/l				percent			
1.27	0.0046	0.0131	0.0129	0.0558	85	92	93	88
2.54	0.0036	0.0217	0.0293	0.0525	91	88	82	87
3.81	0.0079	0.0302	0.0382	0.0757	78	80	74	79
5.08	0.0142	0.0486	0.0524	0.0853	63	66	64	75

TABLE 5-19

BOD REMOVAL DATA FOR SELECTED SLOW-RATE SYSTEMS TREATING POND EFFLUENTS (105)

Location	Hydraulic Loading Rate cm/yr	Surface Soil	BOD In Applied Wastewater mg/l	BOD In Treated Wastewater mg/l	BOD Removal percent	Sampling Depth m
Dickinson, ND	140	Sandy loams and loamy sands	42	<1	>98	<5
Muskegon, MI	130-260	Sands and loamy sands	24	1.3	94	4
San Antonio, TX	290	Clay and clay loam	89	0.7	99	2.1

TABLE 5-20

NITROGEN REMOVAL DATA FOR SELECTED SLOW-RATE SYSTEMS TREATING POND EFFLUENTS (105)

Location	Total Nitrogen in Applied Wastewater mg/l as N	Total Nitrogen in Percolate or Affected Groundwater mg/l as N	Removal percent	Sampling Depth m	Total Nitrogen in Background Groundwater mg/l as N
Dickinson, ND	11.8	3.9	67	11	1.9
Helen, GA	18.0	3.5	80	1.2	0.17
San Angelo, TX	35.4	6.1	83	10	--

TABLE 5-21

PHOSPHORUS REMOVAL DATA FOR SELECTED SLOW-RATE SYSTEMS TREATING POND EFFLUENTS[a] (105)

Location	Hydraulic Loading Rate cm/yr	Surface Soil	PO_4 in Applied Wastewater mg/l as P	Soluble PO_4 in Affected Groundwater mg/l as P	Removal percent	Sampling Depth m	Distance from Site m	Soluble PO_4 in Background Groundwater mg/l as P
Agricultural Systems								
Dickinson, ND	140	Sandy loams and loamy sands	6.9	0.05	99	<5	30-150	0.04
Muskegon, MI	130-260	Sands and loamy sands	1.0-1.3	0.03-0.05	95-98	1.5	0	0.03
Forest System								
Helen, GA	380	Sandy loam	13.1	0.22	98	1.2	0	0.21

[a]Total phosphate concentration.

TABLE 5-22

TRACE ELEMENT BEHAVIOR DURING SLOW-RATE TREATMENT OF POND EFFLUENTS (105)

Element	EPA Drinking Water Standard mg/l	Raw Municipal Wastewater Concentration mg/l	Muskegon, Michigan[a] Percolate Concentration mg/l	Muskegon, Michigan[a] Removal Percent	San Antonio, Texas[b] Percolate Concentration mg/l	San Antonio, Texas[b] Removal Percent	Melbourne, Australia[c] Percolate Concentration mg/l	Melbourne, Australia[c] Removal Percent
Cadmium	0.01	0.004-0.14	<0.002	90	<0.004	--[d]	0.002	80
Chromium	0.05	0.02-0.7	0.004	90	<0.005	>98	0.03	90
Copper	1.0	0.02-3.4	0.002	90	0.014	85	0.02	95
Lead	0.05	0.05-1.3	<0.050	>40	<0.050	--[d]	0.01	95
Manganese	0.05	0.11-0.14	0.26	15	--	--	--	--
Mercury	0.002	0.002-0.05	<0.002	--[d]	--	--	0.0004	85
Zinc	5.0	0.03-83	0.033	95	0.102	25	0.04	95

[a] Average annual concentrations (1975) found in underdrains placed at a depth of 1.5 m below irrigation site.
[b] Average annual concentrations (November 1975 - November 1976) found in two seepage creeks adjacent to the irrigated area.
[c] Average annual concentrations (1977) found in underdrains placed at depths of 1.2 to 1.8 m below the irrigation site.
[d] Percent removal was not calculated since influent and percolate values are below lower detection limit.

There are numerous examples of rapid infiltration of treated effluent. Hydraulic loading rates for secondary effluent range down from 2.1 m/wk (7 ft/wk) at Flushing Meadows, AZ, on sandy soil to 20 cm/wk (8 in/wk) at Westby, WI, on silt loam. For primary effluent, loading rates of 57 cm/wk (22 in/wk) are used at Hollister, CA (105).

The few examples of RI with pond effluent are presented in Tables 5-23 and 5-24. Of particular concern would be whether the algae would clog the soil. The data accumulated so far at Flushing Meadows seem to indicate that algae have a greater clogging potential than an equal mass of SS from secondary treatment (activated sludge) at the higher loading rates employed in that system. This fact is ordinarily of no consequence for the more common low-rate systems applying approximately 2.5 to 7.5 cm/wk (1 to 3 in/wk). However, Hicken et al. (107) observed prolific algal growths on nonvegetated control plots at application rates of 5 to 15 cm/wk (2 to 6 in/wk) (100). Contrary to observations, the algal mats did not increase the hydraulic impedance on these sites. Perhaps the explanation lies in the great difference in application rate between the two systems.

TABLE 5-23

NITROGEN REMOVAL DATA FOR SELECTED RAPID INFILTRATION SYSTEMS TREATING POND EFFLUENTS (105)

Location	Total N in Applied Wastewater, mg/l	Loading Rate, m/yr	BOD:N	Flooding Time:Drying Time	Renovated Water, mg/l: NO_3-N	Renovated Water, mg/l: Total N	Total N Removal, percent
Brookings, SD	10.9	12.2	2:1	1:2	5.3	6.2	43

TABLE 5-24

FECAL COLIFORM REMOVALS AT SELECTED RAPID INFILTRATION SYSTEMS TREATING POND EFFLUENTS (105)

Location	Soil Type	Fecal Coliforms, MPN/100 ml: Applied Wastewater	Fecal Coliforms, MPN/100 ml: Renovated Water	Distance of Travel, m
Hemet, CA	Sand	60,000	11	2
Milton, WI	Gravelly sands	TNTC[a]	0	8-17
Santee, CA	Gravelly sands	130,000 130,000	580 <2	61 762

[a]At least one sample too numerous to count.

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CHAPTER 6

COST AND ENERGY REQUIREMENTS

The costs associated with wastewater treatment facilities are usually divided into capital costs and operations and maintenance (O&M) costs. The O&M costs are significantly influenced by energy consumption in most wastewater treatment facilities. Where O&M costs are available, the data are presented; however, there is a limited amount of data for the operation and maintenance of wastewater ponds. The costs and energy requirements will be discussed individually in the following sections.

6.1 Capital Costs

The majority of the cost data presented in this section were extracted from a technical report distributed by the Environmental Protection Agency (1). These data reflect the grant-eligible costs associated with the construction of publicly owned wastewater treatment facilities and were derived from the actual winning bid documents. These cost data are the most complete data available and represent all types of wastewater processes. If comparison of the pond costs are to be made with other types of treatment facilities, it is suggested that the design engineer consult the above-referenced report, which has a detailed explanation of the data base and techniques used to analyze the data.

These data are useful for preliminary design and planning purposes. Conventional estimating procedures should be used during final design. The costs shown in Figures 6-1 through 6-3 are national averaged costs indexed to Kansas City/St. Joseph, MO, during the fourth quarter of 1978. Individual data points are included on the graphs to illustrate the wide variation that occurred in construction costs around the country.

The results presented represent only construction costs and do not show other costs. These other costs include eligible Step 1 and Step 2 planning costs as well as those associated with Step 3 construction effort: administration, architect/engineer fees, contingency allowances, etc. Table 6-1 contains the average ratios of all these Step 3 cost categories to the total construction costs for new projects. There are 15 categories of costs identified in Table 6-1. Only five of these cost categories were found in the majority of the projects: administrative/legal costs, architect/engineer basic fees, other architect/engineer fees, project inspection costs, and contingencies. These five categories equal approximately 20 percent of the construction costs as a national average. However, including all of the 15 categories, the national average costs were approximately 50 percent of the construction costs. In

FIGURE 6-1

ACTUAL CONSTRUCTION COST vs DESIGN FLOW FOR DISCHARGING STABILIZATION PONDS (COST BASE 1978)

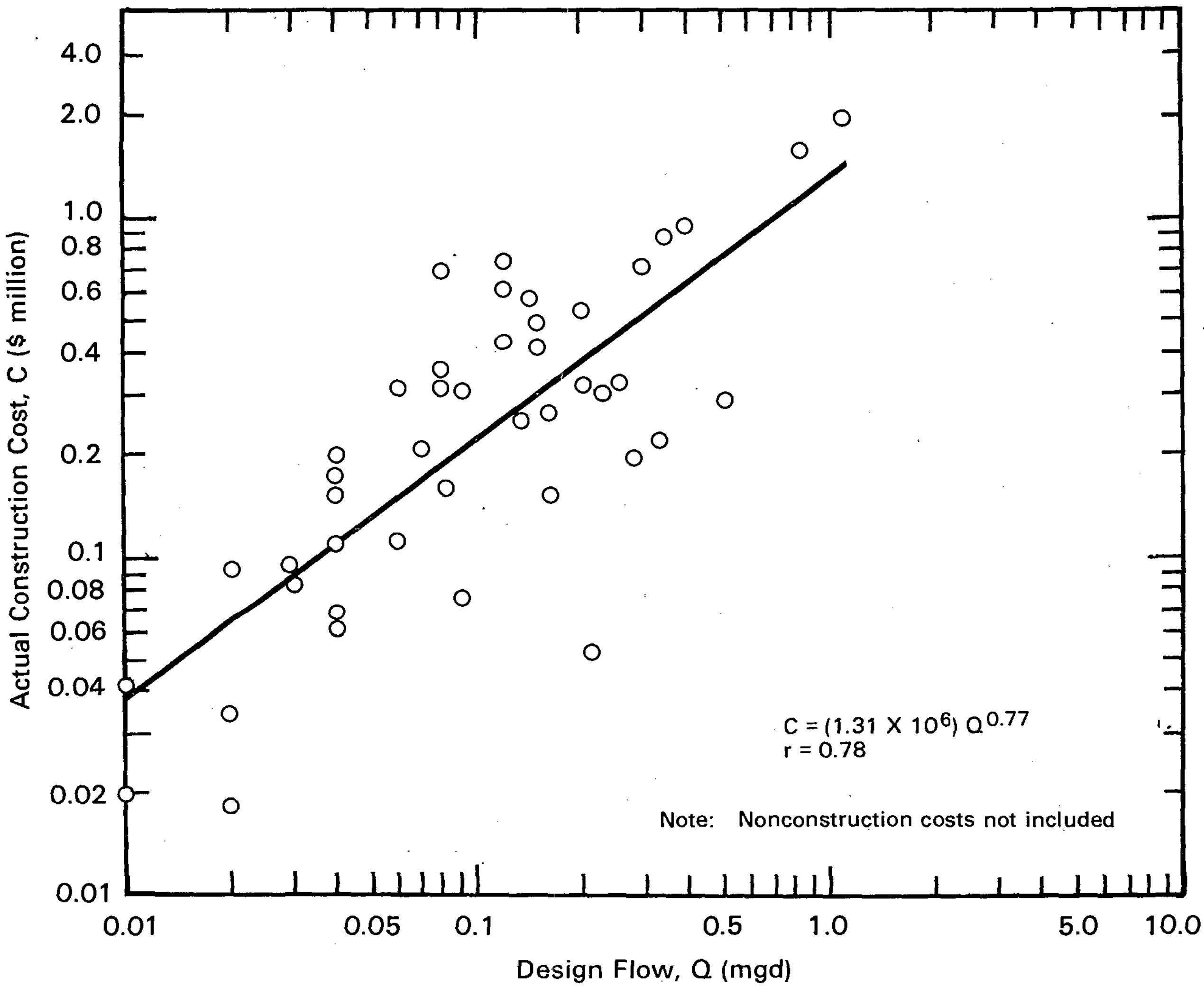

FIGURE 6-2

ACTUAL CONSTRUCTION COST vs DESIGN FLOW FOR NONDISCHARGING STABILIZATION PONDS (COST BASE 1978)

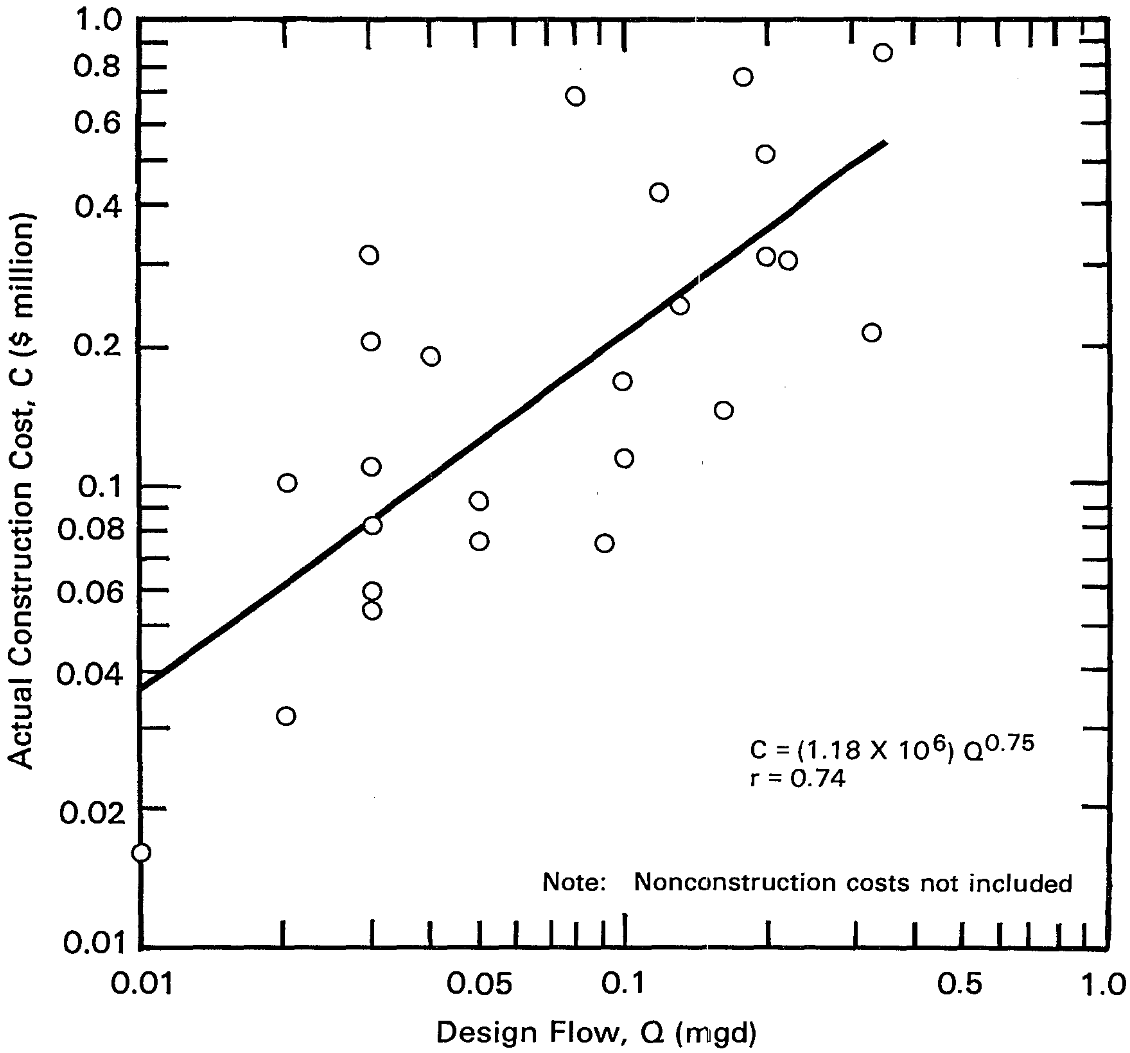

FIGURE 6-3

ACTUAL CONSTRUCTION COST vs DESIGN FLOW FOR
AERATED PONDS
(COST BASE 1978)

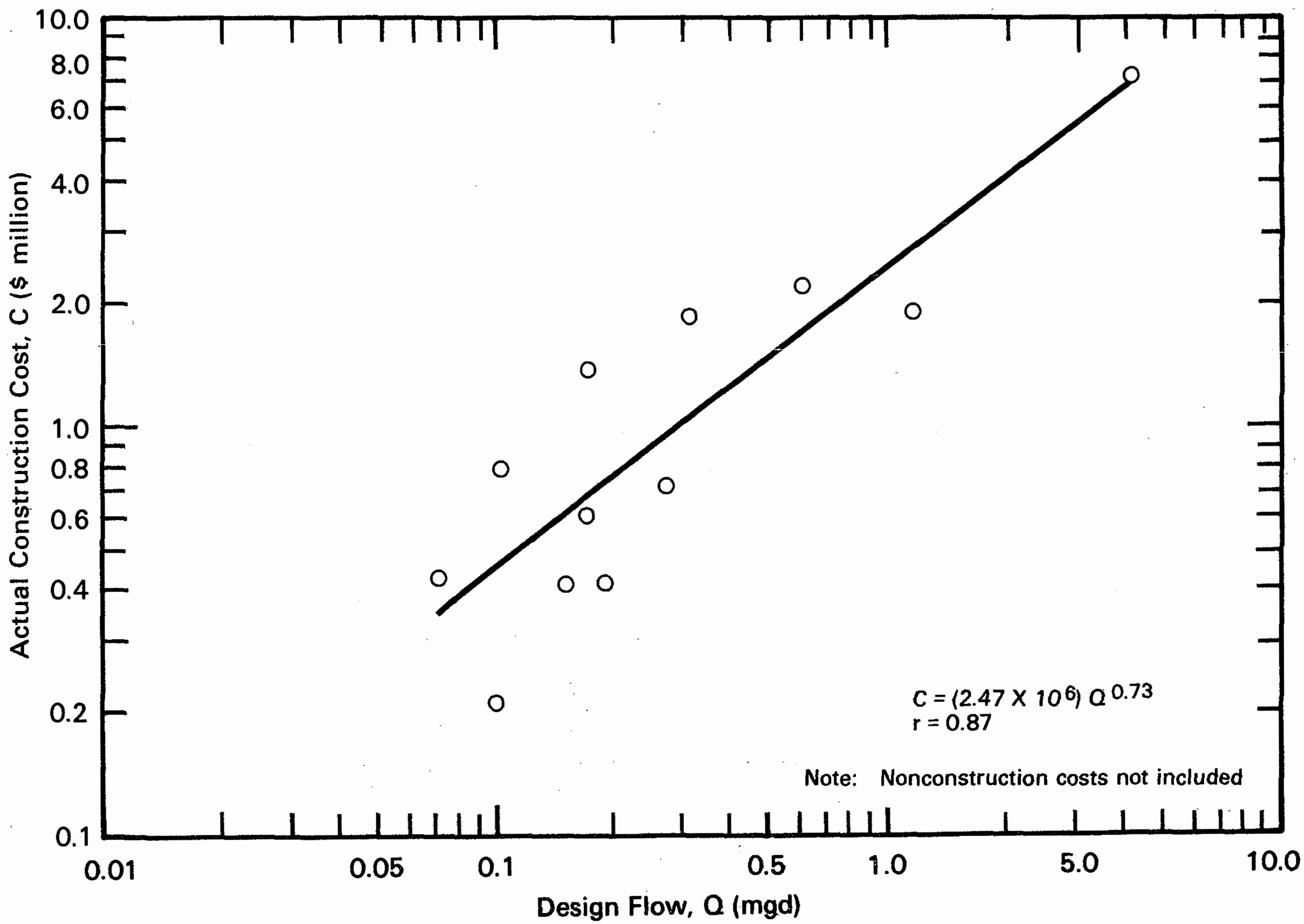

TABLE 6-1

AVERAGE NONCONSTRUCTION COST RATIOS FOR NEW WASTEWATER TREATMENT PLANTS (1) (COST BASE 1978)

Step III Nonconstruction Cost Category	Nonconstruction Cost/Total Construction Costs											
	REG. 01	REG. 02	REG. 03	REG. 04	REG. 05	REG. 06	REG. 07	REG. 08	REG. 09	REG. 10	National	Sample Size
Administration/Legal	0.0119	0.0167	0.0201	0.0068	0.0088	0.0092	0.0071	0.0127	0.0094	0.0112	0.0117	320
Preliminary	0.0316	0.0101	—	—	0.0116	—	0.0053	0.0106	0.0072	0.0141	0.0120	25
Land, Structures, Right-of-Way	0.0144	0.0296	0.0193	0.0186	0.0364	0.2851	0.0760	0.1115	0.0370	0.0338	0.0442	83
A/E Basic Fees	0.1128	0.0652	0.1135	0.0571	0.0759	0.0481	0.0423	0.0757	0.0925	0.0412	0.0739	300
Other A/E Fees	0.0342	0.0409	0.0112	0.0236	0.0386	0.0166	0.0156	0.0252	0.0286	0.0258	0.0287	178
Inspection	0.0516	0.0614	0.0444	0.0227	0.0254	0.0261	0.0416	0.0433	0.0536	0.0440	0.0405	138
Land Development	—	—	0.0096	—	—	—	—	—	—	—	0.0096	1
Relocation	0.0097	—	—	0.0049	0.0104	—	—	0.0048	—	0.0004	0.0068	6
Relocation Payments	—	—	—	0.0049	—	—	—	—	—	—	0.0049	1
Demolition and Removal	—	—	—	—	0.0100	—	—	—	—	0.0454	0.0277	2
Bond Interest	0.0214	—	0.0311	0.0258	—	0.0096	—	0.0287	—	—	0.0224	12
Contingency	0.0564	0.0608	0.0497	0.0693	0.0286	0.0378	0.0517	0.0520	0.0623	0.0368	0.0470	321
Indirect Costs	—	0.0048	0.0022	—	—	—	—	—	0.0059	—	0.0037	7
Miscellaneous	0.0164	—	—	0.0431	—	0.0072	0.0385	0.0051	0.0418	0.0437	0.0297	25
Equipment	—	—	0.0117	0.0070	0.0180	0.0065	0.0250	0.0191	0.0090	0.0768	0.0309	28
ELIGIBLE SUBTOTAL	0.3604	0.2995	0.3128	0.2838	0.2637	0.4462	0.3031	0.3887	0.3473	0.3732	0.3937	—
Ineligible Costs[a]	0.0273	0.0423	0.1168	0.0400	—	0.0292	0.1292	0.2621	—	0.0472	0.1083	51
TOTALS	0.3877	0.3418	0.4296	0.3238	0.2637	0.4754	0.4323	0.6508	0.3473	0.4204	0.5020	1498

[a] Eligible and ineligible costs.

(a) Only the treatment and treatment residue disposal portions of toilets with composting tanks, oil-flush mechanisms, or similar in-house systems are grant eligible.

(b) Acquisition of land in which the individual system treatment works are located is not grant eligible.

(c) Commodes, sinks, tubs, drains, and other wastewater generating fixtures and associated plumbing are not grant eligible. Modifications to homes or commercial establishments are also excluded from grant eligibility.

(d) Only reasonable costs of construction site restoration to preconstruction conditions are eligible. Costs of improvement or decoration associated with the installation of individual systems are not eligible.

(e) Conveyance pipes from wastewater generating fixtures to the treatment unit connection flange or joint are not eligible where the conveyance pipes are located on private property.

STEP 1/TCC = 0.0233
STEP 2/TCC = 0.0555

addition to the Step 3 nonconstruction costs, the Steps 1 and 2 costs (preliminary and detailed design) must also be included. These two costs were calculated as a fraction of the total construction cost and are presented at the bottom of Table 6-1. The costs were 2.33 and 5.55 percent for Steps 1 and 2, respectively. The information presented in Table 6-1 can be used to estimate the nonconstruction costs for a wastewater treatment facility by adding the total construction costs, total Step 3 nonconstruction costs, as well as the Step 1 and Step 2 costs.

6.2 Cost Updating

The costs may be updated to other geographical areas by using the following formula:

$$\text{Total Project Cost from Figures 6-1 through 6-3} \times \frac{\text{Latest LCAT or SCCT Index for Desired Area}}{\text{4th Quarter 1978 LCAT or SCCT Index for Desired Area}} = \text{Updated Cost} \qquad (6\text{-}1)$$

where

LCAT = EPA large city advanced treatment index

SCCT = EPA small city conventional treatment index

The LCAT and SCCT indexes are published quarterly by the Environmental Protection Agency.

The cost data presented in Figures 6-1 through 6-3 do not cover the options available to upgrade pond effluents to meet secondary or advanced secondary levels. Additional data have been collected from selected projects around the country to provide individual cost data for comparative purposes.

Cost and performance values shown in Table 6-2 represent the best available information for all of the processes listed. In several cases the costs are based on estimates derived from pilot plant studies or engineering estimates. Where actual bid prices are available, the location of the facility is given. All costs are site specific and can be expected to vary widely. Costs are reported as shown in the literature, and changes in value of the dollar are not corrected for. This was done to allow the reader to use the system appropriate for his/her area to adjust the costs to a current base. Corrections were made to all capital costs to reflect a 7 percent interest rate and a 20-year life except for systems known to have shorter operating periods. The exceptions are identified in Table 6-2.

The selection of the cost-effective alternative must be made based upon good engineering judgment and local economic conditions. Cost variations in one

TABLE 6-2

COMPARATIVE COSTS AND PERFORMANCE OF VARIOUS UPGRADING ALTERNATIVES AND POND SYSTEMS

Process or System and Location	Design Flow Rate	Design Loading	Annual Costs[a] Capital	O&M	Total	Cost Base	Reference	Effluent Concentration BOD_5	SS
	m^3/d			$/m^3$					
Overland Flow[b]									
EPA Estimate	1,140	5 cm/wk	0.071	0.037	0.108	1973	(2)	<10	<10
EPA Estimate	1,140	20 cm/wk	0.050	0.026	0.076	1973	(2)		
Davis, CA	18,900	20 cm/wk	0.026	0.013	0.039	1976	(3)		
Surface Irrigation[b]									
EPA Estimate	1,140	5 cm/wk	0.053	0.050	0.103	1973	(2)	<10	<10
EPA Estimate	1,140	10 cm/wk	0.045	0.040	0.085	1973	(2)		
Spray Irrigation-Center Pivot[b]									
EPA Estimate	1,140	5 cm/wk	0.050	0.048	0.098	1973	(2)	<10	<10
EPA Estimate	1,140	10 cm/wk	0.042	0.034	0.076	1973	(2)		
Spray Irrigation-Solid Set[b]									
EPA Estimate	1,140	5 cm/wk	0.069	0.040	0.109	1973	(2)	<10	<10
EPA Estimate	1,140	10 cm/wk	0.050	0.032	0.082	1973	(2)		
Rapid Infiltration									
EPA Estimate	1,140	20 cm/wk	0.045	0.026	0.071	1973	(2)	<10	<10
EPA Estimate	1,140	60 cm/wk	0.034	0.021	0.055	1973	(2)		
Intermittent Sand Filtration									
Metcalf & Eddy Capital Cost Est. and European O&M[c]	1,140	—	0.042	0.042	0.084	1975	(4)(5)	<15	<15
Huntington, UT	1,140	2,800 $m^3/ha/d$	0.095			1975	(6)		
Kennedy, AL	318	935 $m^3/ha/d$				1975	(7)		
Ailey, GA	303	5,610 $m^3/ha/d$	0.053	0.005	0.058	1975	(8)(18)		
Moriarty, NM	760	2,810 $m^3/ha/d$	0.032	0.010	0.042	1975	(9)(18)		
White Bird, ID	114	3,740 $m^3/ha/d$	0.048			1978	(10)		
Mt. Shasta, CA	2,650	6,550 $m^3/ha/d$	0.050	0.010	0.060	1976	(18)		
Microscreens									
ENVIREX	6,440-8,520	—	0.029-0.037[e]	0.013[e]	0.042-0.050	1978	(11)	<30	<30
Camden, SC	7,200	—	0.038	0.027	0.065	1979	(24)		
Dissolved Air Flotation									
Snider	3,030	—	0.037	0.016	0.053[d]	1975	(12)	<30	<20
Coagulation-Flotation-Sedimentation-Filtration									
Los Angeles Co., CA	1,890	—	0.034	0.079	0.113	Cap 1970 O&M 1973 1974	(13)	<10	<10
Rock Filters									
Wardell, MO	303	0.76 m^3/pop. eq.	0.011			1974	(14)	<30	<30
Delta, MO	303	0.76 m^3/pop. eq.	0.013			1974	(14)		
California, MO	1,360		0.011			1974	(15)		
Luxemburg, WI	1,510	0.40 $m^3/m^3/d$	0.008[e]			1976	(16)		
Veneta, OR	830	0.27 $m^3/m^3/d$	0.013			1975	(17)		

TABLE 6-2

CONTINUED

Process or System and Location	Design Flow Rate	Design Loading	Annual Costs[a] Capital	O&M	Total	Cost Base	Reference	Effluent Concentration BOD_5	SS
	m^3/d		$/m^3$						
Intermittent Discharge-Chemical Addition									
Canadian Experience	1,140	Alum: 150 mg/l Det. Time: 120 days	0.011[f]	0.021	0.032	1976	(18)	<30	<10
Total Containment Ponds									
Huntington, UT	1,140		0.0959			1975	(6)	No dis-charge	No dis-charge
Weilsville, UT	1,080		0.098			1974	(6)		
Tabiona, UT	114	45 kg BOD_5/ha/d	0.372[e]			1980	(6)	No dis-charge	No dis-charge
Smithfield, UT	3,260	45 kg BOD_5/ha/d	0.148[e]			1979	(6)	No dis-charge	No dis-charge
Southshore, UT	3		0.079			1978	(19)		
Facultative Ponds									
Huntington, UT	1,140		0.087			1975	(6)	Varies with design & time of yr	Varies with design & time of yr
Wardell, MO	303	Primary Cell 38 kg BOD_5/ha/d 2nd Cell 0.3 (Pri. cell surface area) 3rd Cell 0.1 (Pri. cell surface area)	0.077			1974	(14)		
Delta, MO	303	Same as Wardell	0.108			1974	(14)		
Unknown, ID	6,440	22/kg/ha/d	0.082			1978	(19)		
Long Valley, UT	541	45 kg/ha/d	0.135[e]			1979	(6)		
Smithfield, UT	3,250	45 kg/ha/d	0.182[e]			1979	(6)		
Challis, ID	780	45 kg/ha/d in primary cell	0.103[e]			1978	(10)		
Colfax, WA	2,270		0.055[e]				1977	(10)	
Hampton-Princeton, ID	60	45 kg/ha/d	0.201[e]			1979	(10)		
Tensed, ID	114	67 kg/ha/d	0.122[e]			1978	(10)		
Aerated Ponds									
Luxemburg, WI	1,510	Det. time = 35 days	0.082[e]			1977	(16)	Varies with design & time of yr	Varies with design & time of yr
Sugarbush, VA	620		1.127			1974	(20)		
Paw Paw, MI	1,510		0.069			1974	(Anon.)		
Luxemburg, WI	1,510	36 kg BOD_5/d	0.119	0.085	0.203	1978	(21)		
White Bird, ID	114	23 kg BOD_5/d	0.370			1978	(10)		

[a]Costs amortized at 7 percent and a 20-year life.
[b]Values can vary by 50 percent and prices do not include land costs.
[c]Includes land costs with no credit for salvage value.
[d]Excludes sludge disposal costs.
[e]Engineer's estimate.
[f]Amortized at 7 percent and a 10-year life.
[g]Bid but not constructed.

item, such as filter sand or land, can change the relative position of a process dramatically. In brief, Figures 6-1 through 6-3 and Table 6-2 cannot be substituted for good engineering.

All of the processes listed in Table 6-2 are capable of meeting secondary standards, and several are capable of producing a much higher quality effluent. Variations in design and operation also alter the quality of the effluent dramatically in most of the processes. A careful study of all alternatives must be made before selecting a system. The literature referenced herein will provide all details needed, but engineers should remain aware of current developments and use other alternatives as more information becomes available.

6.3 Energy Requirements

Energy consumption is a major factor in the operation of wastewater treatment facilities. Many of the plans for water pollution management in the United States were developed before the cost of energy and the limitations of energy resources became serious concerns for the Nation. As wastewater treatment facilities are built or updated to incorporate current treatment technology and to meet regulatory performance standards, energy must be a major consideration in designing and planning the facilities. Information on energy requirements for various systems must be made available to planners and designers in order that a treatment system may be developed which incorporates the most efficient use of energy for each particular wastewater problem.

6.3.1 Energy Equations

Equations of the lines of best fit for the energy requirements of pond systems based on the data reported by Wesner et al. (22)(25) were used to develop Table 6-3. Details about the conditions imposed upon the equations can be obtained from this reference.

6.3.2 Effluent Quality and Energy Requirements

Table 6-3 shows the expected effluent quality and the energy requirements for various pond systems. Energy requirements and effluent quality are not directly related. Utilizing facultative ponds and land application techniques, it is possible to obtain an excellent quality effluent and expend small quantities of energy.

TABLE 6-3

EXPECTED EFFLUENT QUALITY AND TOTAL ENERGY REQUIREMENTS FOR VARIOUS SIZES AND TYPES OF WASTEWATER TREATMENT PONDS LOCATED IN THE INTERMOUNTAIN AREA OF THE UNITED STATES (23)

Treatment Systems	Effluent Quality, mg/l				Total Energy Requirements at Various Flow Rates											
					0.05 mgd		0.1 mgd		0.5 mgd		1.0 mgd		3.0 mgd		5.0 mgd	
	BOD_5	SS	Total Phos. as P	Total Nitrogen as N	Electricity, kWh/yr	Fuel, Million Btu/yr	Electricity, kWh/yr	Fuel, Million Btu/yr	Electricity, kWh/yr	Fuel, Million Btu/yr	Electricity, kWh/yr	Fuel, Million Btu/yr	Electricity, kWh/yr	Fuel, Million Btu/yr	Electricity, kWh/yr	Fuel, Million Btu/yr
Facultative Pond + Microscreens 23μ	30	30	–	15	11,300	148	20,300	181	83,100	320	154,600	433	419,800	745	670,900	988
Facultative Pond + Intermittent Sand Filter	15	15	–	10	5,840	150	10,920	186	50,540	345	99,270	483	291,800	896	482,200	1,240
Aerated Pond + Intermittent Sand Filter	15	15	–	20	20,800	151	39,500	186	184,800	345	364,500	483	1,079,100	896	1,790,900	1,240
Overland Flow-Facultative Pond Flooding	5	5	5	3	5,700	148	10,700	181	50,070	320	98,810	433	392,600	745	485,080	988
Rapid Infiltration-Facultative Pond Flooding	5	1	2	10	1,540	148	2,810	181	12,140	320	23,050	433	64,300	745	103,900	988
Slow Rate (Irrigation)-Fac. Pond-Ridge and Furrow Flooding	1	1	0.1	3	2,800	149	5,300	183	24,700	330	48,050	453	139,100	805	228,400	1,090

6.4 References

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APPENDIX

EVALUATION OF DESIGN METHODS

A.1 Facultative Ponds

A.1.1 Introduction

A summary of the facultative pond performance data used to evaluate the various design methods is presented in Table A-1. These data were collected for the four facultative pond systems described in Chapter 2. Only the characteristics of the influent wastewater and the effluent from the primary (first) cell of the systems are presented in Table A-1 for the four systems.

Most of the kinetic analyses of the systems are limited to the performance obtained in the primary cell because BOD_5 and COD of the primary cell effluent appear to represent performance of the systems far more than the following cells. Algae succession, changes in nutrient concentration, and the buffering capacity of the total system appear to exert more influence on the cells following the primary cell. The commonly used design methods are discussed individually in the following sections.

Theoretically, most of the models evaluated should have a line of best fit that has an intercept of zero or unity, but an analysis of the data infrequently yields such an ideal relationship. Therefore, all of the attempts to fit the data to a model were evaluated with the least squares technique with an intercept and with the line of best fit forced through an intercept of zero. The equations describing the lines of best fit for both cases are presented on each figure along with the corresponding correlation coefficients. In general, if both correlation coefficients are significant (5 percent level) and approximately equal, it can be assumed that the intercept is approximately zero, and the data describe the model to an acceptable degree.

A.1.2 Empirical Design Equations

In a survey of the first cell of facultative ponds in tropical and temperate zones, McGarry and Pescod (1) found that areal BOD_5 removal (L_r, lb/ac/d) may be estimated through knowledge of areal BOD_5 loading (L_o, lb/ac/d) using

$$L_r = 9.23 + 0.725\ L_o \qquad \text{(A-1)}$$

TABLE A-1

MEAN MONTHLY PERFORMANCE DATA FOR FOUR FACULTATIVE PONDS

MONTH	INF BOD mg/l	CELL #1 BOD	INF SOL BOD	CELL #1 SBOD	INF COD mg/l	CELL #1 COD	INF SOL COD	CELL #1 SCOD	DET TIME DAYS	TEMP °C	LIGHT LAN	TSS mg/l	VSS mg/l	LOCATION
Jan	122	31	40	5	173	118	81	47	44.43	2	190	52	45	Corinne, UT
Feb	107	38	28	5	140	125	64	48	22.72	1	265	61	54	"
Mar	58	57	19	10	135	126	47	39	19.56	5	385	55	52	"
Apr	49	33	16	6	114	113	46	37	23.23	9	495	69	59	"
May	62	33	15	5	105	117	40	38	28.47	12	590	74	61	"
Jun	52	30	17	6	78	95	37	45	27.29	18	630	56	43	"
Jul	40	29	9	5	75	131	35	45	20.73	22	640	65	53	"
Aug	40	36	11	5	81	162	38	44	18.97	19	550	87	76	"
Sep	92	35	25	5	141	168	54	38	21.07	16	480	95	81	Eudora, KS
Oct	87	34	22	4	178	146	50	39	17.64	9	335	85	71	"
Nov	85	30	24	5	132	114	58	42	37.14	4	210	66	57	"
Dec	99	21	32	6	189	89	64	46	63.66	2	145	27	25	"
Jan	140	21	50	9	192	68	88	54	55.76	2	190	18	15	"
Sep	332	41	125	11	633	183	277	83	83.82	22	430	89	76	"
Oct	258	49	116	13	552	226	225	107	87.86	17	285	124	95	"
Nov	303	41	184	11	576	156	265	63	89.00	10	230	79	68	"
Dec	400	53	182	22	614	163	260	71	101.34	4	180	71	65	"
Jan	326	56	169	18	573	174	224	74	102.99	4	190	84	78	"
Feb	303	35	123	10	631	180	234	76	66.08	3	280	74	70	"
Mar	373	49	181	14	635	186	236	60	70.36	7	345	112	97	"
Apr	284	44	129	15	580	172	173	71	80.47	14	440	85	72	"
May	209	57	95	13	375	284	117	76	95.46	21	530	130	102	"
Jun	179	42	78	11	458	204	128	52	101.12	24	560	121	109	"
Jul	270	55	140	19	533	265	201	83	116.83	26	580	172	151	"
Aug	298	69	178	15	544	246	224	91	109.41	25	435	137	120	"
Oct	197	43	70	11	334	203	136	128	48.61	9	240	70	--	Peterborough, NH
Nov	170	46	49	10	303	207	102	82	50.89	6	165	74	--	"
Dec	144	48	49	22	245	155	101	84	50.78	4	115	44	--	"
Jan	123	65	40	51	204	158	94	106	48.29	5	130	27	--	"
Feb	131	70	37	53	201	154	73	100	45.74	3	230	26	--	"
Mar	128	68	43	46	263	151	94	93	37.90	4	280	21	--	"
Apr	101	50	33	36	181	128	78	79	35.87	7	400	23	--	"
May	157	36	73	31	425	110	250	77	42.09	17	450	37	--	"
Jun	133	38	56	21	249	174	116	101	44.61	21	525	47	--	"
Jul	113	37	49	22	223	213	114	126	43.06	24	500	63	--	"
Aug	123	30	56	12	315	230	142	98	43.21	22	450	76	--	"
Sep	137	25	50	9	313	161	125	88	41.30	17	340	49	--	"
Nov	200	23	57	5	254	138	114	68	189.80	14	260	57	--	Kilmichael, MS
Dec	172	24	52	4	333	123	101	52	189.80	9	200	82	--	"
Jan	106	21	36	3	204	120	81	42	189.80	10	205	86	--	"
Feb	135	20	50	3	232	128	81	35	182.50	11	270	74	--	"
Mar	107	27	39	3	225	164	78	30	54.31	12	340	107	--	"
Apr	187	16	55	3	470	103	108	38	115.98	18	450	65	--	"
May	140	17	37	5	312	90	81	49	82.38	23	550	47	--	"
Jun	278	26	80	5	628	89	154	50	185.37	27	530	52	--	"
Jul	278	25	96	6	626	121	192	71	191.48	29	450	43	--	"
Aug	321	31	96	7	746	140	235	94	456.34	29	470	55	--	"
Sep	301	15	74	3	572	52	168	60	171.11	20	370	46	--	"
Oct	247	17	75	4	535	108	164	73	111.93	20	340	33	--	"
Nov	200	14	79	3	458	102	191	67	165.37	18	250	28	--	"

The regression equation had a correlation coefficient of 0.995 and a 95 percent confidence interval of $\pm$ 33 kg/ha/d (29 lb BOD_5/ac/d) removal (Figure A-1). The equation was reported to be valid for any loading between 34 and 560 kg BOD_5/ha/d (30 and 500 lb BOD_5/ac/d). McGarry and Pescod (1) also found that, under normal operating ranges, hydraulic detention time and pond depth have little influence on percentage or areal BOD_5 removal. With such a large 95 percent confidence interval, it is impractical to apply the equation to pond systems loaded at rates of 34 kg/ha/d (30 lb BOD_5/ac/d) or less as was the situation with the majority of the months of operation for the four facultative pond systems described previously.

Relationships between organic removal and organic loading for the lower rates observed at the four facultative pond systems were developed using BOD_5, soluble biochemical oxygen demand ($SBOD_5$), chemical oxygen demand (COD), and soluble chemical oxygen demand (SCOD). Statistically significant relationships were observed for all four organic carbon estimating analyses, but the best relationships were observed when the organic removals were calculated using the influent BOD_5 and the effluent $SBOD_5$ ($ITBOD_5$ and $ESBOD_5$) and the influent COD and the effluent SCOD (ITCOD and ESCOD). The BOD_5 and $SBOD_5$ relationship is shown in Figure A-2, and the COD and SCOD relationship is shown in Figure A-3. The 95 percent confidence intervals for the BOD_5-$SBOD_5$ and the COD-SCOD relationships are much smaller than the value reported by McGarry and Pescod (1) and are shown in Figures A-2 and A-3.

Larsen (2) proposed an empirical design equation, developed by using data from a one-year study at the Inhalation Toxicology Research Institute, Kirtland Air Force Base, NM. The Institute's facultative pond system consists of one 0.66 ha (1.62 ac) cell receiving waste from 151 staff members, 1,300 beagle dogs, and several thousand small animals. Larsen found that the required pond surface area could be estimated by use of the following equation.

$$MOT = (2.468^{RED} + 2.468^{TTC} + 23.9/TEMPR + 150.0/DRY)*10^{6} \qquad (A-2)$$

where the dimensionless products are:

$$MOT = \frac{(\text{surface area, ft}^2)\ (\text{solar radiation, Btu/ft}^2/\text{d})^{1/3}}{(\text{influent flow rate, gal/d})\ (\text{influent BOD}_5,\ \text{mg/l})^{1/3}} \times (1.0783 \times 10^{7})$$

$$RED = \frac{(\text{influent BOD}_5,\ \text{mg/l}) - (\text{effluent BOD}_5,\ \text{mg/l})}{(\text{influent BOD, mg/l})}$$

$$TTC = \frac{(\text{windspeed, miles/hr})\ (\text{influent BOD}_5,\ \text{mg/l})^{1/3}}{(\text{solar radiation, Btu/ft}^2/\text{d})^{1/3}} \times 0.0879$$

FIGURE A-1

MCGARRY AND PESCOD EQUATION FOR AREAL BOD_5 REMOVAL AS A FUNCTION OF BOD_5 LOADING

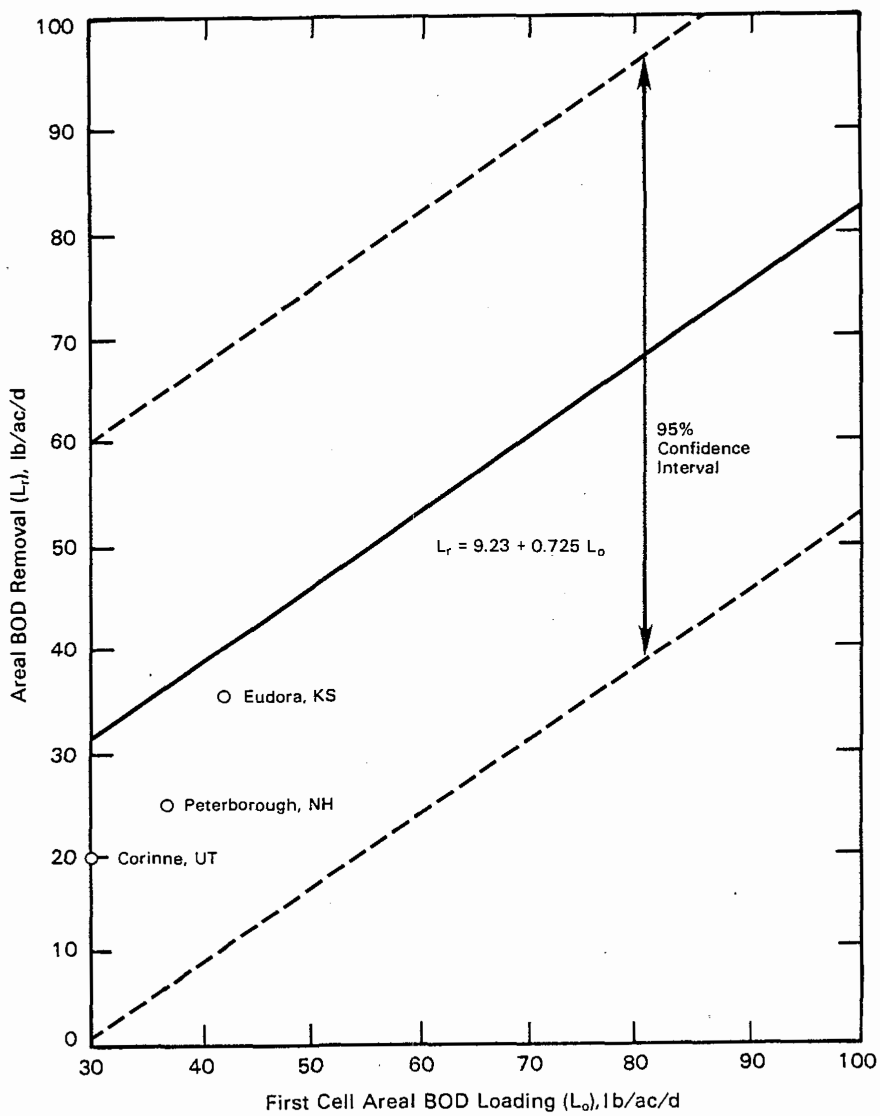


FIGURE A-2

RELATIONSHIP BETWEEN BOD_5 LOADING AND REMOVAL RATES - FACULTATIVE PONDS

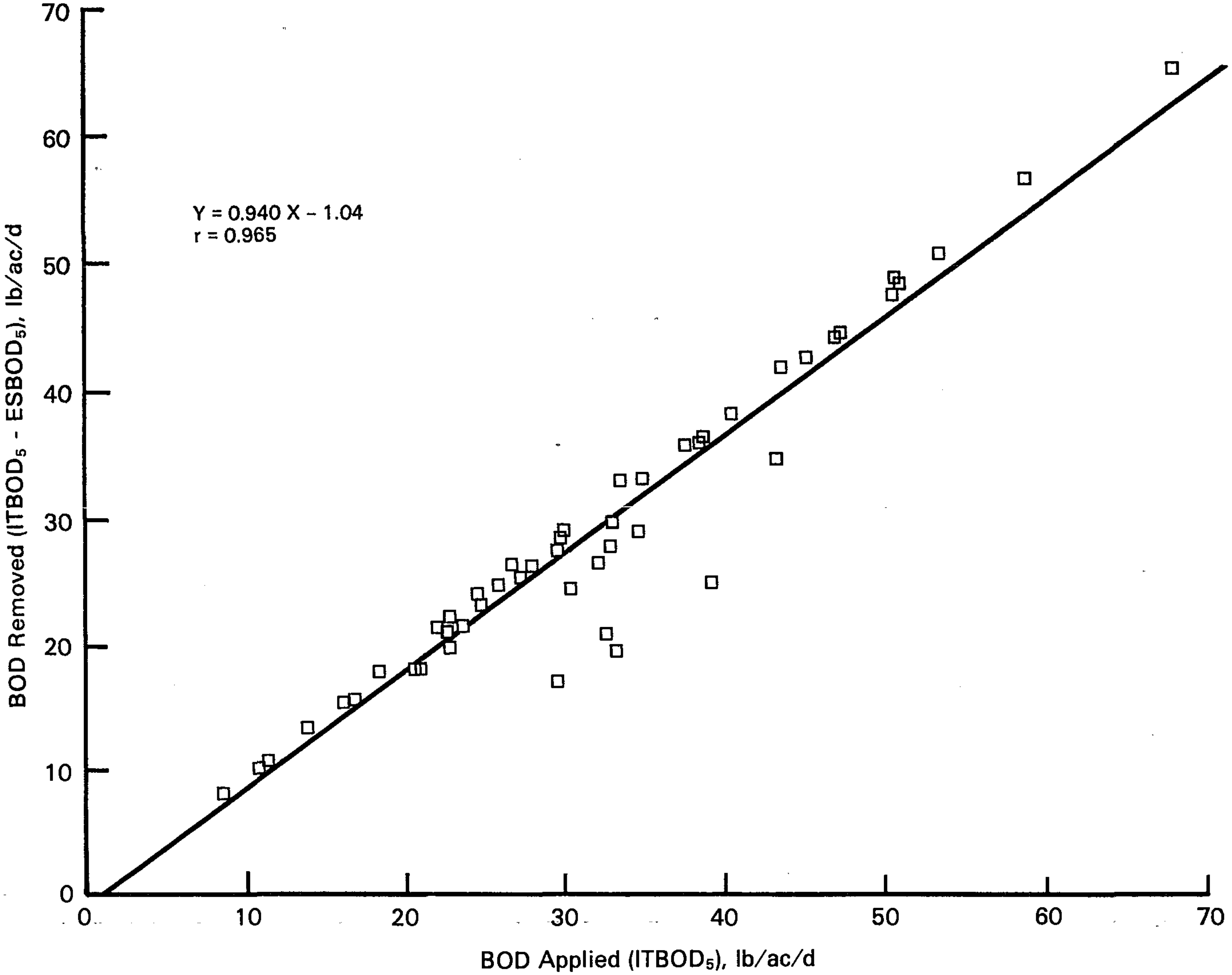
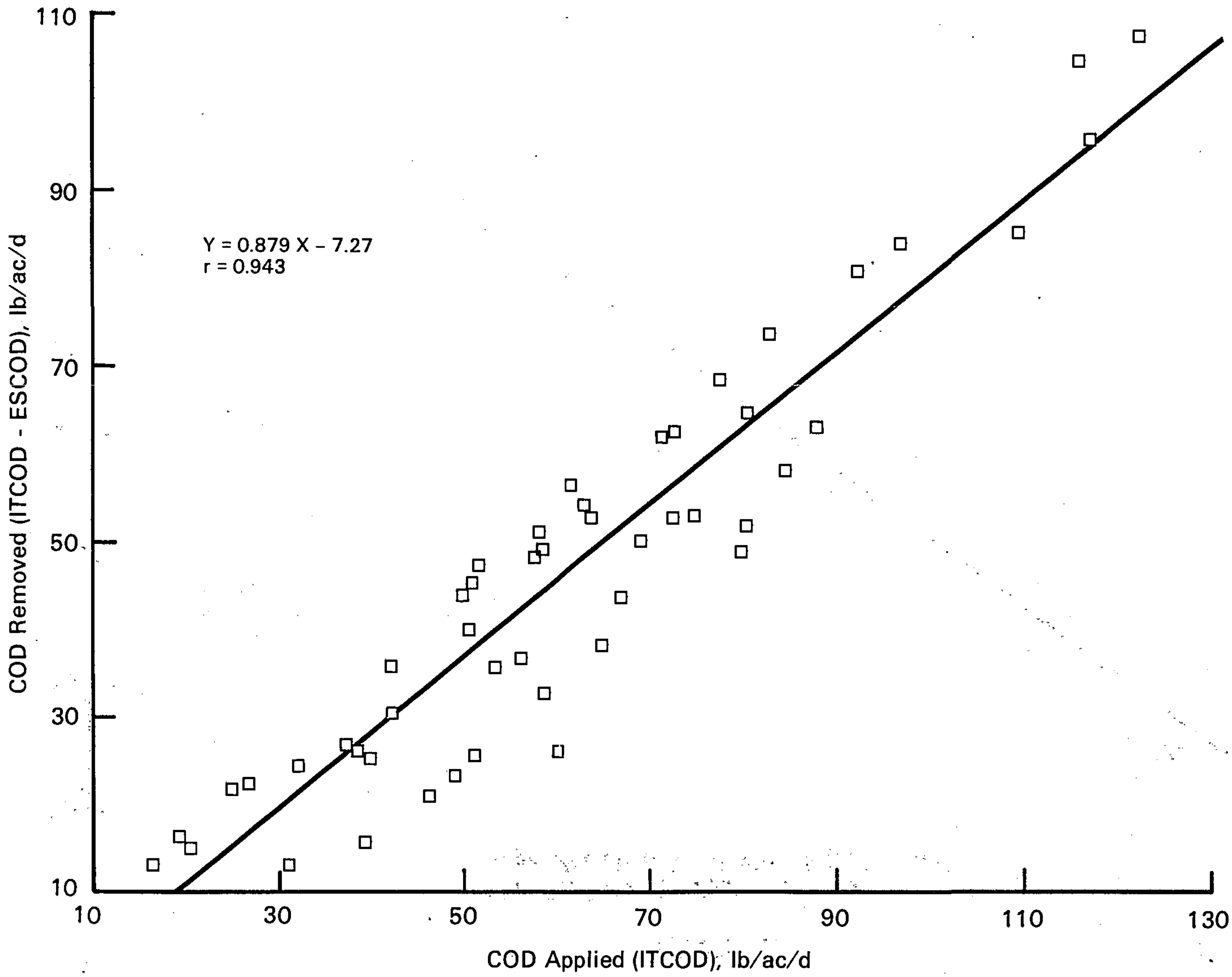


FIGURE A-3

RELATIONSHIP BETWEEN COD LOADING AND
REMOVAL RATES - FACULTATIVE PONDS



$$TEMPR = \frac{(\text{pond liquid temperature, °F})}{(\text{air temperature, °F})}$$

DRY = relative humidity, percent

To determine the effect of using the Larsen equation on multicell facultative ponds, it was applied to both the entire system and the primary cell for each of the four locations. In each case the Larsen equation underestimated the pond surface area required for a particular BOD_5 removal. Prediction errors for multiple cell ponds ranged from 190 to 248 percent. Prediction errors for the primary cell surface areas ranged from 18 to 98 percent. Use of the Larsen equation is not recommended.

Gloyna (3) proposed the following empirical equation for the design of facultative wastewater stabilization ponds:

$$V = 3.5 \times 10^{-5} \, QL_a \left[\theta^{(35-T)}\right] ff' \qquad \text{(A-3)}$$

where

V = pond volume (m^3)

Q = influent flow rate (liters/day)

L_a = ultimate influent BOD_u or COD (mg/l)

θ = temperature coefficient

T = pond temperature (°C)

f = algal toxicity factor

f' = sulfide oxygen demand factor

The BOD_5 removal efficiency can be expected to be 80 to 90 percent based on unfiltered influent samples and filtered effluent samples. A pond depth of 1.5 m is suggested for systems with significant seasonal variations in temperature and major fluctuations in daily flow. Surface area design using the Gloyna equation should always be based on a 1-m depth. The additional 0.5 m of depth is provided to store sludge. According to Gloyna (3), the algal toxicity factor (f) can be assumed to be equal to 1.0 for domestic wastes and many industrial wastes. The sulfide oxygen demand (f') is also equal to 1.0 for $SO_4^{=}$ ion concentration of less than 500 mg/l. Gloyna (3) also suggests using the average temperature of the pond in the critical or coldest month. In this equation, sunlight is not considered to be critical in pond design but may be incorporated into the Gloyna equation by multiplying the pond volume by the ratio of sunlight in the particular area to the average found in the Southwest.

The data used to evaluate the Gloyna equation are shown in Table A-1. Although ultimate BOD data were not available, COD, SCOD, BOD_5, and $SBOD_5$ data were used. Use of the Gloyna equation with the data in Table A-1 failed to produce any good relationships. The relationships obtained with the COD, SCOD, BOD_5, and $SBOD_5$ data were statistically significant, but the data points were scattered. The relationship shown in Figure A-4 was the best fit obtained for the data, and the resulting design equation follows:

$$V/Q = 0.035\ (BOD_5,\ mg/l)\ (1.099)^{\frac{LIGHT\ (35-T)}{250}} \qquad (A-4)$$

t = detention time, days

$V/Q = t$

where

T = temperature, °C

LIGHT = solar radiation, langleys/day

V = volume of primary pond, m^3

Q = flow rate, m^3/d

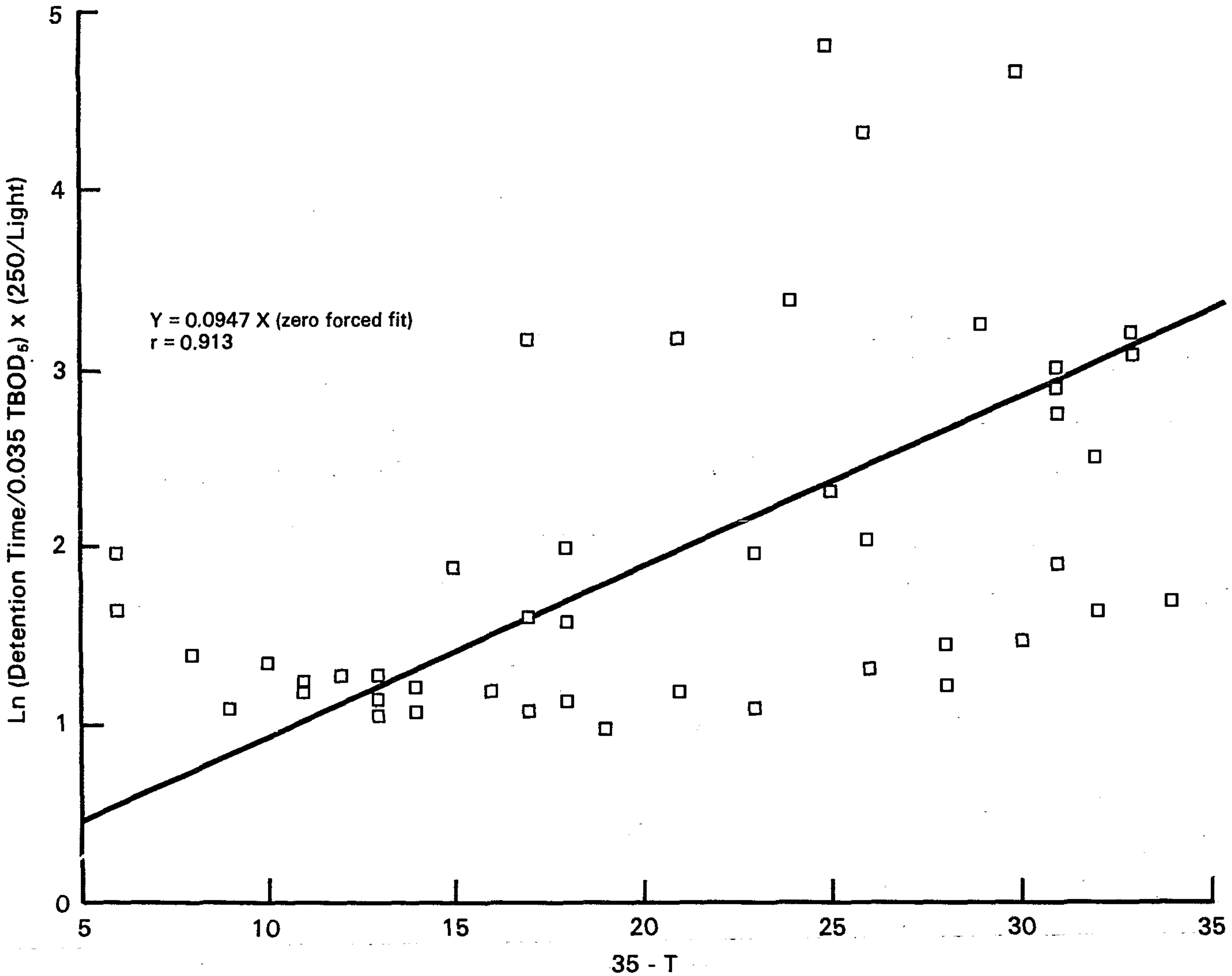
The validity of the above expression is questionable because of the scattered data, but the relationship is statistically significant. The use of this relationship will be left to the discretion of the design engineer.

Although not directly comparable, the results obtained with this equation and results obtained with the relationship shown in Figure A-2 are presented below to show the variation between the two approaches.

Assuming a design flow rate of 3,785 m^3/d (1 mgd), a solar radiation intensity of 250 langleys, an influent BOD_5 concentration of 300 mg/l, and a temperature of 10°C, Equation A-4 yields a surface area of 420,918 m^2 (assuming a water depth of 1 m), and the organic loading rate relationship (Figure A-2) yields a loading rate of 27 kg/ha/d (24 lb/ac/d). At this rate, organic removal will be 24.1 kg/ha/d (21.5 lb/ac/d) (Figure A-2) or an 89.7 percent reduction in BOD_5. The percent reduction is within the range of 80 to 90 percent expected with a design using the Gloyna equation. The results obtained with Equation A-4 appear to be conservative when compared with the organic loading-removal relationship (Figure A-2) principally because of the temperature correction factor in Equation A-4. Although it is logical to expect temperature to exert an influence on BOD_5 removal, the plot shown in Figure A-2 was unaffected when temperature relationships were incorporated into the relationship. The most logical explanation for this phenomenon is that the systems are so large that the temperature influence is masked in the process. This observation is also pursued in the following section.

FIGURE A-4

RELATIONSHIP OBTAINED WITH MODIFIED GLOYNA EQUATION - FACULTATIVE PONDS (EQUATION A-4)



A.1.3 Rational Design Equations

Kinetic models based on plug flow and complete mix hydraulics and first order reaction rates have been proposed by many authors to describe the performance of wastewater stabilization ponds. The basic models are modified to reflect the influence of temperature. The basic models are:

Plug Flow:

$$\frac{C_e}{C_o} = e^{-k_p t} \tag{A-5}$$

or: $$\ln C_e/C_o = -k_p t \tag{A-6}$$

Complete Mix:

$$\frac{C_e}{C_o} = \frac{1}{1 + k_c t} \tag{A-7}$$

or: $$\left(\frac{C_o}{C_e} - 1\right) = k_c t \tag{A-8}$$

where

C_o = influent BOD_5 concentration, mg/l

C_e = effluent BOD_5 concentration, mg/l

k_p = plug flow first order reaction rate constant, $time^{-1}$

t = hydraulic residence time, days

e = base of natural logarithms, 2.7183

k_c = complete mix first order reaction rate constant, $time^{-1}$

The influence of temperature on the reaction rate constants is most frequently expressed by using the Arrhenius (4) relationship:

$$\frac{d\,(\ln k)}{dt} = \frac{E_a}{RT} \tag{A-9}$$

where

k = reaction rate constant

E_a = activation energy, calories/mole

R = ideal gas constant, 1.98 calories/mole-degree

T = reaction temperature, °Kelvin

Integrating Equation A-9 yields the following expression:

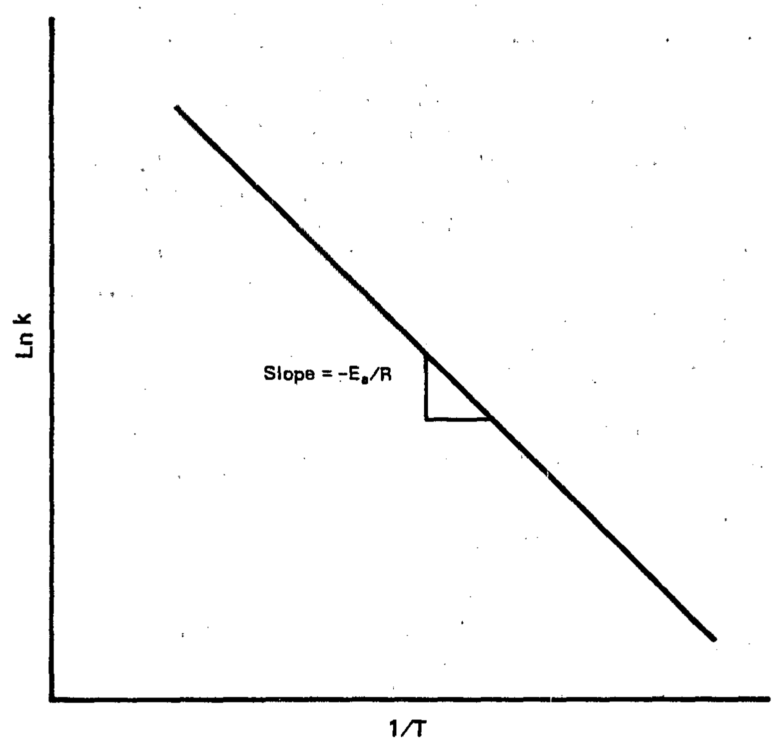
$$\ln k = -\frac{E_a}{RT} + \ln B \qquad \text{(A-10)}$$

where B is a constant.

Experimental data can be plotted as shown in Figure A-5 to determine the value of E_a. Equation A-9 can be integrated between the limits of T_1 and T_2 to obtain Equation A-11.

FIGURE A-5

ARRHENIUS PLOT TO DETERMINE ACTIVATION ENERGY



$$\ln\left(\frac{k_2}{k_1}\right) = \frac{E_a}{R}\left(\frac{T_2 - T_1}{T_2 T_1}\right) \qquad \text{(A-11)}$$

In most biological wastewater treatment processes, it is assumed that E_a/RT_2T_1 is a constant, C, and Equation A-11 reduces to

$$\ln\left(\frac{k_2}{k_1}\right) = C\,(T_2 - T_1) \qquad \text{(A-12)}$$

or

$$\frac{k_2}{k_1} = e^{C\,(T_2 - T_1)} \qquad \text{(A-13)}$$

or

$$\frac{k_2}{k_1} = \theta^{(T_2 - T_1)} \qquad \text{(A-14)}$$

where θ = temperature factor. Plotting experimental values of the natural logarithms of k_2/k_1 versus (T_2-T_1) as shown in Figure A-6, the value of θ can be determined.

The plug flow and complete mix models, along with modifications suggested by various investigators, were evaluated using the data shown in Table A-1.

The influence of temperature on the calculated reaction rates was evaluated. As shown in Figures A-7 and A-8, the reaction rates calculated with the plug flow and complete mix equations were essentially independent of the temperature. This lack of influence by the liquid temperature was also observed in the section on empirical design equations (Figures A-2 and A-3). The logical explanation for the lack of influence by temperature is that the pond systems are so large that the temperature effect is masked by other factors. There is no doubt that temperature influences biological activity, but for the systems listed in Table A-1, the influence was overshadowed by other parameters that may include dispersion, detention time, light, species of organisms, etc.

After observing the lack of influence by the water temperature, the data in Table A-1 were fitted to the plug flow (Equation A-5) and the complete mix (Equation A-7) models to determine if the systems could be defined by these simple relationships. As shown in Figures A-9 through A-12, the fit of the data is less than ideal but is statistically highly significant (1 percent level). Further attempts to incorporate various types of temperature and light intensity relationships into the plug flow and complete mix models were not successful. The best statistical relationships obtained with the data are shown in Figures A-9 thorugh A-12.

Thirumurthi (5) stated that a kinetic model based on plug flow or complete mix hydraulics should not be used for the rational design of stabilization ponds. Thirumurthi found that facultative ponds exhibit nonideal flow patterns and recommended the use of the following chemical reactor equation developed by Wehner and Wilhelm (6) for pond design:

$$\frac{C_e}{C_o} = \frac{4ae^{(1/2)D}}{(1+a)^2\ e^{a/2D} - (1-a)^2\ e^{-a/2D}} \tag{A-15}$$

where

C_e = effluent BOD_5, mg/l

C_o = influent BOD_5, mg/l

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

k = first order BOD_5 removal coefficient, day^{-1}

FIGURE A-6

PLOT OF REACTION RATE CONSTANTS AND TEMPERATURE TO DETERMINE THE TEMPERATURE FACTOR

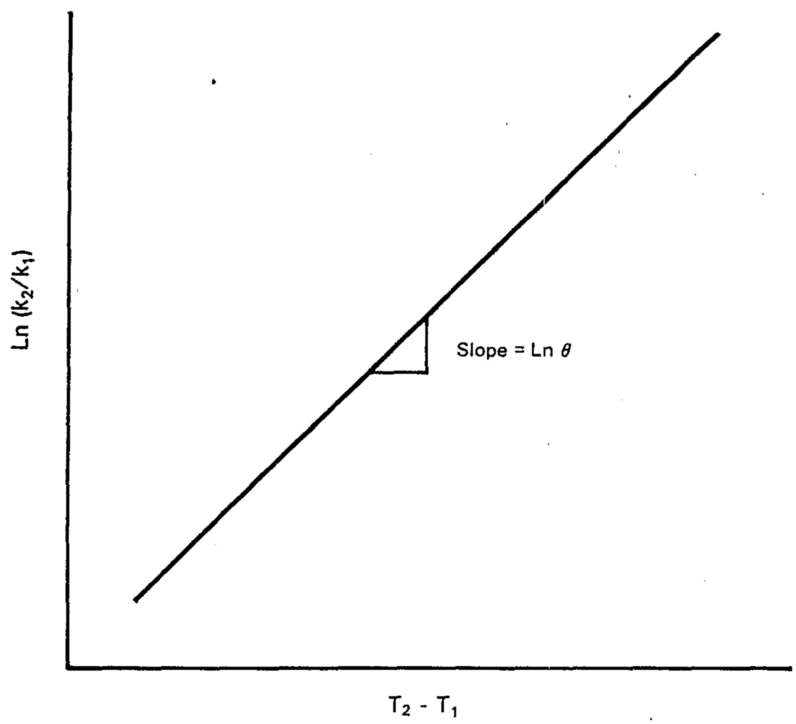


FIGURE A-7

RELATIONSHIP BETWEEN PLUG FLOW DECAY RATE AND TEMPERATURE - FACULTATIVE PONDS

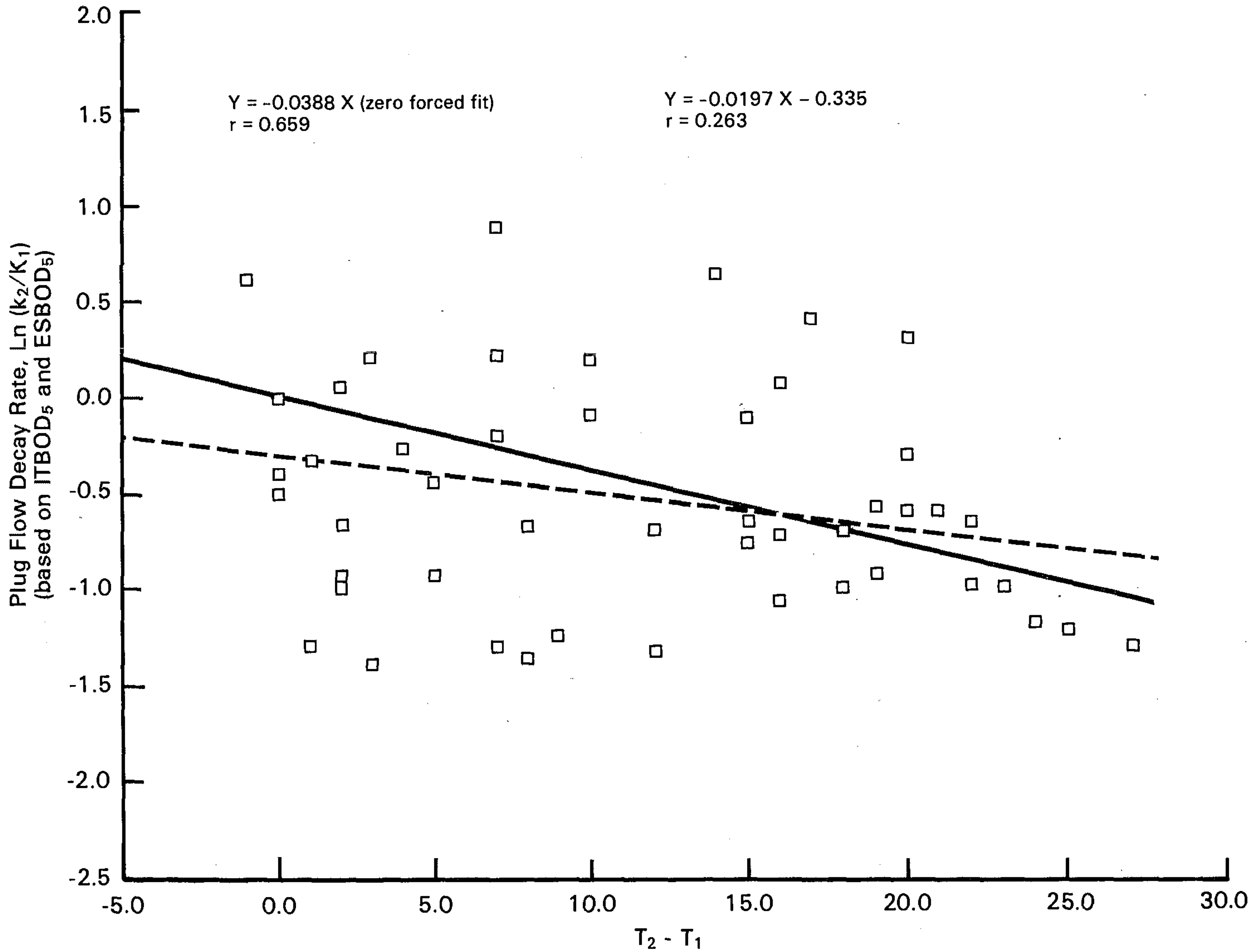


FIGURE A-8

RELATIONSHIP BETWEEN COMPLETE MIX DECAY RATE AND TEMPERATURE - FACULTATIVE PONDS

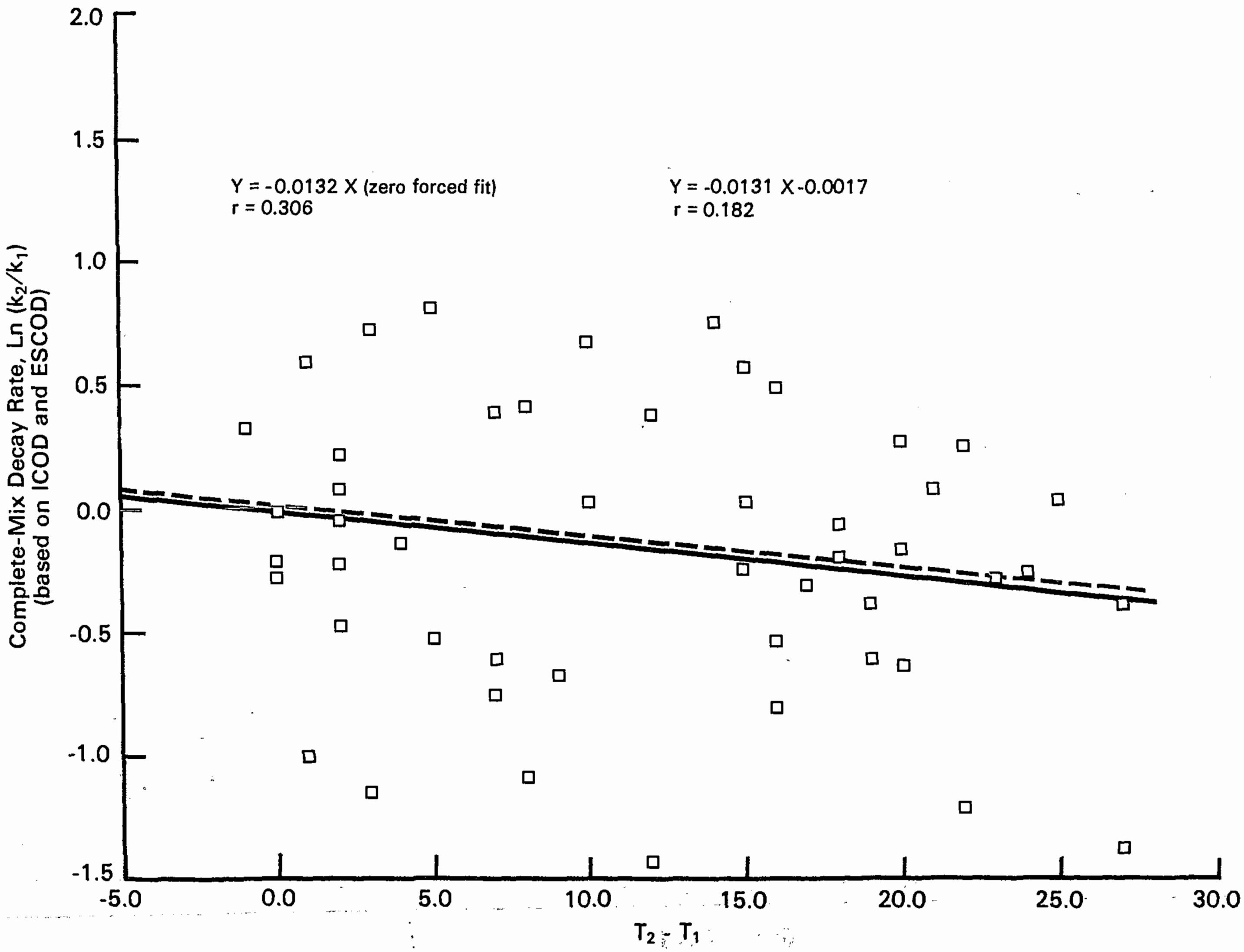


FIGURE A-9

PLUG FLOW MODEL - FACULTATIVE PONDS

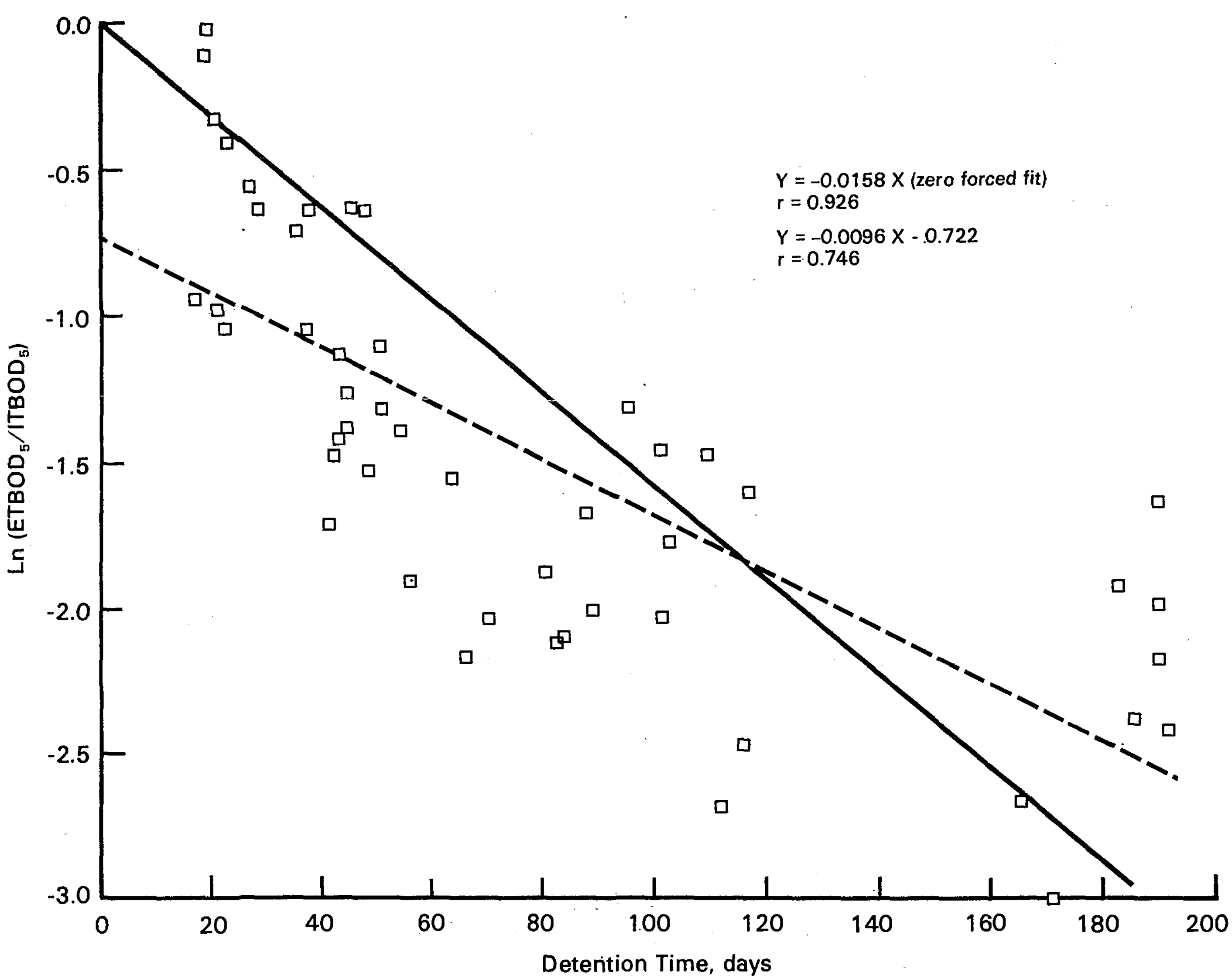


FIGURE A-10

PLUG FLOW MODEL - FACULTATIVE PONDS

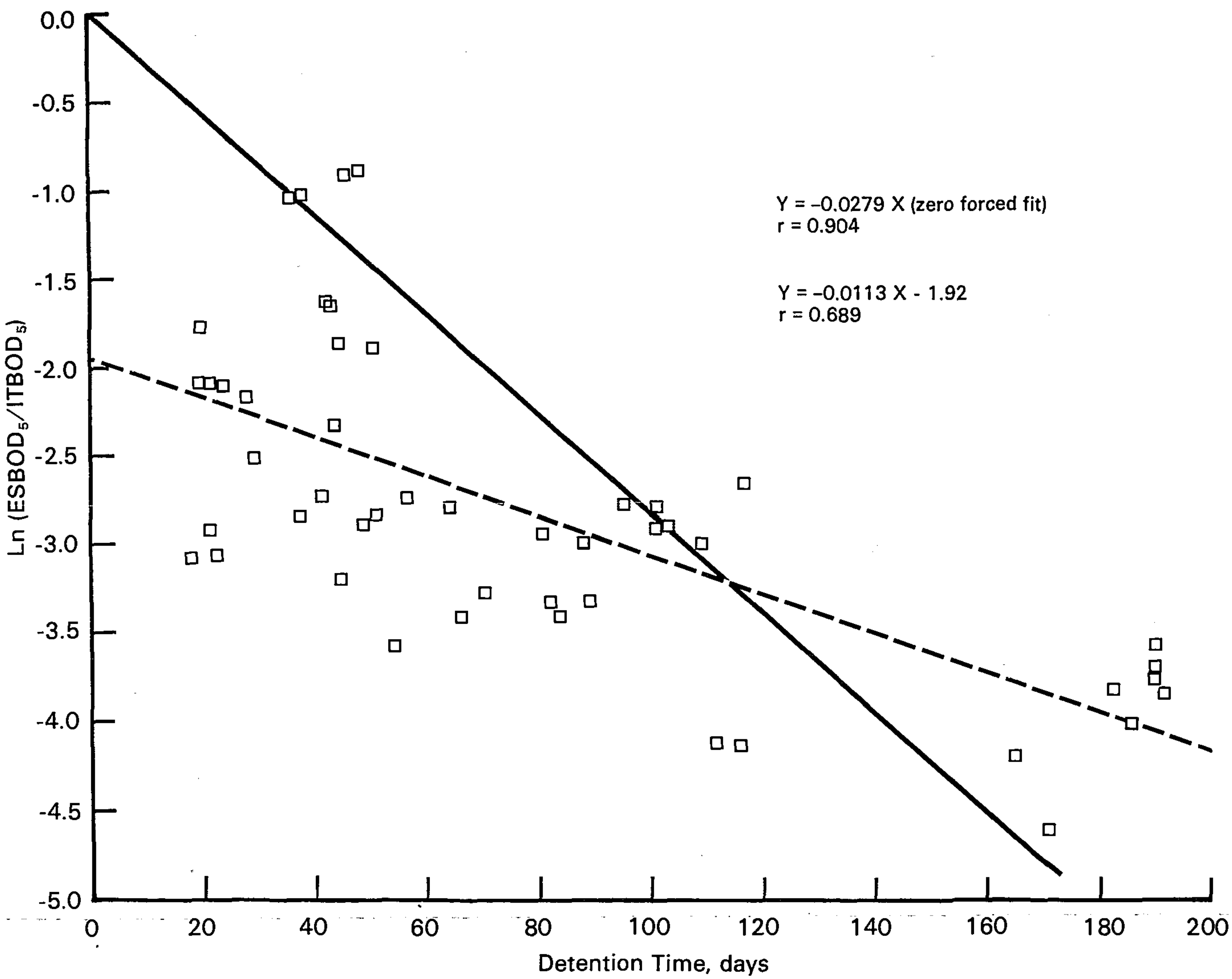


FIGURE A-11

COMPLETE MIX MODEL - FACULTATIVE PONDS

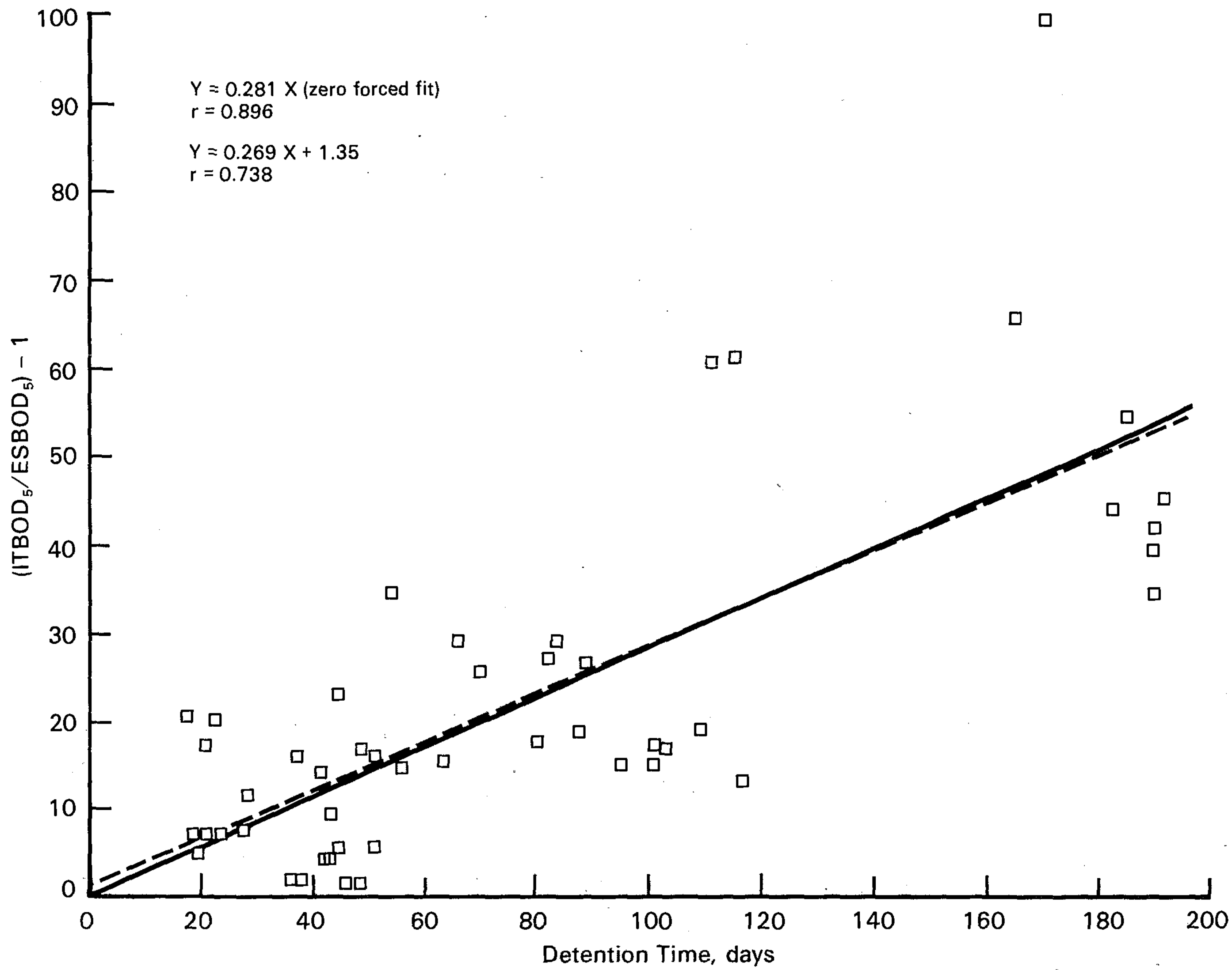
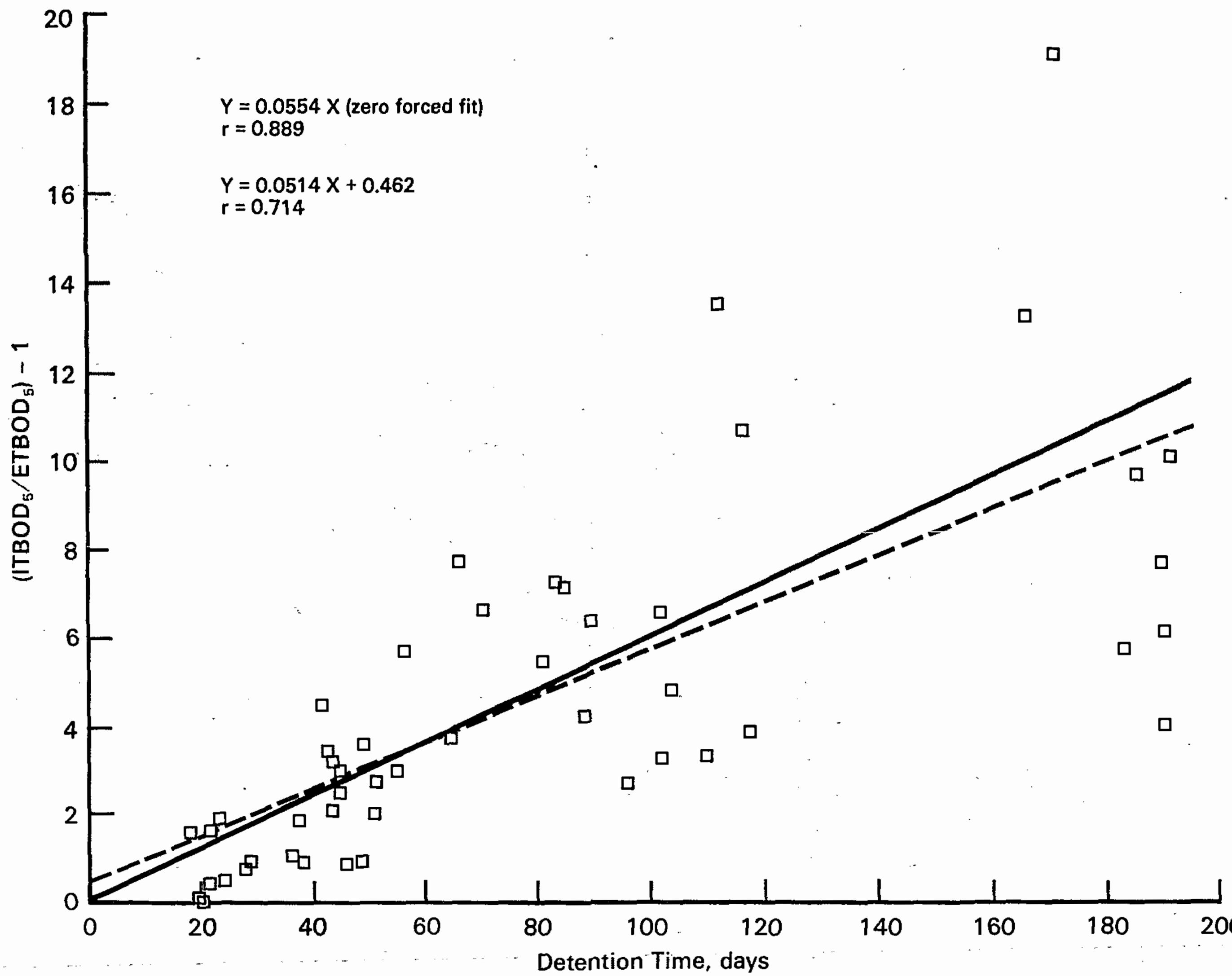


FIGURE A-12

COMPLETE MIX MODEL - FACULTATIVE PONDS



t = mean detention time, days

D = dimensionless dispersion number

$$D = \frac{H}{vL} = \frac{Ht}{L^2}$$

where

H = axial dispersion coefficient, area/time

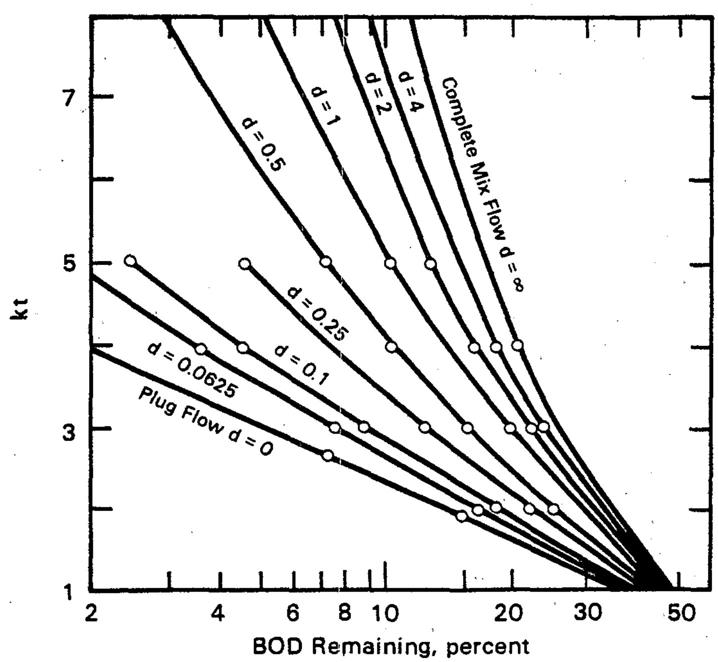
v = fluid velocity, length/time

L = length of travel path of typical particle, length

Thirumurthi (5) prepared the chart shown in Figure A-13 to facilitate the use of Equation A-15. The dimensionless term "kt" is plotted versus the percent BOD_5 remaining for dispersion numbers varying from zero for an ideal plug flow unit to infinity for a complete mix unit. Dispersion numbers measured in stabilization ponds range from 0.1 to 2.0 with most values less than 1.0.

FIGURE A-13

WEHNER AND WILHELM EQUATION CHART



The data in Table A-1 were used to calculate values of k for three different dispersion numbers (0.1, 0.25, and 1.0) by an iterative process. The values of k were normalized and plotted versus temperature as illustrated in Figure A-14. Less than 12 percent of the regression was explained by a linear relationship indicating that temperature exerts little influence on the performance of the ponds.

Values of k calculated using the influent total BOD_5 and the effluent soluble BOD_5 concentrations with a dispersion number of 0.25 ranged from 0.011 to 0.282/d with a mean value of 0.073/d and a median value of 0.055/d. The mean temperature was 13.5°C and the median temperature was 12°C. Using the design data presented with the Gloyna equation (Q = 3.785 m^3/d, BOD_5 = 300 mg/l, and T = 10°C), a k of 0.055/d adjusted linearly for temperature to yield a value of 0.046/d (10°C/12°C x 0.055 = 0.046), a dispersion number of 0.25, and, assuming 90 percent removal of the BOD_5 ($TBOD_5$-$SBOD_5$), a detention time of 74 days is obtained. This detention time is less than the value of 100 days obtained with the Gloyna equation (Equation A-4), but considering the differences in approach, agreement is reasonable.

A.2 Aerated Ponds

A.2.1 Introduction

A summary of the partial mix aerated pond performance data used to evaluate the various design methods is presented in Table A-2. These data were collected at the five aerated pond systems described in Chapter 2. Only the characteristics of the influent wastewater and the effluent from the primary (first) cell of the systems are presented in Table A-2.

Most of the kinetic analyses of the systems are limited to the performance obtained in the primary cells because BOD_5 and COD of the primary cell effluent appear to represent performance of the systems far more than the following cells. Species succession, nutrient concentrations, and the buffering capacity of the systems appear to have more of an effect on the cells following the primary cell. The more commonly used design methods for aerating ponds are discussed individually in the following paragraphs.

A.2.2 Design Equations

The most commonly used aerated pond design equation is presented in the Ten State Standards (7).

$$t = \frac{E}{2.3\ k_1\ (100-E)} \qquad \text{(A-16)}$$

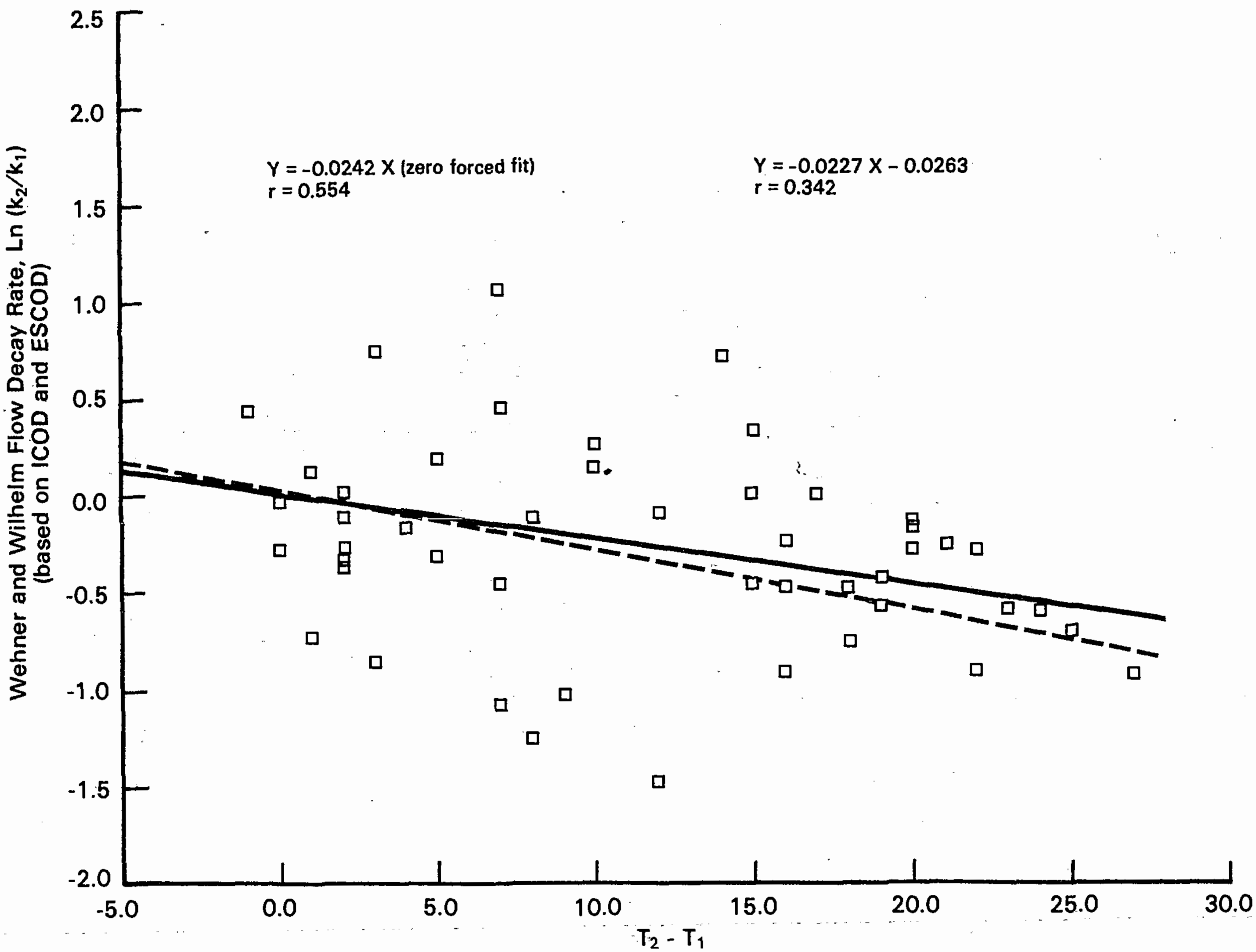
TABLE A-2

MEAN MONTHLY PERFORMANCE DATA FOR FIVE PARTIAL MIX AERATED PONDS

MONTH	INF BOD mg/l	CELL #1 BOD	INF SOL BOD	CELL #1 SBOD	INF COD mg/l	CELL #1 COD	INF SOL COD	CELL #1 SCOD	DET TIME DAYS	TEMP °C	LIGHT LAN	TSS mg/l	VSS mg/l	LOCATION
Jan	368	68	222	55	664	152	321	68	66.59	5	190	93	64	Bixby, OK
Feb	422	150	244	87	524	186	368	133	68.26	10	250	63	40	"
Mar	414	60	227	40	671	192	319	101	60.26	16	350	101	63	"
Apr	392	98	127	23	610	212	215	79	45.87	19	435	74	59	"
May	379	81	136	9	552	202	240	61	52.49	20	590	74	74	"
Jun	413	87	122	13	606	175	267	104	66.94	26	600	61	53	"
Jul	355	58	140	14	594	138	254	54	59.92	29	560	67	50	"
Aug	313	64	105	9	589	180	223	62	33.32	28	550	89	76	"
Sep	330	87	142	5	646	285	259	68	52.89	24	470	109	78	"
Oct	388	106	136	8	773	268	278	57	59.58	13	350	166	140	"
Nov	364	117	147	7	774	254	249	71	55.28	9	265	127	103	"
Dec	283	61	119	9	619	178	224	67	49.85	7	200	59	44	"
Apr	277	13	58	7	440	86	83	61	85.15	5	520	56	20	Pawnee, IL
May	468	33	75	17	753	131	146	89	77.53	12	485	51	33	"
Jun	470	20	89	10	1147	113	231	63	82.36	19	415	41	29	"
Jul	452	16	130	10	1208	82	252	60	86.40	23	555	33	24	"
Aug	602	20	136	11	1921	143	361	99	83.69	26	510	43	30	"
Sep	578	28	143	10	1208	132	298	68	88.44	19	395	58	41	"
Oct	799	9	127	5	1572	74	290	62	86.61	11	255	31	17	"
Nov	548	9	129	5	1156	77	294	58	96.76	5	220	53	27	"
Dec	554	9	126	4	1115	78	246	70	87.09	3	165	23	16	"
Jan	395	10	97	5	779	68	213	60	115.11	3	245	17	8	"
Feb	296	18	83	11	649	75	170	59	76.36	8	320	37	21	"
Mar	233	14	39	7	367	89	53	61	66.57	6	345	51	18	"
Nov	177	19	74	15	267	37	58	33	29.73	19	140	19	12	Windber, PA
Dec	203	43	76	42	376	48	97	36	29.53	14	110	21	13	"
Jan	152	30	61	25	211	57	74	47	14.81	12	135	38	28	"
Feb	220	68	110	65	192	52	64	35	14.47	12	225	32	22	"
Mar	186	38	81	34	206	56	75	38	15.71	13	275	25	18	"
Apr	155	46	58	21	270	95	135	16	14.52	14	470	52	37	"
May	202	66	108	58	435	207	211	97	20.00	20	460	104	72	"
Jun	182	69	103	46	431	190	190	101	22.92	26	500	123	81	"
Jul	145	64	91	44	356	137	210	77	16.24	25	510	51	31	"
Aug	172	48	82	37	749	109	199	65	15.71	24	470	60	41	"
Sep	173	39	69	14	1062	131	242	52	16.48	22	345	88	73	"
Oct	106	45	66	34	537	146	191	52	15.94	17	225	26	21	"
Dec	89	11	34	--	209	34	34	22	26.15	4	85	7	6	Lake Koshkonong, WI
Jan	96	12	46	--	229	52	46	44	28.91	1	150	5	4	"
Feb	75	10	41	--	244	45	41	37	25.74	1	225	7	6	"
Mar	37	14	29	--	116	52	29	30	18.82	4	290	22	20	"
Apr	55	12	31	--	127	48	31	28	20.30	11	415	16	15	"
May	71	18	26	--	160	54	26	24	22.41	12	510	19	18	"
Jun	86	14	37	--	186	56	37	29	22.65	21	590	20	19	"
Jul	93	23	34	--	229	74	34	36	22.86	23	575	30	27	"
Aug	118	25	18	--	299	44	18	25	22.77	22	530	12	10	"
Sep	113	34	21	--	194	48	21	26	23.40	16	400	20	13	"
Oct	102	25	18	--	200	36	18	21	24.23	12	255	6	5	"
Nov	87	17	38	--	168	32	38	27	48.95	7	180	6	5	"
Dec	178	49	94	32	510	105	144	60	7.06	12	240	38	33	North Gulfport, MS
Jan	214	63	88	41	457	96	120	56	6.04	14	250	37	33	"
Feb	199	68	86	46	454	105	115	55	8.22	19	365	50	44	"
Mar	192	76	117	35	369	98	138	52	8.21	16	280	53	41	"
Apr	178	67	79	36	431	105	112	55	7.64	22	505	52	43	"
May	175	58	76	31	291	120	102	56	7.03	23	490	51	35	"
Jun	171	66	71	37	359	101	92	46	6.73	28	550	98	63	"
Jul	134	53	55	30	203	80	69	46	5.75	29	480	56	33	"
Aug	151	56	68	36	201	86	86	51	5.84	29	475	61	42	"
Sep	170	62	74	38	230	82	89	49	7.19	27	400	42	30	"
Oct	171	82	74	41	221	101	86	57	9.30	24	345	75	55	"
Nov	206	73	119	42	324	108	151	69	7.84	18	255	40	27	"

FIGURE A-14

RELATIONSHIP BETWEEN WEHNER AND WILHELM DECAY RATE AND TEMPERATURE - FACULTATIVE PONDS



where

t = detention time, days

E = percent BOD_5 to be removed in an aerated pond

k^1 = reaction coefficient, aerated pond, base 10. [For normal domestic wastewater, the K_1 value may be assumed to be 0.12/d at 20°C and 0.06/d at 1°C according to the standards (7).]

Equation A-16 is equivalent to Equation A-8 presented in the section on Rational Design Equations for facultative wastewater stabilization pond design. By manipulating Equation A-16 as shown below, it can be shown that the two equations are equal.

$$2.3\ k_1 t = \frac{E}{100-E} \qquad \text{(A-17)}$$

$$E = \frac{C_o - C_e}{C_o} \times 100 \qquad \text{(A-18)}$$

$$2.3\ k_1 t = \frac{\left(\frac{C_o - C_e}{C_o}\right) 100}{100 - \left[\left(\frac{C_o - C_e}{C_o}\right) 100\right]} \qquad \text{(A-19)}$$

$$2.3\ k_1 t = \frac{100 - 100\ C_e/C_o}{100 - 100 + 100\ C_o/C_e} \qquad \text{(A-20)}$$

$$2.3\ k_1 t = \frac{C_o}{C_e} - 1 \qquad \text{(A-21)}$$

The only difference between Equation A-16 and Equation A-8 is that the constant in Equation A-8 is expressed in terms of base e and the other is expressed in terms of base 10.

Equation A-8 is the most commonly used equation to design aerated ponds. Practically every aerated pond in the United States has been designed with this simple complete mix model. In the design process, the reaction rate coefficient is adjusted to reflect the influence of temperature on the biological reactions in the aerated ponds by using Equation A-14.

The plug flow (Equation A-5), complete mix (Ten State Equation and Equation A-8), and Wehner-Wilhelm (Equation A-15) models were evaluated using the BOD and COD data shown in Table A-2. The effect of water temperature on the reaction rates calculated with each of the models using the influent total

BOD_5 and the effluent $SBOD_5$ are shown in Figures A-15 through A-17. The Wehner-Wilhelm equation was solved using a dispersion number of 0.25.

Although the relationships between decay rates and temperature differences are statistically significant at the 1 or 5 percent level, the data points are scattered and less than 30 percent of the variation is explained by the regression analyses. Using the analyses for the regression of data through the origin, temperature factors (θ) of 1.07, 1.09, and 1.07 were obtained for the plug flow, complete mix, and Wehner-Wilhelm models, respectively, using the influent total BOD_5 and effluent soluble BOD_5 to estimate the substrate strength, respectively. The temperature factors are in agreement with values reported in the literature (3)(8)(9). The above values are recommended for use when adjusting the reaction rates to compensate for temperature effects in aerated ponds.

The relationships shown in Figures A-15 through A-17 were the best obtained for all combinations of the BOD and COD data. Better relationships were observed between the reaction rates and temperature for the aerated ponds than for the facultative ponds. This was probably attributable to the mixing in the aerated ponds. The relationships between the plug flow and Wehner-Wilhelm models' reaction rates and temperature provided a better fit of the data than the relationship observed for the complete mix model. These results indicate that the plug flow and Wehner-Wilhelm models provide a better approximation of the performance of the aerated ponds than the complete mix model. All of the aerated ponds were designed using the complete mix model (Equation A-8).

Because of the relatively poor relationship between the reaction rates and temperature, the data in Table A-2 were used to determine if the plug flow and complete mix models could be used to estimate the performance of the aerated ponds. As shown in Figures A-18 and A-19, the fit of the data to both the plug flow and complete mix models yields statistically significant (1 percent level) relationships. The data fit the plug flow model better than the complete mix model even though the complete mix model was used to design the systems. This is not surprising if the flow patterns in the diffused air aeration systems is considered. Ponds using the Hinde Engineering Company aeration system (diffused air) are operated essentially as a plug flow, tapered aeration, activated sludge system without cellular recycle. Therefore, the plug flow model would be expected to more closely describe the performance of these systems. Surface aeration systems could be designed so that either flow model would describe the performance of a pond. For example, an aeration basin with a high length to width ratio with surface aerators would approximate a plug flow pattern, but a circular or square basin with surface aerators would be expected to approach complete mix conditions. In practice the flow patterns in ponds are imperfect and vary with each system. Logically, a model such as the Wehner-Wilhelm equation (Equation A-15) would be expected to provide the best estimate of the performance of pond systems. Unfortunately, none of the simple models is obviously superior to the others.

FIGURE A-15

RELATIONSHIP BETWEEN PLUG FLOW DECAY RATE AND TEMPERATURE - AERATED PONDS

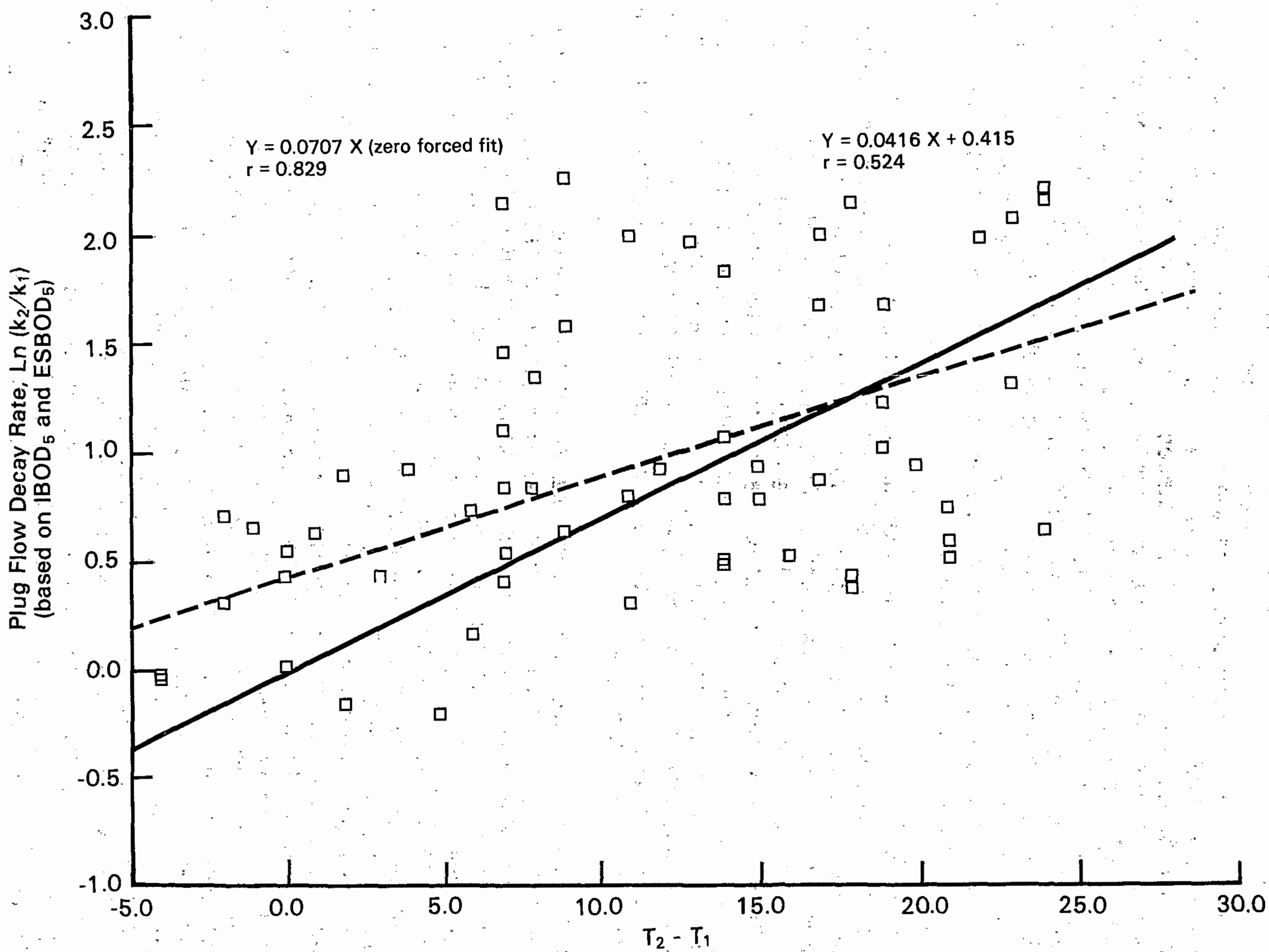


FIGURE A-16

RELATIONSHIP BETWEEN COMPLETE MIX DECAY RATE
AND TEMPERATURE - AERATED PONDS

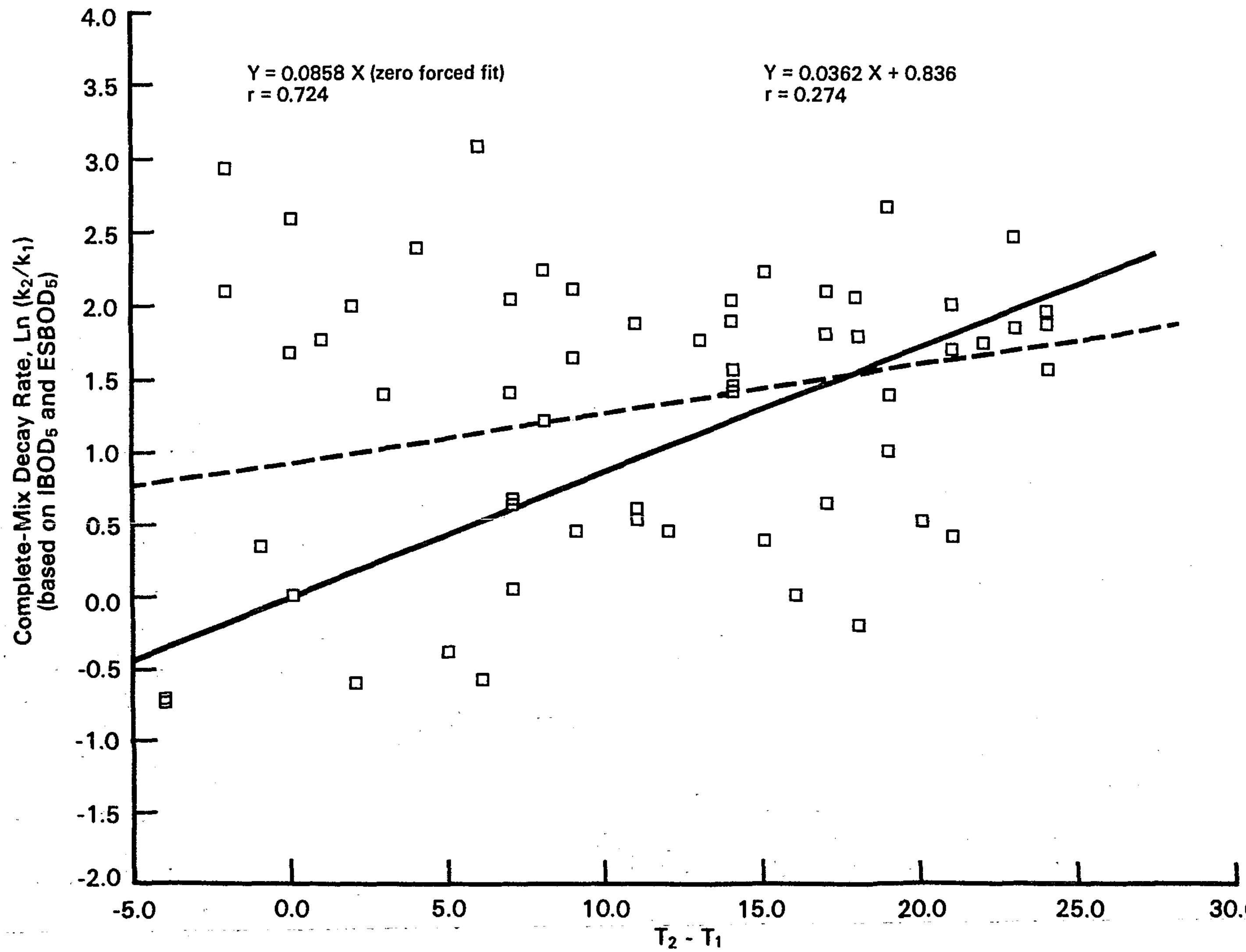


FIGURE A-17

RELATIONSHIP BETWEEN WEHNER AND WILHELM DECAY RATE AND TEMPERATURE - AERATED PONDS

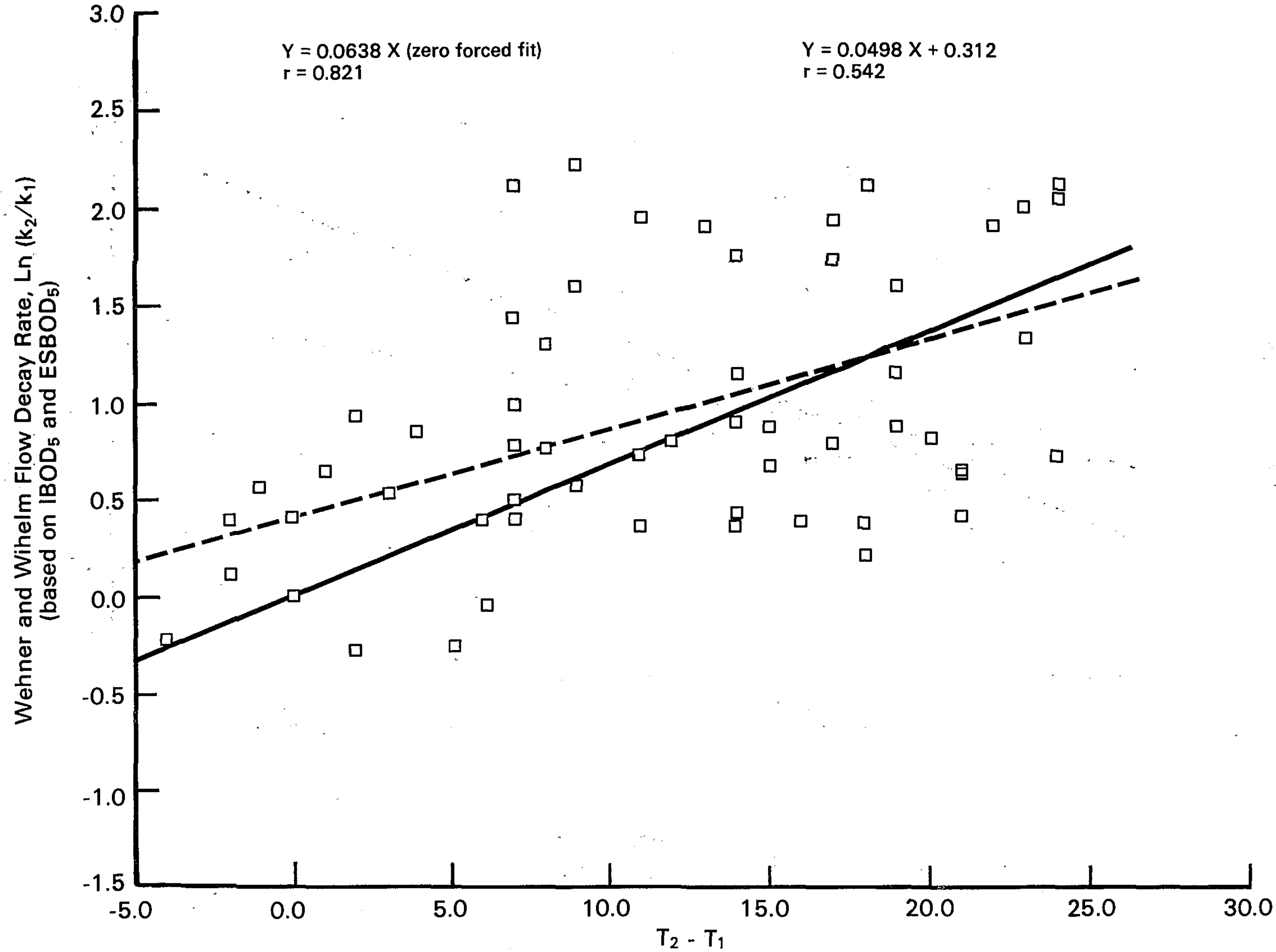


FIGURE A-18

PLUG FLOW MODEL - AERATED PONDS

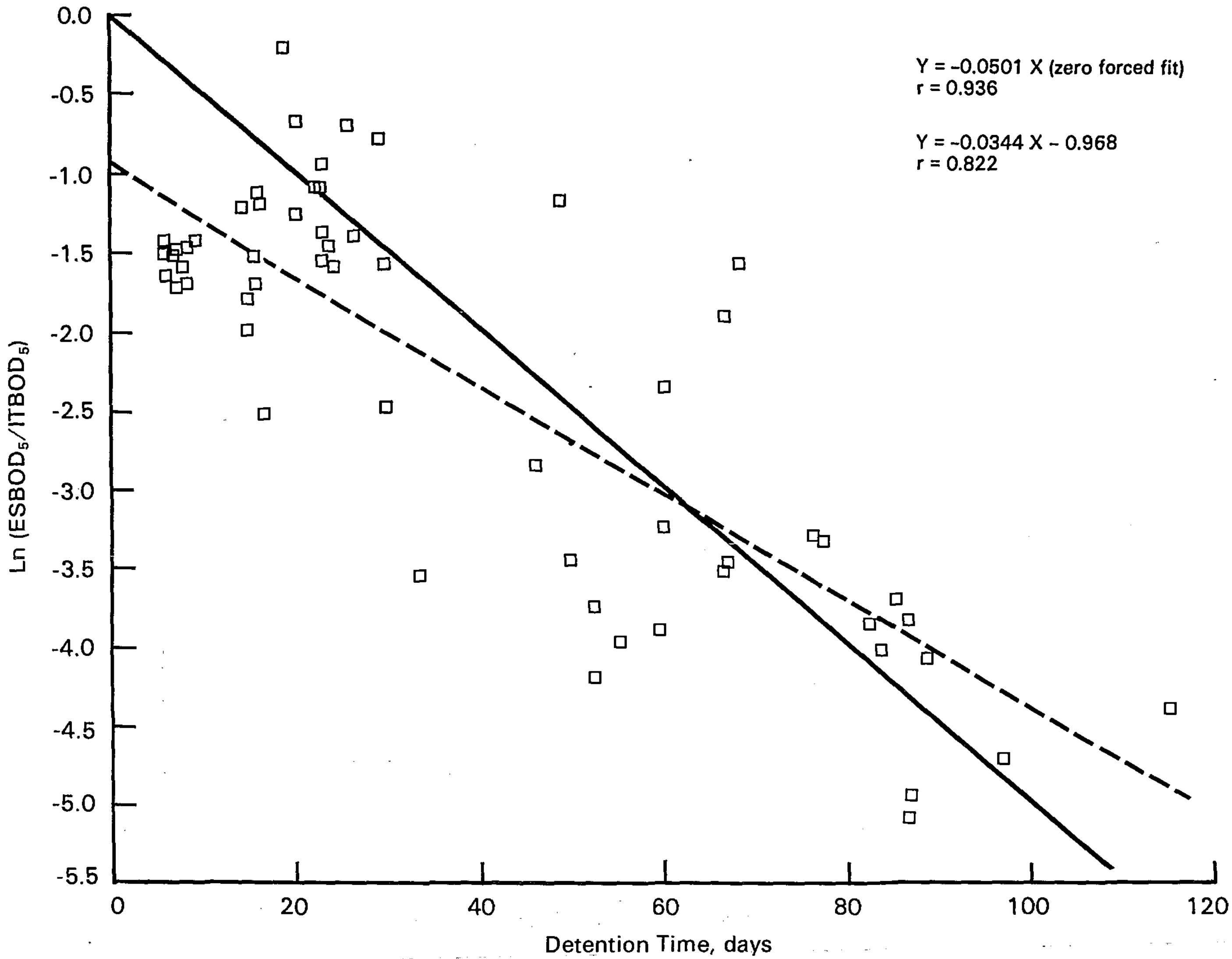
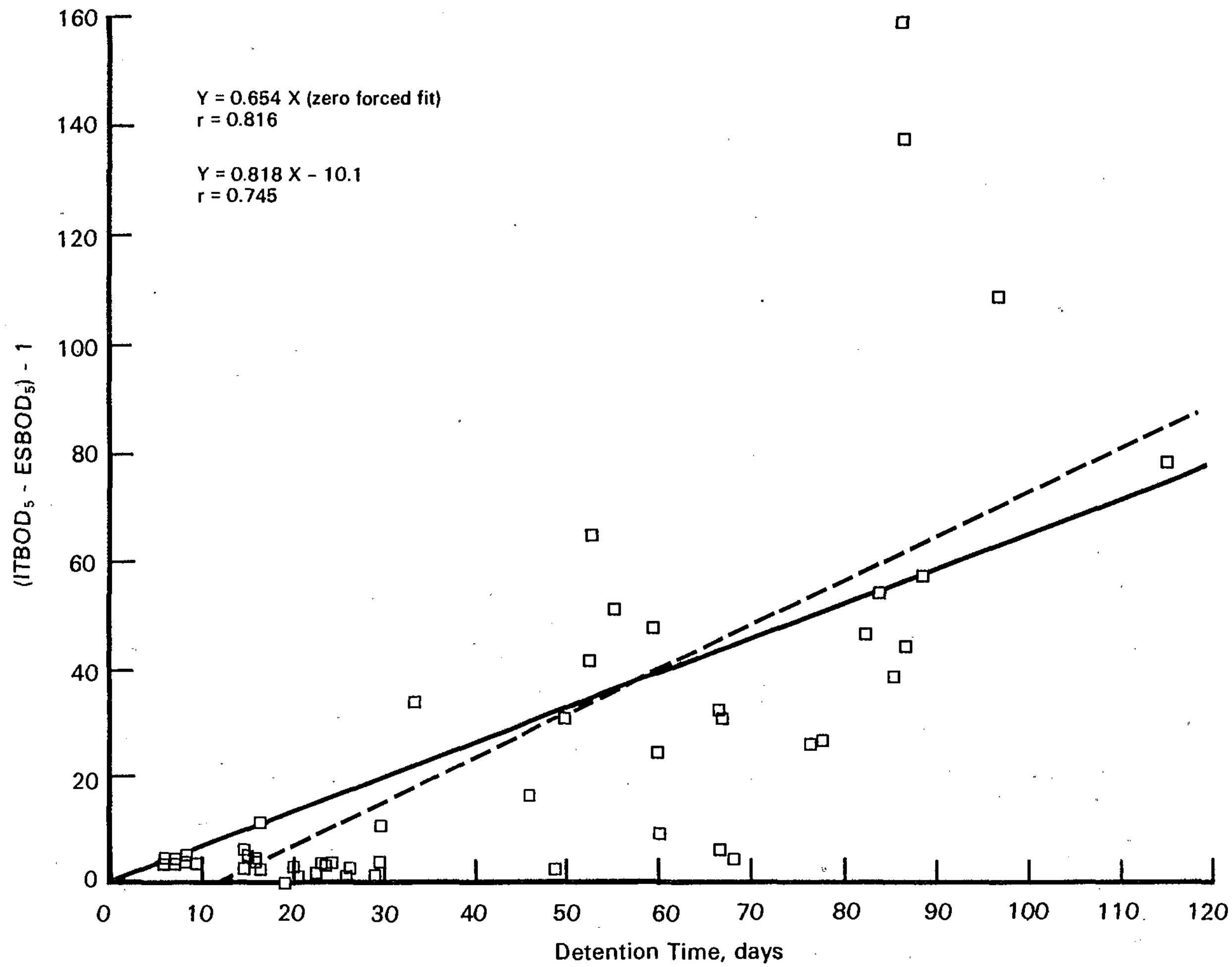


FIGURE A-19

COMPLETE MIX MODEL - AERATED PONDS



A.2.3 Oxygen Requirements

There is no rational design equations to predict the mass rate of transfer of DO and the required mixing to keep the solids suspended in an aerated pond. Using the mass of BOD_5 entering the system as a basis to calculate the oxygen requirements is simple and as effective as other approaches. To predict the aeration needed for mixing, it is necessary to rely on empirical methods developed by equipment manufacturers. Oxygen requirements do not control the design of aeration equipment in aerated ponds unless the detention time in the aeration tank is approximately one day. Such a short detention time is not recommended for aerated ponds; therefore, mixing is the major concern in the design.

Catalogs from equipment manufacturers must be consulted to ensure that adequate pumping, or mixing, is provided. The types of aeration equipment available are listed in Table A-3 and shown schematically in Figure A-20. Graphs or tables are available from all aeration equipment manufacturers, and all types of equipment must be evaluated to ensure that the most economical and efficient system is selected. The oxygen demand can be estimated using the procedure outlined in Table A-4. Suspension of solids (complete mix system) will require approximately 10 times as much power for mixing than for oxygen supply. Therefore, an economic analysis along with engineering judgment must be used to select the proper aeration equipment.

The system should be divided into a minimum of three basins and preferably four basins to improve the hydraulic characteristics and improve mixing conditions. The division of the basins can be accomplished by using separate basins or baffles. There are simple plastic baffles commercially available. Aerators should be selected and spaced through the basins to provide overlapping zones of mixing, and spaced in proportion to the expected oxygen demand (see Chapter 3). The oxygen demand will decrease in each succeeding cell.

TABLE A-3

TYPES OF AERATION EQUIPMENT FOR AERATED PONDS (10)

	O_2, lb (Standard Condition)	Oxygen Production lb O_2/hp	Power Requirements hp/10^6 gal	Common Depth ft	Advantages	Disadvantages
Floating Mechanical Aerator	1.8-4.5/hp			10-15	Good mixing and aeration capabilities; easily removed for maintenance	Ice problems during freezing weather; ragging problems without clogless impeller
High Speed		1.5	35			
Low Speed		2.5 to 3.5	25			
Rotor Aeration Unit (brush type)	3.5/hp			3-10	Probably unaffected by freezing; not affected by sludge deposits; good for oxidation ditches	Requires regular cleaning of air diffusion holes; energy conversion efficiency is lower
Plastic Tubing Diffuser	0.2-0.7/100 ft			3-10	Not affected by floating debris or ice; no ragging problem; uniform mixing & oxygen distribution	Requires regular cleaning of air diffusion holes; energy conversion efficiency is lower
Diffused Aeration		0.5 to 1.2	100			
Air-Gun	0.8-1.6/unit	12-20		12-20	Not affected by ice; good mixing	Calcium carbonate build-up blocks air holes; potential ragging problem affected by sludge deposits
Helical Diffuser	1.2-4.2/unit			8-15	Not affected by ice; relatively good mixing	Potential ragging problem; affected by sludge deposits

FIGURE A-20

SCHEMATIC VIEW OF VARIOUS TYPES OF AERATORS (10)

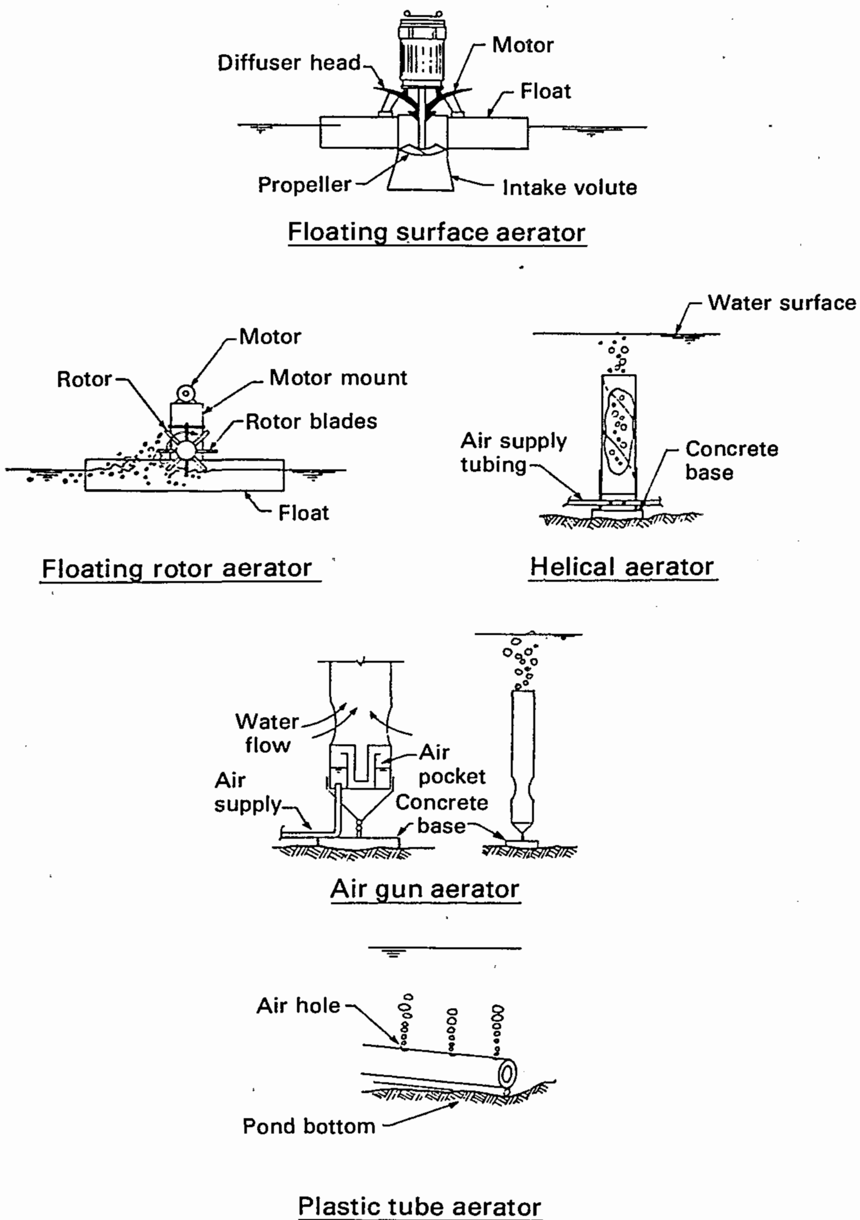


TABLE A-4

CALCULATION OF OXYGEN DEMAND AND SURFACE AERATOR SIZE FOR AERATED PONDS

Design Conditions

Design flow = 3,785 m^3/d

Influent BOD_5 = 300 mg/l

Pond temperature = 15°C

Barometric pressure = 760 mm of mercury

Oxygen Requirements

BOD_5 in wastewater = (300 mg/l) (3,785 m^3/d)(1,000 liters/m^3)/10^6

= 1,136 kg/d

Assume that O_2 demand of the solids and at peak flows will be 1.5 times the mean value of 1,136 kg/d

Oxygen requirements (N_a) = 1,136 x 1.5 = 1,703 kg/d

= 71 kg/hr

Aerator Sizing

After determining N_a, the equivalent oxygen transfer to tapwater (N) at standard conditions in kg/hr can be calculated using the following equation:

$$N = \frac{N_a}{\alpha \left[\frac{C_{SW} - C_L}{C_S}\right] (1.025)^{T-20}}$$

Where $\alpha = \dfrac{\text{oxygen transfer to waste}}{\text{oxygen transfer to tapwater}}$

C_{SW} = oxygen saturation value of the waste in mg/l calculated from the expression:

$$C_{SW} = \beta(C_{SS})\ P$$

Where $\beta = \dfrac{\text{oxygen saturation value of the waste}}{\text{oxygen saturation value of tapwater}}$

C_{SS} = oxygen saturation value of tapwater at the specified waste temperature

$$P = \frac{\text{barometric pressure at the plant site}}{\text{barometric pressure at sea level}}$$

C_L = DO concentration to be maintained in the waste (mg/l)

C_S = oxygen saturation value of tapwater at 20°C and 1 atmosphere pressure = 9.17 mg/l

T = pond water temperature (°C)

The design conditions are:

N_a = 71 kg O_2/hr

α = 0.9

β = 0.9

P = 1.0 atmosphere

C_L = 2.0 mg/l

T = 15°C

C_{SW} = 0.9 (10.15) 1.0 = 9.14 mg/l

$$N = \frac{71}{0.9\left[\frac{9.14 - 2.0}{9.17}\right] 0.884} = \frac{71}{0.9 \times 0.78 \times 0.884}$$

$= 114$ kg O_2/hr

Assume 1.4 kg O_2/hp-hr for the aerator for estimating purposes (taken from catalogs)

Total hp required is 114/1.4 = 81 brake horsepower (bhp) (60 kW)

Aerator drive units should produce at least 81 bhp (60 kW) at the shaft

Assume 90 percent efficiency for gear reducer

Therefore, total motor horsepower = 90 hp (67 (kW)

A.3 References

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