THE EFFECTS OF INCREASED SALINITY LEVELS ON THE REACTION RATES OF BIOLOGICAL WASTEWATER TREATMENT

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THE EFFECTS OF INCREASED SALINITY LEVELS ON THE REACTION RATES
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ABSTRACT

A series of eight 31 liter bench-scale aerobic waste stablization ponds were operated for 73 days to assess the potential performance of waste stabilization ponds in tropical Micronesia and to determine how elevated salinity levels affect pond kinetics. The outdoor reactors were loaded with screened raw domestic sewage and operated on a draw and fill basis. Salinity levels in the reactors varied from 600 to 36,000 mg/l with detention times ranging from 6.2 to 15.5 days.

The mean influent values for biochemical oxygen demand, BOD_5, and suspended solids, SS, were 86.7 and 61.9 mg/l, respectively. The mean effluent values for the last 20 days of the experiment ranged from 2.3 to 11.2 mg/l BOD_5 and from 3.8 to 44.3 mg/l SS. Reactor 1, which contained no supplemental salt, consistently had the highest treatment efficiency. All of the reactors easily met the EPA secondary treatment standard of less than 30 mg/l BOD_5 but all the saline reactors had SS concentrations in excess of 30 mg/l. Effluent SS concentrations were high in the saline reactors due to the presence of large populations of algae. Suspended solids concentrations were very low in the freshwater reactor presumably due to the presence of a large population of crustaceans, rotifers, insect larvae and other algae predators. These algae predators or effluent polishers never became well established in the saline reactors.

The results of this study show that waste stabilization ponds are an extremely viable waste treatment option in Micronesia. The ideal climatic conditions of the area minimize land requirements and make their use very cost effective.

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INTRODUCTION

The treatment and disposal of human wastes is one of the most critical problems facing any community. This is especially true in the small tropical islands of the U.S. Trust Territory of the Pacific Islands in Micronesia because of tremendous increases in the urban population due to migration from rural areas and high birth rates. The 1982-1983 epidemic of cholera in Truk and the endemic occurrence of gastro-intestinal disease throughout the islands (and other tropical underdeveloped areas of the world) provide ample evidence that present waste disposal facilities and practices are inadequate. The problem is due not only to a lack of economic resources but also to a lack of appropriate waste disposal technologies which address the distinctive environmental and cultural aspects of Micronesia.

Currently, collective wastewater treatment and disposal is practiced only in the district centers (equivalent to state or national capitals) of the Trust Territory while the majority of the population uses individual systems such as benjos (over water out-houses), pit privys and other traditional disposal practices. In the district centers, millions of dollars have been spent building technologically advanced mechanical treatment facilities. These systems have met with only limited success because their designers did not adequately consider local conditions.

Important factors which must be considered in the design of facilities in Micronesia include maintenance problems associated with corrosive tropical marine environments, a lack of trained supervisory and operating personnel, unreliable power supplies, difficulty in obtaining replacement parts and chemicals, and the limited financial resources of local governments.

One simplified wastewater treatment system which overcomes many of the problems mentioned above is treatment using stabilization ponds. These facilities generally have few if any mechanical components except for pumps to transport sewage to the site if gravity flow is impossible, hand operated gates to control pond depths and flow rates, tools for pond and grounds maintenance and some laboratory facilities for effluent monitoring and systems control. Along with lower operating and maintenance costs, the capital cost of stabilization pond construction is much lower than with mechanized systems except where land costs are prohibitively high.

In spite of the numerous advantages of stabilization ponds over mechanical systems only one pond system has been built in Micronesia. Although this system has received little maintenance over the years, it continues to perform satisfactorily and probably produces the best quality effluent in Micronesia.

Wastewater disposal in Micronesia is further complicated by severe freshwater shortages which occur in many areas during the dry season. During these drought periods, there is insufficient freshwater available for drinking, food preparation, and washing. This effectively precludes the use of freshwater as a wastewater carriage medium. In addition, water supplies are so short on many low islands and atolls that there is rarely excess freshwater available for waste disposal.

If collective waste disposal facilities are to be built in these areas, the only feasible waste carriage medium is seawater or brackish groundwater. One such seawater carriage system currently exists in Ebeye, Marshall Islands, where the only sources of freshwater are rainwater catchments and water barged in from nearby Kwajalein.

The purpose of this report is to investigate the potential of waste stabilization pond performance in tropical Micronesia and to evaluate the effects of elevated salinity levels on pond kinetics using laboratory scale ponds. This should enable future planners to better evaluate the merits of stabilization ponds in relation to more mechanized systems.

OBJECTIVES

The design and operation of waste stabilization ponds is generally based upon empirical methods which have been developed by observing the functioning of existing stabilization ponds. Typical design standards are based upon allowable areal loadings and detention times. For example, the "Ten States Standards" (APHA, 1973) for wastewater treatment ponds state that BOD_5 loadings may range from 17 to 45 kg/hr/day with a detention time of 90 to 120 days. Similar standards exist for other regions of the U.S. and the world but none are appropriate for the more favorable climatic conditions of Micronesia.

Empirical design equations have been developed by McGarry and Pescod (1970), Larsen (1974), Gloyna (1976), and others but none of these have been found to be very accurate or appropriate over a broad range of environmental conditions. The rational design methods of Marais (1970) and Thirumurthi (1969) and others also suffer from the same problems.

Without accurate design equations or historical pond performance data for the Micronesian area, it is very difficult to design cost effective stabilization ponds. The objectives of this study are therefore to investigate pond performance and kinetics required for rational pond design in Micronesia.

The specific objectives of this study are:

1. to investigate the effects of elevated salinity levels on waste stabilization pond performance under Micronesian climatic conditions;

2. to generate data which may be used to develop design standards for waste stabilization ponds in Micronesia; and

3. to evaluate the suitability of using waste stabilization ponds for wastewater treatment in Micronesia.

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LITERATURE REVIEW

Background

Since the passage of the Federal Water Pollution Control Act in 1972 (P.L. 92-500) the U.S. Environmental Protection Agency (EPA) has been charged with developing effluent limitation guidelines for industrial and municipal wastewater dischargers. The initial secondary treatment standards were published on August 17, 1973 (FR Vol. 38, No. 159, Part II, pp. 22298-22299). The regulations set the following standards: (a) five-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) of the effluents shall not exceed an arithmetic mean of 30 mg/l over a 30 consecutive day period nor 45 mg/l over a seven day period; (b) the arithmetic mean of the effluent BOD_5 and TSS shall not exceed 15 percent of the influent values over 30 consecutive days; and (c) the geometric mean of the fecal coliform bacteria in the effluent shall not exceed 200/100 ml for a 30 consecutive day period not 400/100 ml over a seven day period.

In subsequent years, waste stabilization ponds were found to easily meet the BOD_5 standard but they had difficulty meeting the fecal coliform and TSS standard. The fecal coliform problem was eliminated in 1976 when the EPA deleted the fecal coliform standard from its definition of secondary treatment.

In 1977, the EPA acknowledged the difference between the impacts of TSS from pond systems, which are predominantly live algae cells, and the TSS from conventional high-rate biological systems. The pond TSS were acknowledged to be substantially less damaging to receiving waters than the sewage solids and solids from other treatment processes. The EPA therefore dropped the uniform effluent TSS standard for publicly owned waste stabilization pond systems treating less than 7600 m^3/d (2 mgd). The current regulation allows the states and EPA regions to set TSS effluent limitations for individual facilities which reflect local conditions and needs (Gloyna and Fischler, 1981).

These changes in the regulations are particularly important in view of the advantages of wastewater treatment using ponds. The major capital costs of waste stabilization pond systems are associated with land acquisition and site preparation. Therefore, ponds are particularly attractive where land costs are reasonable and where climatic conditions are favorable. Pond systems, because of their inherent simplicity, also have much lower operation and maintenance costs than other treatment methods. In many systems, the only mechanical equipment required consists of pumps that may be necessary if gravity feed cannot be used or if recirculation of pond contents is desired.

The energy advantages of waste stabilization ponds are also very significant. Assuming a wastewater flow of 3800 m^3/d (1 mgd) of waste with an influent BOD_5 of 360 mg/l, a waste stabilization pond system would save over one million kwh/yr. of electricity as compared to an activated sludge system (Gloyna and Fischler, 1981).

Waste stabilization ponds, also called oxidation ponds and lagoons, are the most common type of sewage treatment facilities being installed in developing areas around the world. This is particularly true in the tropics where the climate is ideal for the process. The use of waste stabilization ponds is not, however, limited only to developing areas. They are now once again gaining increased acceptance and popularity in the United States and other industrialized countries as a reliable and low cost waste treatment alternative.

According to the USEPA, waste stabilization ponds are "Fully demonstrated and in moderate use especially for the treatment of relatively weak municipal wastewater in areas where real estate costs are not a restricting factor. The service life of lagoons is estimated to be 50 years. Little operator expertise is required. Overall, the systems are highly reliable," (EPA, 1980).

Process Description

Waste stabilization ponds are shallow bodies of water contained within earthen basins as shown in Figure 1. which use natural processes involving algae and bacteria to treat the wastewater. They are very popular in small communities because their low construction and operating costs give them a significant advantage over conventional mechanized treatment systems.

Stabilization ponds generally receive raw untreated wastes and usually consist of two or more ponds in series or parallel. The first pond (or ponds) which receives raw wastewater is called the primary cell. It is generally followed by secondary and tertiary cells.

The first step in pond treatment is the removal of settleable solids by sedimentation in the primary pond. Once settled, the solids or sludge undergoes anaerobic decomposition on the bottom of the pond as in an anaerobic digester. Stabilization of the settled waste is then brought about through the anaerobic conversion of organic wastes to CO_2, methane, other gases, organic acids and cell tissue. This is commonly referred to as primary treatment.

The next step in the treatment process is called secondary treatment. This step involves the biological oxidation of both soluble and suspended organic matter to stable end-products. This results in a reduction in BOD, TSS and coliform bacteria. The major reduction in pollutant concentrations is usually accomplished in the primary and secondary ponds (cells). Additional wastewater treatment may be provided by subsequent lightly loaded tertiary or polishing ponds which remove additional BOD and TSS.

There are four main types of stabilization pond systems: high-rate aerobic, aerobic, anaerobic and facultative. The type or types to be used in a particular situation depends upon the characteristics of the waste being treated and the degree of treatment required.

stabilization ponds to digest organic wastes. Because photosynthesis requires sunlight as an energy source, the process is active only during the daylight hours. At night, when light is no longer available for photosynthesis, algae use free oxygen for respiration and release carbon dioxide. This leads to a diurnal variation in the dissolved oxygen content and pH of stabilization ponds. Because of their need for sunlight, algae are generally found near the surface of ponds and in the aerobic zone. This zone is generally 15 to 50cm deep but it may extend down to a depth of 1.2m or more in well mixed ponds or lightly loaded ponds.

Algae require inorganic compounds for growth and reproduction. Their primary food source in stabilization ponds is the inorganic waste products of bacteria. Principal nutrients required by algae include carbon dioxide, and nitrogen, phosphorus. Required trace elements include metals such as iron, copper molybdenum, and others.

Many classes of algae are found in stabilization ponds. Five major groups are green (Chlorophyta), motile green (Volvocales and Euglenophyta), yellow green or golden brown (Chrysophyta), and blue-green (Cyanophyta). The most important forms in wastewater treatment appear to be green algae and blue-green algae. A predominance of green algae is generally indicative of a well functioning pond, with high pH and a nutritionally balanced waste. Blue-green algae appear to predominate when nutrient and pH levels are low, at higher temperature levels and when the green algae are devoured by animals such as Daphnia. The presence of large numbers of blue-green algae is therefore usually indicative of a poorly functioning pond.

The effects of elevated salinity on the algae populations of stabilization ponds has not been addressed previously in the literature. One would expect, however, that blue-green algae would predominate in saline ponds since they are normally the predominant algae group in nutrient enriched tropical shallow marine waters.

Protozoa are another important class of organisms in biological waste treatment. They are motile, heterotrophic, microscopic unicellular animals. Most favor aerobic environments but a few can survive under anaerobic conditions. They are important in waste stabilization ponds because they consume bacteria and organic particulate matter. Common members of this class include species of <u>Amoeba</u>, <u>Sporozoa</u>, <u>Paramecium</u>, <u>Suctoria</u> and <u>Mastigophoria</u>.

Higher animals which are important in ponds include rotifers, crustaceans, insert larvae and large organisms such as fish. Rotifers and crustaceans are multicelled aerobic heterotrophs. They feed on bacteria, protozoa, algae, and particulate organic matter. Their presence is an indication of a very efficient lagoon. They are usually not found in heavily loaded primary ponds but are common in well functioning secondary and tertiary ponds and are commonly referred to as effluent polishers. Little is known about their tolerance of high salinity levels.

Salinity Effects

Elevated and changing salinity levels in biological waste treatment have been studied in the past only for completely mixed aerated systems and trickling filters (Lawton and Eggert, 1957; Engineering Science, Inc., 1961; Ludzack and Noran, 1965; Hall and Smallwood, 1967; Kincannon and Gaudy, 1968). These studies have shown that the bacteria involved are capable of acclimating to the increased salinity levels and sewage treatment proceeds satisfactorily albeit at a slightly lower rate. The only major problem encountered with salinity was poor performance after the systems were shocked by abrupt changes in salinity. The biota of stabilization ponds should be much less susceptible to these shocks because of the moderating effects of the large storage volume in pond systems.

Design Procedures

Many empirical and rational design equations have been proposed for the design of waste stabilization ponds. Empirical design equations have been presented by McGarry and Pescod (1970), Larsen (1974), and Gloyna (1976). Their use for design purposes has been limited due to their inability to accurately predict the performance of existing waste stabilization ponds (Finney and Middlebrook, 1980). Presumably, this is due to climatic, kinetic and hydraulic differences which the design equations do not consider.

Rational design equations may be divided into two distinct categories, those based upon oxygen production and those based upon bacterial growth kinetics and reactor hydraulics.

Oswald and Gotaas (1957) presented one of the first rational design equations. It relates pond oxygen resources to applied loading and is therefore an oxygen production type equation. The method assumes that the decomposition of organic matter in a pond by bacteria is related to the rate at which oxygen is being produced or at which organic matter is being synthesized by algae, providing that sufficient light is available as an energy source. The design equation presented by Oswald and Gotaas is:

$$D = \frac{h\, L_t\, d}{F\, p\, T_c\, 1000\, S} \qquad (1)$$

where: D = detention time in days,

h = unit heat of combustion of algae, approximately 6 kg-cal/gram,

L_t = BOD at time t in days,

d = pond depth in cm.,

F = energy conversion factor,

P = ratio of weight of oxygen produced by algae to the weight of organic matter synthesized, typically 1.25 to 1.75,

T_c = temperature correction coefficient, and

S = insolation; gm. cal/cm^2-day.

The use of this equation results in relatively short detention times and high rates of algal production. The equation is best suited for the design of aerobic and high-rate aerobic pond systems. The method does not predict an effluent BOD but rather the detention time required for optimum algae synthesis. BOD reduction is expected to be in the range of 80 to 90 percent.

Most wastewater treatment process designs are based upon the hydraulic flow regime of the system and assumed first order biological reactions. The two general types of flow regimes in reactors are plug flow and completely mixed flow.

In plug flow, influent enters one end of the reactor and exits from the other with no lateral dispersion or mixing in between. A plug flow reactor with first order kinetics can be described using the following formula:

$$\frac{S_e}{S_o} = \exp(-kt) \qquad (2)$$

where: S_e = effluent concentration,

S_o = influent concentration,

k = first order reaction coefficient, and

t = reactor detention time.

Unfortunately, true plug flow rarely exists in waste stabilization ponds (unless there are many ponds connected in series) because of wind induced mixing, short circuiting, and flow turbulence.

At the other extreme of reactor flow models is the completely mixed flow regime. In this condition, the reactor contents are assumed to be completed mixed, with all areas of the reactor being of uniform composition. This model is appropriate only in cases where the reactor contents are maintained in suspension by intense mechanical mixing as in the activated sludge process.

The completely mixed flow regime with first order reaction kinetics can be described as:

$$\frac{S_e}{S_o} = 1 + kt \qquad (3)$$

where the terms are defined as before.

Marais (1970) proposed a design procedure based upon the completely mixed flow equation and the Arrhenius equation which is used for adjusting the reaction rate coefficient, k, to account for temperature changes. The Arrhenius equation is:

$$k_2 = k_1 \theta^{(T_2 - T_1)} \tag{4}$$

where: k_2 = reaction rate coefficient at T_2,

k_1 = reaction rate coefficient at a reference temperature, T_1,

θ = temperature coefficient (1.04-1.08),

T_1 = reference temperature, °C, and

T_2 = new temperature, °C.

In waste stabilization ponds, as in most other flow systems, neither plug flow nor completely mixed flow actually exists. Instead, there is a combination of both. In an effort to combine plug flow and completely mixed flow characteristics, Wehner and Wilheim (1958) presented an equation of the following form for non-ideal flow reactors:

$$\frac{Se}{So} = \frac{4a\exp(1/2d)}{(1+a)^2\exp(a/2d)-(1-a)^2\exp(-a/2d)} \tag{5}$$

where: $a = \sqrt{1 + 4ktd}$,

d = dispersion number = D/uL (dimensionless),

D = axial-dispersion coefficient, (L^2/T),

u = fluid velocity, (L/T), and

L = characteristic length, (L).

Thirumurthi (1969) used Equation 5 in his study of waste stabilization pond design and reduced the equation to the design formula chart shown in Figure 2 where kt is plotted against S_e/S_o for dispersion numbers ranging from zero to infinity. A dispersion number of zero represents the ideal plug flow case while a value of infinity represent the completely mixed flow condition. For mechanically mixed reactors such as activated sludge systems which are designed to operate as completely mixed systems, values of d range from 4 to infinity. For waste stabilization ponds, values of d will seldom exceed 1.0 due to low hydraulic loads (Nashashibi, 1967).

To use Equation 5 or Figure 2, one normally selects the BOD reduction required, S_e/S_o, and a dispersion factor, d. This determines kt from which the required detention time can be estimated. The factors d and k are best determined from bench or pilot plant scale model studies. The dispersion factor is best determined using the tracer study method of Levenspiel and Smith (1957). The reaction rate coefficient, k, should be determined from pilot plant or model studies for the particular location and wastewater being considered as it is a function of waste composition, local climatic factors (temperature, solar radiation, etc.) and the hydraulic characteristics of the proposed plant. Extreme care should be taken if published values of k are used because of the importance of the influencing factors mentioned previously and because the value of k varies with the equations or methods which were used to derive it. For example, k values derived using the plug flow equation are totally inappropriate for use with the completely mixed flow reactor equations.

Previous Work in Micronesia

In 1979, Barrett, Harris and Associates, Inc. of Guam was contracted by the Trust Territory of the Pacific Islands to provide an engineering evaluation of the wastewater treatment ponds on Kosrae and to develop basic criteria for the design of oxidation ponds throughout Micronesia. The report (Barrett, Harris, Inc., 1979) concluded: "The results of this study indicate that the oxidation pond/aerated lagoon type systems are well suited for application in remote island areas. This type of system is relatively inexpensive, easy to operate and very reliable. It is strongly recommended that this treatment method be considered for future use throughout the islands."

The Barrett-Harris study was superficial at best because of the limited amount of field data that was collected and because little data or rationale were presented to support their conclusions and proposed design criteria. The design criteria of 100 to 125 lbs BOD/acre/day was derived from Equations 4 and 5 using the following assumed parameters: a reaction rate coefficient of 0.2, a dispersion factor of 1.0, a pond depth of 5 ft., a temperature coefficient of 1.06, an influent BOD of 180 mg/l and a safety factor of 0.5 to 0.6. Unfortunately, little discussion was presented in the report to justify the use of these particular parameters in Micronesia, so it is impossible to justify the use of their design criteria for BOD loadings. A similar problem exists with the proposed detention time criteria of 30 days since no quantitative rationale was presented for its selection.

EXPERIMENTAL APPARATUS AND METHODS

Experimental Apparatus

A series of eight laboratory scale waste stabilization ponds was used to evaluate biological reaction rate coefficients and to develop waste stabilization pond design criteria appropriate for Micronesia.

The model ponds were constructed from rectangular glass tanks. The tanks had a volume of 31 liters with a freeboard of one centimeter and were sub-divided into four equal volume compartments as shown in Figure 3. The experiment was conducted outdoors to account for local climatic conditions. The tanks were placed in a large cage (for protection against vandalism)

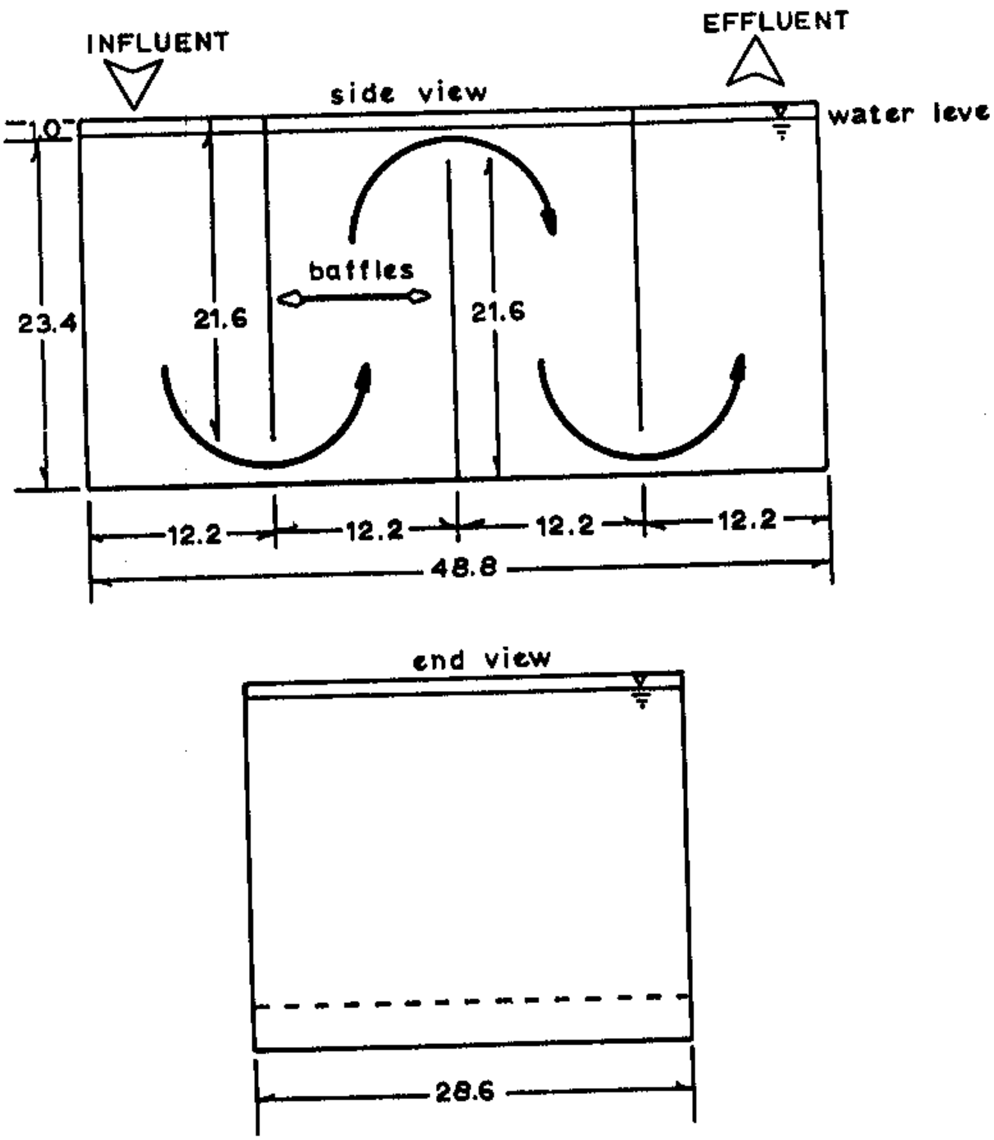

Figure 3. Experimental reactors.

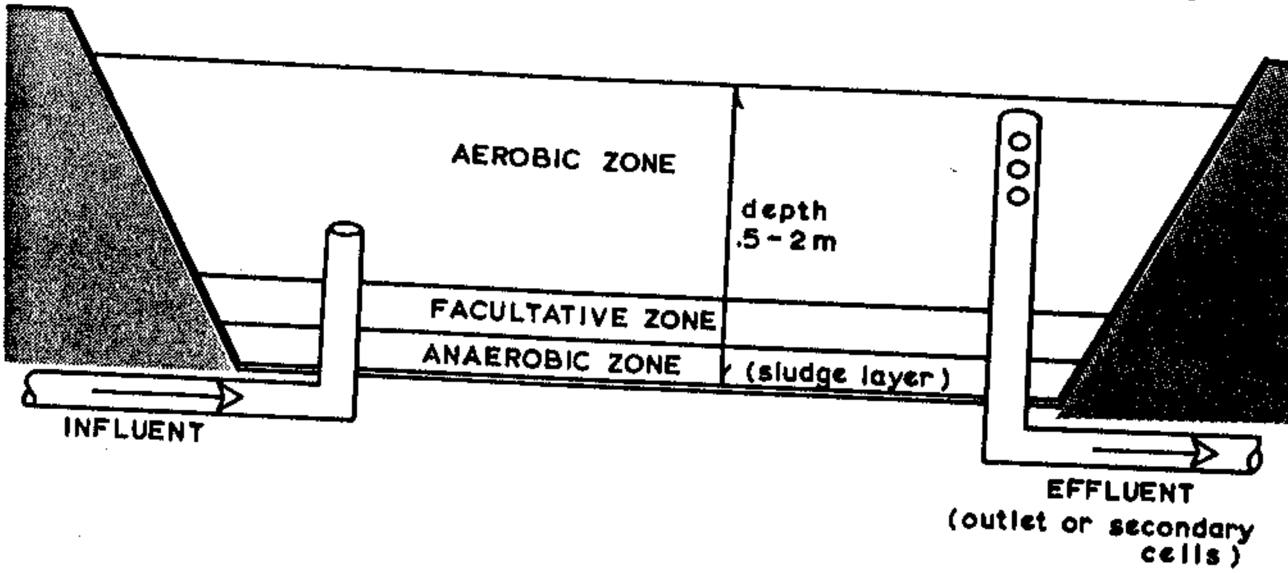

Figure 1. Aerobic or facultative waste stabilization pond design.

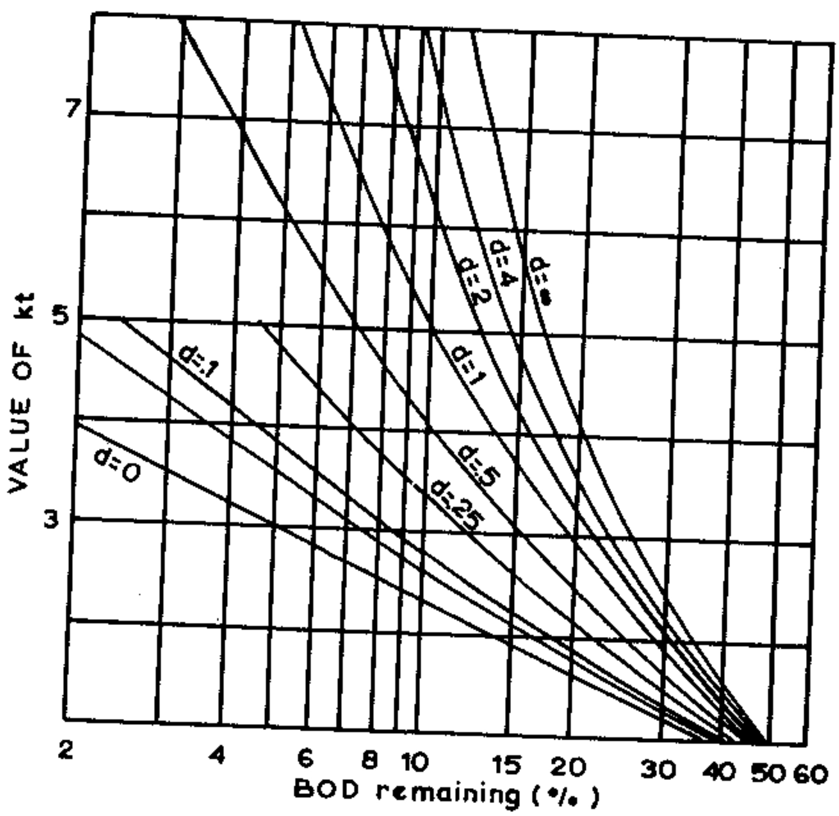

Figure 2. Design formula chart. Figure adapted from Thirumurthi, 1969.

High-rate aerobic ponds are usually used where a high algae mass is desired for harvesting or where a high concentration of algae in the effluent is not considered to be a problem. These ponds are shallow with depths ranging from .15 to .50m and may have detention times of one day or less.

Conventional aerobic ponds are designed to achieve maximum photosynthetic oxygen production. The oxygen is used by the bacteria which are responsible for BOD removal. Aerobic ponds are typically 1 to 1.5m in depth with detention times of 2 to 40 days.

Anaerobic ponds are designed to treat strong wastes (high oxygen demand). Organic loadings are so high that the ponds are typically devoid of oxygen throughout their entire depth. They are typically 2.5 to 5m in depth with detention times of 20 to 50 days.

Facultative ponds (see Figure 1) are the most common type of stabilization ponds. They have two treatment zones: an aerobic surface layer and an anaerobic lower layer. They are typically 1 to 2.5m in depth and have detention times of 5 to 30 days.

Waste Stabilization Pond Biology

Waste treatment in stabilization ponds is a natural process involving the activities of many different types of co-existing organisms. The four main types of organisms are bacteria, algae, protozoa and higher animals.

Bacteria are single celled protists. They break down complex organic matter into soluble matter which passes through their cell walls and is converted to energy, cell tissue and waste products. Bacteria can be classified according to how they obtain and utilize oxygen. There are three main types of bacteria in the oxygen utilization classification system: aerobic, anaerobic and facultative. Aerobic bacteria live only in the presence of free oxygen. Their waste products typically consist of carbon dioxide, ammonia and phosphates. Anaerobic bacteria, which exist only in the absence of oxygen, produce carbon dioxide, hydrogen sulfide, ammonia, organic acids and other soluble products. Facultative bacteria can live with or without oxygen. The composition of their waste products is dictated by the oxygen condition under which they are living.

In general, aerobic bacteria reduce organic matter to completely oxidized end products which are consumed by algae. Anaerobic bacteria produce partially oxidized waste products which are used as a food source by both algae and aerobic bacteria.

Algae are unicellular or multicellular autotrophic, photosynthetic protists. Most algae are considered to be members of the plant kingdom but the blue-green algae, being protists, are more related to bacteria. Algae produce their own carbon energy stores from atmospheric carbon dioxide through the process of photosynthesis. Free oxygen is a by-product of this process. This oxygen is available for use by aerobic bacteria in situated in an open field. The cage was constructed of hog wire so there was minimal shading of the reactors.

Because Guam is located in a tropical area with very heavy rainfall rates, the reactors were protected from excessive rainfall and overflow by installing a movable sheet of plexiglass over the reactors. The plexiglass was installed about 1 meter above the tanks to minimize interference with air flow. During storms, the plexiglass was lowered for increased protection and it was placed directly on the tanks during tropical storms and typhoons. On sunny days, the plexiglass was removed to allow maximum solar radiation flux to the tanks.

Experimental Procedures

The reactors were operated as shown in Table 1. The indicated amounts of raw domestic sewage were added to the tanks each morning. The sewage was collected from the Dededo Pump Station in northern Guam every 5 to 7 days and was stored in a refrigerator at 5°C. The sewage was screened during the collection process with conventional mesh window screen to remove trash and other large solid matter.

The reactors were started initially by filling them with sewage which had been inoculated with bacteria and algae from an aerated lagoon sewage treatment plant which was then being operated as a facultative waste stabilization pond due to low loadings. The salinity levels in reactors 2 to 8 were then adjusted to the levels indicated in Table 1 by adding a synthetic saltwater aquarium mix (Aqua-Marin, Aquatrol, Inc., Anaheim, CA). The reactors were then allowed to acclimate for one week to allow the bacteria and algae to adjust to the new salinity levels. Tap water was added as required to make up for evaporation.

After the initial acclimation period, the reactors were operated on a draw and fill basis for 73 days. Each morning, a sample of the volume indicated in Table 1 was withdrawn from the effluent end of each reactor using a siphon hose to minimize turbulent mixing. The siphon inlet was maintained at a depth of 2 to 3 cm. The effluent sample was saved for chemical analysis if required. The required amount of salinity adjusted wastewater was then added to the influent end of each reactor by pouring it slowly onto a removable baffle plate. Tap water was added two to three times per week to make up for evaporation which was as high as 6 mm/day.

Dispersion Experiment

A tracer study was also conducted on the reactors to estimate the dispersion number, d, and the mean detention time, t, required by Equation 5. Three of the same reactors used as model waste stabilization ponds were operated on a draw and fill basis with the same hydraulic loadings as the model ponds (2, 3 and 5 liters/day).

On the first day, the reactors were filled with deionized water. A sodium chloride solution (150 gm per 2, 3 and 5 liter for reactors 1, 2 and 3, respectively) was then added to the influent end each reactor. On each subsequent day, the required amounts of deionized water (2, 3 and 5 liter)

Table 1. Sewage loading, total dissolved solids and detention times for experimental reactors.

Reactor	Hydraulic Loading, 1/day	Detention Time, days	Total Dissolved Solids, mg/1
1	2	15.5	600
2	2	15.5	12000
3	2	15.5	22000
4	2	15.5	36000
5	3	10.3	12000
6	3	10.3	22000
7	5	6.2	12000
8	5	6.2	22000

Table 2. Physical and chemical water quality parameters measured during the study of reactor effluent and influent.

Parameters		Methods
1.	pH	Glass electrode
2.	Turbidity	Nephelometer, Hach Model 2100A
3.	Total dissolved solids	Glass fiber filter filtration and filtrate evaporation then drying at 180°C
4.	Nonvolatile dissolved solids	Above followed by incineration at 500°c
5.	Suspended solids	Glass fiber filter filtration and drying at 180°C
6.	Volatile suspended solids	Above followed by incineration at 550°C
7.	Specific conductance	Wheatstone bridge
8.	BOD_5	5-day incubation at 20°C

were then added to the reactors each day and the effluents were analyzed for specific conductivity. The data obtained by conductivity monitoring were then used to determine d and t.

Levenspiel and Smith (1957) found that the mean detention time, t, and dispersion number, d, could be determined using the following equations:

$$t = \frac{\Sigma TC}{\Sigma C} \qquad (6)$$

$$\sigma_t^2 = \frac{\Sigma T^2 C}{\Sigma C} - \left(\frac{\Sigma TC}{\Sigma C}\right)^2 \qquad (7)$$

$$\sigma^2 = \frac{\sigma_t^2}{t^2} = 2d - 2\,d^2(1-\exp(-1/d)) \qquad (8)$$

where C is the concentration of the tracer (sodium chloride) on day T. The mean detention time can be determined directly from Equation 6 while the dispersion number, d, can be calculated by trial and error from Equation 8.

Chemical and Biological Analyses

The raw sewage and reactor effluents were analyzed for the parameters listed in Table 2. All parameters were analyzed in accordance with the procedures given in "Standard Methods" (APHA, 1981).

Initially, it had been planned to evaluate treatment efficiency and reactor kinetics using the chemical oxygen demand (COD) test, however, no satisfactory COD analysis technique was found which was accurate for low COD values at high salinity levels. Several different methods specifically developed for determining COD in saline waters (Bums and Marshall, 1965 and EPA, 1979) were tested using standard COD solutions over a range of salinity levels; however, all the methods produced inconsistent and inaccurate results presumably due to chloride ion interference. Attempts were made to reduce chloride ion interference by diluting the samples but this resulted in very low COD values which produced even more inconsistent results. In view of these problems, it was decided to evaluate treatment efficiencies using the 5-day biochemical oxygen demand (BOD_5) test which is much less sensitive to chloride levels.

Both unfiltered and filtered BOD_5 tests were performed. Filtering was conducted using Whatman grade 934 AH glass fiber filters that were also used in the suspended and volatile suspended solids analyses. Filtered BOD_5 tests were conducted to evaluate the efficiency of the pond in reducing initial soluble and insoluble organic matter to soluble organic matter which had not been converted to cell tissue. Combined with the unfiltered BOD_5 data, one can then differentiate between that BOD_5 due to soluble organic matter, which is the primary source of oxygen demand in receiving waters, and that due to suspended solids which are principally algae in waste stabilization ponds effluents. Living algae are not very detrimental to receiving water because they do not exert an immediate oxygen demand since they are living and produce oxygen by photosynthesis until they die. The difference in the filtered and unfiltered BOD_5 data is also useful because it allows an estimate to be made of potential treatment efficiency if algae harvesting or removal is later practiced.

Initially, the reactors were also to be analyzed for nutrient levels and samples were collected, frozen and stored after collection for subsequent analysis. Unfortunately, the samples were lost when a freezer failed during a holiday period.

At the end of the experiment, the reactors were also sampled to evaluate qualitative differences in the algae and predator communities. It was expected that population differences would exist due to the different salinity levels and organic loadings. Both scrapings from the side walls of the reactors and suspended algae were collected from each reactor and examined under the microscope to identify the broad algae groupings present. Since diatoms were common in some reactors and expertise in their identification was readily available, permanent slide mounts were made of diatoms in order to identify the major species present.

CLIMATIC CONDITIONS IN MICRONESIA

The climatic factors of temperature and solar radiation are extremely important in waste stabilization pond design. Researchers have determined that pond reaction rate coefficients are directly proportional to pond temperature and available solar radiation (Oswald, et.al., 1958). Waste stabilization ponds are therefore expected to perform well in Micronesia due to the high ambient temperatures and abundant sunshine.

The climate in Micronesia is warm and humid throughout the year. Daytime temperatures are generally 29 to 32°C while nighttime temperatures

Table 3. Average monthly precipitation (cm) for selected Micronesian islands.

Location	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Annual
Guam	13.1	10.8	7.5	10.2	11.4	13.2	24.3	30.9	35.8	36.6	21.6	14.9	230.3
Palau	25.1	18.9	19.8	22.1	36.9	34.9	41.1	39.1	32.0	32.0	27.1	32.7	365.0
Yap	21.4	13.7	14.2	15.2	25.5	27.7	37.2	38.5	33.3	31.5	25.5	25.1	308.7
Truk	21.8	16.3	20.3	33.5	40.0	32.0	39.1	34.1	33.2	34.0	30.5	35.4	370.1
Ponape	29.9	29.1	38.2	49.7	49.3	39.8	45.3	42.4	42.6	41.7	43.6	40.5	492.1
Kosrae	38.8	45.8	49.4	58.8	49.8	47.1	41.5	38.3	36.9	31.8	40.1	49.3	527.7
Kwajalein	10.2	6.4	15.3	16.5	25.9	25.7	25.6	25.9	28.8	30.7	27.9	24.0	263.0

Table 4. Average monthly air temperature (°c) for selected Micronesian islands.

Guam	25.2	25.2	25.4	26.1	26.3	26.5	26.3	26.2	26.2	26.2	25.9	25.4	25.9
Palau	27.3	26.4	27.3	27.8	27.8	27.5	27.3	27.3	27.5	27.7	27.7	27.4	27.5
Yap	26.9	26.9	27.2	27.6	27.7	27.6	27.3	27.2	27.4	27.4	27.5	27.2	27.3
Truk	27.2	27.3	27.4	27.4	27.4	27.4	27.1	27.2	27.2	27.3	27.4	27.4	27.3
Ponape	27.2	27.2	27.7	27.2	27.2	27.0	26.8	26.8	26.8	26.9	27.0	27.2	27.1
Kosrae	27.5	27.4	27.3	27.2	27.3	27.2	27.2	27.4	27.4	27.6	27.4	27.5	27.3
Kwajalein	27.3	27.4	27.6	27.7	27.7	27.7	27.8	28.1	28.1	28.0	27.6	27.5	27.7

* Based on 1941-1970 period of record, NOAA

**1941-1981 period of record

Table 5. Guam weather conditions during the study.

	Air Temperature Max. (°C)	Air Temperature Min. (°C)	Air Temperature Avg. (°C)	Precipitation (cm)	Wind Avg. Speed (km/h)	Wind Avg. Direction	Global Solar Radiation[b] (g-cal/cm^2-d)	Evaporation (cm)	Sunshine (% of possible)	Sky Cover (tenths)
Aug. 1982	30	22	26	16.1	12.6	E	410	15.9	29	8.7
Sept. 1982	30	22	26	68.9	13.6	W	410	16.7	20	8.6
Oct. 1982	29	21	25	30.1	13.3	SE	400	13.4	21	8.5
Nov. 1982	29	23	26	23.7	19.3	ENE	400	15.4	25	7.5
Annual	29.5	22.4	25.9	230.3	11.9	E	430	205.1[c]	53	8.0

a 1941-1970 period of record

b 1979 data

c 1978 data

range from 21 to 25°C. Relative humidities range from 80 to 95 percent during the night and from 75 to 85 during the day.

Almost all the islands of Micronesia have seasons characterized as wet and dry. The dry seasons generally extend from January to April while the wet seasons run from July to November. Secondary or transition seasons exist between the primary seasons. These seasons are not absolute and vary from one island group to the next. While some areas such as Guam, Palau, the Northern Marianas, Yap, the Marshalls and Truk have distinct seasons, others such as Ponape and Kosrae are fairly wet throughout the year. Precipitation varies greatly throughout Micronesia. Average annual rainfall varies from a low of 220 cm in Guam to a high of 550 cm in Kosrae.

The trade winds are generally the dominant winds throughout Micronesia. They generally blow from the east or northeast and are the strongest and most constant during the dry season with speeds of 20 to 40 km/h. During other portions of the year winds average 10 to 15 km/h.

Cloud cover is generally high throughout Micronesia and ranges from 70 to 90 percent on a monthly basis. Percent of possible sunshine as measured by the National Weather Service varies from roughly 40 to 70 percent. Solar radiation data are very rare in Micronesia but limited records in Guam indicate that average monthly values of 350 to 550 g cal/cm^2/day (global radiation) are expected with an average annual value of approximately 430 g cal/cm^2/day (1979 data). Pan evaporation data are also scarce but generally average about 5 mm/day.

Tables 3 and 4 are a summary of climatic conditions in the various islands of Micronesia. Table 5 presents a more detailed summary of the weather conditions observed on Guam during the experimental study.

RESULTS AND DISCUSSION

The primary objectives of this research were to evaluate the potential performance of waste stabilization ponds in Micronesia and to investigate the effects of elevated salinity levels on kinetics. Performance was investigated using a series of laboratory-scale waste stabilization ponds which were operated for a period of 73 days.

A detailed listing of the influent and effluent water quality for the experimental reactors is presented in Tables A-1 to A-9 of Appendix A. As shown in the tables, both influent and effluent water quality varied greatly throughout the study. A primary factor influencing effluent variability was the changing strength of the influent. This variability was unavoidable since the strength of domestic sewage is naturally highly transient. An influent of uniform composition could have been obtained by using a synthetic sewage, but the derived reaction rates would then have been inappropriate for domestic sewage. Other causes of effluent variability were changing biological communities during the acclimation period and possibly, changing dissolved oxygen (DO) levels due to weather induced variations in photosynthetic activity. Observed DO levels were never found to drop below 4 mg/l after large algae populations developed so

Table 6. Biochemical oxygen demand (5-day, mg/1) of unfiltered reactor effluent a
sewage influent. (Total dissolved solids of reactors indicated in parenthesis).

		REACTOR							
Date Day	1 (600)	2 (12000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	Influe (700
26-11 53	2.7	2.5	9.5	10.7	9.4	9.5	4.6	5.3	41.
1-12 58	2.3	5.0	8.5	4.3	8.9	9.9	10.8	7.8	112.
6-12 63	1.9	8.5	6.3	4.0	8.1	3.9	9.9	9.4	88.
11-12 68	2.0	7.3	5.3	5.7	12.7	9.4	11.1	11.4	122.
16-12 73	2.8	7.9	4.7	4.8	16.7	5.7	9.8	6.4	69.
MEAN	2.3	6.2	6.9	5.9	11.2	7.7	9.2	8.1	86.
MAXIMUM	2.8	8.5	9.5	10.7	16.7	9.9	11.1	11.4	122.
MINIMUM	1.9	2.5	4.7	4.0	8.1	3.9	4.6	5.3	41.
STD. DEV.	0.4	2.2	1.8	2.5	3.2	2.4	2.4	2.2	29.

Table 7. Biochemical oxygen demand (5-day, mg/l) of unfiltered reactor effluent and sewage influent. (Total dissolved solids of reactors indicated in parenthesis).

	REACTOR								
Date Day	1 (600)	2 (12000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	Influent (700)
26-11 53	4.2	12.1	6.1	3.6	4.7	2.9	3.3	3.0	-
1-12 58	1.6	1.6	1.7	0.9	0.6	1.6	1.1	1.3	47.1
6-12 63	1.1	0.5	0.6	0.6	2.0	0.5	0.5	0.7	31.3
11-12 68	0.8	0.7	0.8	2.1	1.0	0.9	2.6	1.6	43.2
16-12 73	1.1	1.3	0.9	1.4	3.1	1.5	2.7	1.4	22.2
MEAN	1.8	3.2	2.0	1.7	2.3	1.5	2.0	1.6	35.9
MAXIMUM	4.2	12.1	6.1	3.6	4.7	2.9	3.3	3.0	47.1
MINIMUM	0.8	0.5	0.6	0.6	0.6	0.5	0.5	0.7	22.2
STD. DEV.	1.2	4.4	2.1	1.1	1.5	0.8	1.1	0.8	9.8

effluent variability was probably not affected significantly by a lack of oxygen for aerobic biological activity.

Because of the effluent variability just discussed, only the last 20 days of water quality data (days 53 to 73) were used for reactor analysis and reaction rate coefficient determinations. Prior to day 53, the biota of the reactors were assumed to be acclimating to the sewage and to the elevated salinity levels.

Effluent Water Quality

Tables 6 to 9 are a summary of the more important water quality data from the last 20 days of the study. As shown in Table 9, all of the reactors easily met the 30 mg/l BOD effluent limitation but most had difficulty meeting the old SS limitation. BOD removal efficiencies averaged 92% and ranged from a low of 87 (Reactor 5) to a high of 97% (Reactor 1). SS removal rates averaged 50% with a minimum of 29% (Reactor 7) and a maximum of 94% (Reactor 1). The observed mean pH values ranged from 8.9 to 9.2. Values in this range are indicative of a well functioning waste stabilization pond (Neel et al., 1961). Effluent turbidity values ranged from 1.2 to 12.7 NTU.

Biological Observations

The biological communities of the reactors developed along two main lines. The freshwater reactor (TDS 600 mg/l) developed a complex trophic level structure consisting of a moderate standing crop of green algae which served as a primary food source for subsequent animal trophic levels consisting of protozoans, rotifers, Daphnia and insect larva. The remaining reactors developed heavy concentrations of algae but had fewer higher organisms.

The second major biological development of the reactors concerns the shift in dominant algal group and species as the TDS level increases. In the freshwater reactor, Chlorococcum and Palmella genera were the predominant green algae observed. No blue-green algae were observed. In the 12,000 and 22,000 mg/l TDS reactors (3 of each) blue-green algae appear and the green algae are less abundant though they remain dominant over blue-green algae. The primary blue-green algae appearing in these reactors were Anacystis and Nostoc. In the 36,000 mg/l TDS reactor nearly all green algae were replaced by blue-greens. It was also noted (not quantified) that this reactor had less algal production than the 12,000 and 22,000 TDS reactors. Animal life was not observed in the highest saline reactor. It was also observed that blue-greens slightly increased in preponderance over green algae as the detention time decreased.

Few diatom genera were observed in the reactors which corresponds to past findings that diatoms species as a group are less adaptable to enriched or polluted environments than green or blue-green algae

Table 8. Suspended solids (mg/l) for reactor effluent and sewage influent. (Total dissolved solids of reactors indicated in parenthesis).

		REACTOR								
Date	Day	1 (600)	2 (12000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	Influent (700)
27-11	53	5.9	37.6	28.4	14.7	40.0	36.8	0.0	22.3	0.0
1-12	58	3.0	56.0	15.7	53.1	32.8	50.8	48.0	43.6	66.0
6-12	63	0.2	47.6	43.2	13.7	57.2	43.2	53.2	36.8	57.3
11-12	68	8.9	33.2	25.2	17.8	36.4	35.2	40.0	44.4	30.8
16-12	73	1.2	32.6	10.1	12.1	34.6	20.0	35.8	20.5	93.7
MEAN		3.8	41.4	24.5	22.3	40.2	37.2	44.2	33.5	61.9
MAXIMUM		8.9	56.0	43.2	53.1	57.2	50.8	53.2	44.4	93.7
MINIMUM		0.2	32.6	10.1	12.1	32.8	20.0	35.8	20.5	30.8
STD. DEV.		3.2	9.1	11.4	15.5	8.8	10.2	6.8	10.3	22.5

Table 9. Reactor performance summary[1]

Parameter	Reactor 1	2	3	4	5	6	7	8	Influe
TDS, mg/l	600	12000	22000	36000	12000	22000	12000	22000	700
Hydraulic load, l/day	2	2	2	2	3	3	5	5	---
Waste load, kg/BOD/ha/day	12.5	12.5	12.5	12.5	18.8	18.8	31.4	31.4	---
BOD, mg/l (% removal)	2.3 (97)	6.2 (93)	6.9 (92)	5.9 (93)	11.2 (87)	7.7 (91)	9.2 (89)	8.1 (91)	86.7 ---
SS, mg/l (% removal)	3.8 (94)	41.4 (33)	24.5 (60)	22.3 (64)	40.2 (35)	37.2 (40)	44.2 (29)	33.5 (46)	61.9 ---
pH	8.9	9.2	9.0	8.9	9.1	9.0	9.1	9.1	7.4
Turbidity, NTU	1.2	8.8	7.2	7.2	9.3	10.4	12.7	8.8	59.0
t, theoretical[2] (days)	15.5	15.5	15.5	15.5	10.3	10.3	6.2	6.2	---
t, dispersion tests[3], (days)	16.5	16.5	16.5	16.5	12.0	12.0	7.5	7.5	---
d, dispersion number	.14	.14	.14	.14	.16	.16	.20	.20	---
k[4] @ ambient °C	.33	.22	.21	.22	.22	.27	.42	.45	---
k @ 20°C (θ= 1.072 in Eq.4)	.20	.13	.13	.13	.13	.16	.25	.27	---

[1] Based on data from day 53 to 73 or from the dispersion experiment

[2] Calculated detention time from reactor volume/hydraulic load

[3] Estimated from dispersion test

[4] k is the reaction rate coefficient l/day at ambient temperature (27.2 ± 3.9 °C) a determined by Equation 5.

Table 10. Aerobic waste stabilization pond design example.

Reaction Rate Coefficient, R(@27°C)	Required Detention Time, Days (Required Pond Surface Area, Hectares) Dispersion Number 0.10	0.15	0.20	0.25	0.30
.15	14.8 (4.2)	15.7 (4.5)	16.6 (4.7)	17.4 (4.9)	18.1 (5.1)
.20	11.1 (3.2)	11.8 (3.4)	12.4 (3.5)	13.0 (3.7)	13.5 (3.9)
.25	8.9 (2.5)	9.4 (2.7)	10.0 (2.8)	10.4 (3.0)	10.8 (3.1)
.30	7.4 (2.1)	7.9 (2.2)	8.3 (2.4)	8.7 (2.5)	9.0 (2.6)
.35	6.3 (1.8)	6.7 (1.9)	7.1 (2.0)	7.4 (2.1)	7.7 (2.2)
.40	5.6 (1.6)	5.9 (1.7)	6.2 (1.8)	6.5 (1.9)	6.8 (1.9)
.45	4.9 (1.4)	5.2 (1.5)	5.5 (1.6)	5.8 (1.7)	6.0 (1.7)
.50	4.4 (1.3)	4.7 (1.3)	5.0 (1.4)	5.2 (1.5)	5.4 (1.5)

Design Assumptions:

1) Design population = 10,000

2) Hydraulic Load = 285 l /capita-day (75 gpcd)

   = 2850 m^3/day

3) Influent BOD_5 = 150 mg/l

4) Treatment efficiency = 85% BOD_5 reduction

5) Effluent BOD_5 = 22.5 mg/l

6) Pond depth = 1 m

(Note: 1 Hectare = 2.47 acres)

(Fjeringstad, 1964). However, there were definite diatom communities in each reactor corresponding to the TDS concentration. Reactor 1 (freshwater) possessed at least three species of diatoms dominated clearly by Gomphonema clevei Fricke. This diatom is a common diatom (occasional dominant) in pools in small rivers in southern Guam (Zolan, 1981). The fact that this diatom was dominant in the freshwater reactor attests that the sewage influent was digested to a great extent since Nitzschia palea Wm. Smith, which is the dominant diatom species observed in nutrient enriched bodies of water (e.g. drainage ditches) on Guam, was not the dominant species. Because Nitzschia palea is known as a common inhabitant of polluted waters (Lowe, 1974) it would have been interesting to observe a freshwater reactor at a shorter detention time to see if Gomphonema clevei remained the dominant species. Besides these two species a very small-sized Navicula species was also common.

In the 12,000 mg/1 TDS reactors Nitzschia palea was clearly dominant. Amphora turgida Gregory was also common. No Gomphonema clevei were present. Reactor 7 (with a shorter detention time) had greater numbers of Nitzchia palea than other reactors.

At the 22,000 mg/1 TDS concentration all reactor diatom communities were dominated by Amphora turgida though Nitzschia palea was still relatively abundant (20-40% of all frustules). In the 36,000 mg/1 TDS reactor Amphora turgida was the only diatom observed. Clearly, the replacement of Nitzchia palea by Amphora turgida is related to TDS concentrations.

Another interesting feature was the variation in the morpholoxy of the Nitzschia palea frustules between the freshwater reactor and the saline reactors. According to Lange-Bertalot and Simonsen (1978) the frustule of Nitzchia palea becomes slightly concave in polluted environments. The Nitzschia palea frustules were linear with rare specimens showing slight concavity in the freshwater reactor. The frustules in the remaining reactors showed various degrees of concavity. The cause, either TDS concentration or nutrient (plus carbon) enrichment, cannot be pinpointed. It should be kept in mind that in terms of biomass the green and bluegreen algae were clearly the dominant algal components of all reactors with the diatoms being a minor to negligible component.

Salinity Effects

As shown in Table 9, the exact effects of salinity level or total dissolved solids content on reactor performance are difficult to assess. From Table 9, there appears to be a trend for BOD and SS to decrease with increasing salinity levels for reactors with TDS in excess of 12,000 mg/1. However, the freshwater reactor (Reactor 1) had the lowest effluent BOD and SS values so it is impossible to accept this trend entirely. Presumably, the effect of increasing salinity changes significantly between the freshwater and 12,000 mg/1 TDS situations. Unfortunately, insufficient chemical and biological data were collected to further define this cause and effect relationship.

It is assumed that the superior performance of the freshwater reactor (Reactor 1) is due to a larger population of protozoans, crustaceans and insect larvae which acted as effluent polishers by consuming bacteria and algae. As the salinity level increased in the other reactors, fewer larger organisms were found which could act as effluent polishers. These organisms, which were abundant in the freshwater reactor, apparently could not adapt to the higher salinity levels of reactors 2 to 8. It is possible that predator organisms would have appeared in these tanks if the acclimation period had been longer or if the tanks had been seeded with marine organisms but this was not attempted.

Dispersion Study

Three of the reactors were used to determine reactor dispersion numbers and mean reactor detention times for the hydraulic loading used in the biological study. Dispersion numbers and reactor detention times were determined because they are required by Equation 5 to estimate the biological reaction rate coefficients for non-ideal fluid flow.

Determination of Reaction Rate Coefficients

Biological reaction rate coefficients, k, for reactors 1 to 8 were derived using Equation 5 from the BOD data, detention times and dispersion numbers given in Table 9. The derived reaction rates are presented in the same table.

As shown in Table 9, the reaction rate coefficients did not vary dramatically with salinity level. Reactors 2, 3 and 4 which had the same waste and hydraulic loadings had nearly identical reaction rate coefficients, 0.22, 0.21 and 0.22, respectively, although their salinity levels differed greatly. Similar trends are found with the reaction rate coefficients of reactors 5 to 8 although salinity effects seem to be slightly more significant for these loading rates.

DESIGN EXAMPLE FOR MICRONESIA

The results of this study indicate that aerobic waste stabilization ponds are an attractive alternative to convention mechanical secondary wastewater treatment facilities. In addition to their lower capital, operating and maintenance costs they appear to be capable of providing a higher degree of treatment than conventional facilities.

As an example, consider the design of a stabilization pond system for an area with 10,000 people. Important design parameters which must be estimated include:

1) <u>Hydraulic loading</u> - In Micronesia, water use figures vary dramatically from island to island and from village to village depending upon the availability of water and local water use customs. The selection of a per capita water use rate is therefore difficult. For the design example, a per capita water use rate of 380 l/d (100 gpd) was selected with 75% of this flow assumed to be discharged to the sewers. Neglecting infiltration, this would

result in a flow rate of 2850 m^3/day (.75 mgd) for a population of 10,000 people.

2) Waste Loading - The BOD of sewage varies greatly like water use throughout Micronesia but, in general, BOD values for sewage have been found to be substantially less than those observed in the United States. BOD values for Micronesian sewage typically range from 75 to 200 mg/l but are generally closer to 100 mg/l. For the purposes of the design example, an influent BOD of 150 mg/l is assumed.

3) Pond Characteristics - The waste stabilization pond design in this example is assumed to have a liquid depth of 1 m and to consist of a single large cell. In actuality, a waste stabilization pond system would probably consist of 3 or more ponds in series but the design based upon a single cell is conservative so it will be used here. The dispersion number required for Equation 5 is estimated to be in the range of 0.1 to 0.3 for a properly designed pond system (3 or more ponds in series).

4) Reaction Rate Coefficients, k - Based upon the laboratory model study, the reaction rate coefficients for municipal sewage can be estimated to range from 0.1 to 0.3 or more at 20°C. With an assumed ambient annual pond temperature of 27°C the reaction rate can be expected to range from about 0.15 to 0.50 with rates in the higher range being more likely.

5) Treatment Efficiency - The design assumes that a BOD reduction of 85% is required (EPA secondary treatment requirement). This would allow wastes with BOD values of up to 200 mg/l to be treated and still maintain a theoretical effluent of 30 mg/l BOD as required by the EPA standards. For the design example, the effluent BOD should be about 22.5 mg/l based upon Equation 5.

Almost all of the design parameters used above are conservative in nature. That means that the performance of the design example facility should be better than predicted.

Table 10 is a summary of the results obtained using Equation 5 for the design example. The table gives the required pond detention time and surface area required to achieve an 85% reduction in BOD over a range of dispersion numbers and reaction rate coefficients. Based upon the reactor experiments the authors believe that a reaction rate coefficient of 0.35 and dispersion number of .3 would be very conservative for freshwater ponds in Micronesia. Table 9 predicts a pond detention time of 7.7 days and a pond surface area of 2.2 hectares (5.4 acres) for these values of k and d. These values are considerably smaller than those recommended by the previous Micronesia waste stabilization pond study (Barrett, Harris & Assoc., 1979) which recommended a surface area of 3.8 hectares and a minimum detention time of 30 days for the design example conditions.

As indicated by the results of the reactor studies, the effects of elevated salinity levels are not very detrimental to the performance of waste stabilization ponds. In fact, reactors 7 and 8 which where saline had the highest biological reaction rate coefficients, although this was probably due to their higher waste loading rate.

CONCLUSIONS

1. Waste stabilization ponds have great potential for wastewater treatment in Micronesia. The climate of Micronesia is particularly favorable for waste stabilization ponds because of the high ambient temperatures and abundant sunshine for photosynthesis. The advantages of stabilization ponds include low capital, maintenance and operating costs, few mechanical components, low maintenance requirements, low sludge production rates, and efficient wastewater treatment.

2. Elevated salinity levels were found to increase effluent BOD values but were still well within the EPA limits. Increasing salinity levels above 12,000 mg/l TDS for a given waste loading did not decrease BOD removal efficiencies. In fact, there was a tendency for effluent BODs to decrease slightly as salinity increased above 12,000 mg/l TDS.

3. There was a significant increase in the TSS between the freshwater and the saline reactors. The freshwater reactor had low effluent TSS because algae stocks were reduced by predation. Very few algae predators were observed in the saline reactors and their effluents consequently had high TSS because of high algae concentrations. While TSS was observed to increase dramatically from the freshwater to the saline reactors, the TSS of the saline reactors for a given loading was found to decrease as the salinity level increased. The cause of this decrease was not determined.

4. The study does not adequately identify appropriate waste stabilization pond reaction rate coefficients for freshwater sewage in Micronesia. The study was intended to investigate the effects of salinity on pond performance and did not directly investigate freshwater reaction rates. Before large stabilization ponds are constructed in Micronesia, it is critical that additional research be conducted to more accurately determine reaction rate coefficents. Otherwise it will be difficult to design facilities which are both economic and efficient. Reaction rate coefficients determined in this study ranged from .21 to .45 at ambient Micronesian temperatures. At higher and more realistic loading rates and with freshwater sewage, much higher coefficients are expected.

5. The Wehner-Wilhem equation, equation 5, is suitable for designing waste stabilization ponds in Micronesia. It's use

however requires the careful selection of appropriate factors for the reaction rate coefficient, k, and the dispersion number, d. Because of the uncertainty in values of these parameters waste stabilization ponds should be designed only by qualified sanitary engineers with experience in stabilization pond design and operation.

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APPENDIX

TABLE A-1. BIOCHEMICAL OXYGEN DEMAND DATA (5-DAY, MG/L)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/l) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
5-10	1	20.3	33.7	21.5	21.1	18.2	22.8	10.0	39.1	-
12-10	8	8.0	10.8	23.0	20.0	11.1	31.0	22.6	-	175.0
18-10	14	1.5	7.3	4.6	3.3	10.7	18.0	5.0	10.7	-
26-10	22	0.8	6.9	4.2	7.7	6.5	5.0	6.5	13.4	155.0
3-11	30	9.2	9.1	6.9	8.5	6.3	5.4	13.2	11.4	-
17-11	44	4.4	6.1	5.3	8.1	5.8	7.0	11.9	3.9	-
26-11	53	2.7	2.5	9.5	10.7	9.4	9.5	4.6	5.3	41.4
1-12	58	2.3	5.0	8.5	4.3	8.9	9.9	10.5	7.8	112.6
6-12	63	1.9	8.5	6.3	4.0	8.1	3.9	9.9	9.4	89.3
11-12	68	2.0	7.3	5.3	5.7	12.7	9.4	11.1	11.4	122.9
16-12	73	2.8	7.9	4.7	4.8	16.7	5.7	9.8	6.4	69.6
MEAN		5.1	9.6	9.2	8.9	10.4	11.6	10.5	11.9	109.3
MAXIMUM		20.3	33.7	23.0	21.1	18.2	31.0	22.6	39.1	175.0
MINIMUM		0.8	2.5	4.2	3.3	5.8	3.9	4.6	3.9	41.4
STD. DEV.		5.5	7.9	6.4	5.9	3.9	8.3	4.7	9.5	43.5

TABLE A-2. BIOCHEMICAL OXYGEN DEMAND (FILTERED, 5-DAY, MG/L)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/l) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
26-11	53	4.2	12.1	6.1	3.6	4.7	2.9	3.3	3.0	-
1-12	58	1.6	1.6	1.7	0.9	0.6	1.6	1.1	1.3	47.1
6-12	63	1.1	0.5	0.6	0.6	2.0	0.5	0.5	0.7	31.3
11-12	68	0.8	0.7	0.8	2.1	1.0	0.9	2.6	1.6	43.2
16-12	73	1.1	1.3	0.9	1.4	3.1	1.5	2.7	1.4	22.2
MEAN		1.8	3.2	2.0	1.7	2.3	1.5	2.0	1.6	35.9
MAXIMUM		4.2	12.1	6.1	3.6	4.7	2.9	3.3	3.0	47.1
MINIMUM		0.8	0.5	0.6	0.6	0.6	0.5	0.5	0.7	22.2
STD. DEV.		1.2	4.4	2.1	1.1	1.5	0.8	1.1	0.8	9.8

TABLE A-3. SUSPENDED SOLIDS DATA (MG/L)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/1) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
5-10	1	10.4	7.2	16.2	6.8	4.8	15.4	4.6	18.0	23.6
7-10	3	0.7	11.6	10.6	12.8	4.3	9.8	6.3	12.3	–
12-10	8	1.7	2.7	6.8	10.0	5.2	12.4	9.6	14.3	78.3
15-10	11	1.4	8.0	5.1	17.4	15.6	8.5	9.6	8.2	76.0
18-10	14	1.6	10.0	4.4	4.1	8.9	5.4	14.9	10.6	–
22-10	18	0.1	6.8	3.3	2.5	6.4	6.6	56.4	8.2	–
26-10	22	1.8	23.5	6.8	3.9	43.0	11.2	14.0	6.0	25.5
2-11	29	1.5	17.0	23.4	24.8	16.0	26.0	36.4	14.4	101.0
15-11	42	0.5	35.5	24.4	78.6	37.6	12.8	36.0	14.8	45.0
19-11	46	1.1	32.0	20.8	25.4	44.0	37.0	32.5	15.8	127.0
27-11	53	5.9	37.6	28.4	14.7	40.0	36.8	–	22.3	–
1-12	58	3.0	56.0	15.7	53.1	32.9	50.8	48.0	43.6	66.0
6-12	63	0.2	47.6	43.2	13.7	57.2	43.2	53.2	36.8	57.3
11-12	68	8.9	33.2	25.2	17.8	36.4	35.2	40.0	44.4	30.8
16-12	73	1.2	32.5	10.1	12.1	34.6	20.0	35.8	20.5	93.7
MEAN		2.7	24.5	16.3	19.9	25.8	22.1	28.4	19.3	65.8
MAXIMUM		10.4	56.0	43.2	78.8	57.2	50.8	56.4	44.4	127.0
MINIMUM		0.1	2.7	3.3	2.5	4.3	5.4	4.6	6.0	23.6
STD. DEV.		3.1	16.0	10.9	19.8	17.1	14.4	17.4	12.0	31.8

TABLE A-4. VOLATILE SUSPENDED SOLIDS DATA (MG/L)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/l) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
5-10	1	17.6	4.8	12.0	4.0	1.8	11.7	2.1	14.0	16.9
7-10	3	–	11.6	9.4	8.8	3.4	7.3	7.4	10.3	–
12-10	8	1.7	2.7	6.4	8.3	5.2	10.8	9.6	13.4	73.0
15-10	11	0.7	4.6	3.6	11.9	13.6	6.2	6.8	7.3	57.0
18-10	14	0.7	7.4	2.7	1.6	6.0	3.8	13.4	8.0	–
22-10	18	–	4.4	1.2	0.4	3.8	3.8	40.0	5.2	–
26-10	22	1.8	20.0	4.2	1.8	42.0	9.6	13.6	5.2	20.0
2-11	29	1.5	14.5	16.6	14.8	16.0	19.3	33.2	14.4	90.0
15-11	42	–	31.5	23.6	36.8	22.8	8.4	20.5	17.6	11.0
19-11	46	1.1	28.8	17.0	20.8	41.1	29.4	28.3	15.0	87.3
27-11	53	4.1	29.6	21.4	11.2	33.6	29.2	16.5	18.8	86.3
1-12	58	1.3	41.2	12.7	34.5	32.0	37.2	38.0	28.8	29.2
6-12	63	–	41.6	34.8	11.2	51.6	35.6	49.2	33.2	39.3
11-12	68	7.1	33.6	25.6	15.6	35.2	30.4	37.2	42.0	26.0
16-12	73	0.7	29.7	9.4	11.8	34.1	18.2	35.2	19.1	61.3
MEAN		3.5	20.4	13.4	12.9	22.8	17.4	23.4	16.8	49.8
MAXIMUM		17.6	41.6	34.8	36.8	51.6	37.2	49.2	42.0	90.0
MINIMUM		0.7	2.7	1.2	0.4	1.8	3.8	2.1	5.2	11.0
STD. DEV.		4.8	13.6	9.4	10.5	16.2	11.5	14.2	10.2	28.4

TABLE A-5. PH DATA

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/l) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
5-10	1	7.5	7.7	7.7	8.1	7.8	7.8	7.9	7.8	–
7-10	3	7.8	8.0	8.0	8.1	8.0	8.0	8.0	8.0	–
15-10	11	9.2	8.6	8.0	8.1	8.6	8.0	8.4	8.1	–
18-10	14	8.5	8.6	8.4	8.2	8.5	8.2	8.4	8.0	–
22-10	18	9.2	9.1	8.8	8.6	8.8	8.5	8.6	8.3	–
26-10	22	8.9	9.3	9.1	8.6	9.2	8.8	9.1	8.5	7.7
15-11	42	8.9	9.5	9.4	9.3	9.6	9.4	9.5	9.5	7.4
17-11	44	8.5	9.2	9.1	9.0	9.3	9.1	9.2	9.3	7.5
19-11	46	8.9	9.3	9.2	9.1	9.3	9.2	9.4	9.3	7.7
22-11	49	8.3	9.1	9.1	9.1	9.3	9.2	9.4	9.4	7.8
27-11	53	8.7	9.3	9.2	9.2	9.4	9.3	9.4	9.4	7.4
1-12	58	9.2	9.0	9.0	9.0	9.1	9.0	9.0	9.1	7.3
6-12	63	8.9	9.3	8.9	8.7	8.9	8.7	8.8	8.8	–
11-12	68	8.4	8.7	8.7	8.6	8.8	8.8	8.8	8.8	7.3
16-12	73	9.2	9.5	9.3	9.1	9.4	9.4	9.5	9.5	7.5
MEAN		8.7	9.0	8.8	8.7	8.9	8.8	8.9	8.8	7.5
MAXIMUM		9.2	9.5	9.4	9.3	9.6	9.4	9.5	9.5	7.8
MINIMUM		7.5	7.7	7.7	8.1	7.8	7.8	7.9	7.8	7.3
STD. DEV.		0.5	0.5	0.5	0.4	0.5	0.5	0.5	0.6	0.2

TABLE A-6. TURBIDITY DATA (NTU)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/1) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
5-10	1	6.1	16.0	10.0	5.0	5.0	7.0	3.5	12.0	-
7-10	3	3.7	42.0	7.5	9.9	3.2	6.5	5.7	17.0	-
15-10	11	6.0	3.4	3.8	8.3	5.9	3.1	3.4	4.4	-
18-10	14	5.8	4.1	1.8	1.0	4.2	5.3	3.5	8.7	-
22-10	18	4.7	4.0	2.0	1.2	3.8	2.3	5.4	4.2	-
26-10	22	3.7	5.4	1.9	1.5	7.0	2.8	5.1	2.6	23.0
2-11	29	2.5	4.5	3.0	2.5	2.7	3.2	9.0	4.5	-
15-11	42	1.2	4.7	3.7	4.4	4.4	2.8	4.1	2.6	23.0
17-11	44	1.5	4.5	3.0	3.0	4.2	3.8	4.7	1.5	40.0
19-11	46	1.9	4.3	3.5	3.2	5.2	5.6	3.7	2.8	52.0
22-11	49	1.6	6.3	5.3	3.6	4.7	5.8	4.0	2.2	-
6-12	63	2.2	12.0	9.3	5.0	14.0	8.8	13.0	7.9	-
11-12	68	0.6	11.0	7.2	8.7	12.0	9.5	11.0	12.0	38.0
16-12	73	0.8	12.0	5.2	7.9	11.0	13.0	14.0	6.4	80.0
MEAN		3.0	9.6	4.8	4.7	6.2	5.7	6.4	6.3	42.7
MAXIMUM		6.1	42.0	10.0	9.9	14.0	13.0	14.0	17.0	80.0
MINIMUM		0.6	3.4	1.8	1.0	2.7	2.3	3.4	1.5	23.0
STD. DEV.		1.9	9.8	2.6	2.9	3.4	3.0	3.6	4.5	19.5

TABLE A-7. CONDUCTIVITY DATA (MHOS/CM)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/l) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
5-10	1	952.	16700.	35200.	26600.	15400.	33700.	16450.	36700.	-
7-10	3	902.	15950.	33600.	46700.	15750.	33100.	15570.	32700.	-
15-10	11	678.	12910.	28200.	52100.	14870.	24600.	14280.	28000.	868.
18-10	14	767.	14580.	27300.	49900.	18000.	30800.	17400.	30000.	-
22-10	18	772.	15800.	30560.	50900.	16550.	36700.	16420.	38280.	-
26-10	22	807.	18900.	37620.	53130.	16910.	39040.	20180.	36700.	904.
2-11	29	952.	18350.	37400.	52400.	17740.	35900.	18500.	31000.	-
15-11	42	892.	17100.	28700.	41700.	17400.	29700.	19400.	29300.	829.
17-11	44	968.	15850.	31500.	45600.	16370.	33100.	17690.	31600.	1052.
19-11	46	912.	15800.	31600.	45600.	16070.	33800.	18280.	34900.	1063.
22-11	49	1007.	16460.	33200.	47400.	16200.	34600.	15840.	36000.	987.
27-11	53	944.	15970.	33300.	44900.	16240.	34000.	16470.	35800.	1006.
1-12	58	816.	16800.	35400.	44000.	17360.	38100.	21200.	37300.	950.
6-12	63	798.	16310.	34500.	45500.	17070.	37700.	20900.	36200.	-
11-12	68	811.	15610.	35100.	50500.	20300.	36900.	17220.	35600.	865.
16-12	73	860.	16550.	34600.	50600.	16970.	35600.	17020.	35300.	980.
MEAN		865.	16290.	32986.	46720.	16825.	34208.	17676.	34086.	950.
MAXIMUM		1007.	18900.	37620.	53130.	20300.	39040.	21200.	38280.	1063.
MINIMUM		678.	12910.	27300.	26600.	14870.	24600.	14280.	28000.	829.
STD. DEV.		88.	1313.	3012.	6146.	1214.	3502.	1899.	3075.	77.

TABLE A-8. TOTAL DISSOLVED SOLIDS DATA (MG/L)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/l) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
7-10	3	812.	10400.	21500.	32000.	11300.	17100.	11100.	20900.	-
15-10	11	452.	9784.	21844.	47148.	12664.	17916.	11500.	20280.	
18-10	14	888.	10948.	19424.	43312.	12124.	22668.	11572.	22940.	506.
22-10	18	452.	10640.	19844.	36976.	12112.	23116.	11356.	24848.	-
26-10	22	508.	13024.	23960.	37180.	12656.	25132.	11456.	22948.	-
15-11	42	732.	11172.	-	36604.	11936.	22380.	12940.	20364.	548.
17-11	44	512.	12116.	21868.	45984.	11696.	23128.	13196.	21236.	644.
19-11	46	576.	11560.	24488.	39080.	11932.	22636.	13972.	26000.	532.
27-11	53	532.	10652.	19948.	30808.	11116.	21584.	10800.	22600.	712.
1-12	58	572.	10980.	27904.	22588.	11524.	24972.	11912.	-	756.
6-12	63	540.	11308.	22764.	30652.	11468.	24572.	11944.	23536.	1072.
16-12	73	456.	11012.	21760.	37832.	11900.	22492.	11756.	23876.	928.
MEAN		586.	11133.	22300.	36847.	11869.	22308.	11959.	22675.	536.
MAXIMUM		888.	13024.	27904.	47148.	12664.	25132.	13972.	26000.	693.
MINIMUM		452.	9784.	19424.	22588.	11116.	17100.	10800.	20280.	1072.
STD. DEV.		139.	799.	2342.	6731.	464.	2393.	897.	1767.	506.
										186.

TABLE A-9. NON-VOLATILE DISSOLVED SOLIDS DATA (MG/L)

DATE	DAY	REACTOR NUMBER (Total dissolved solids in mg/1) 1 (600)	2 (12,000)	3 (22,000)	4 (36,000)	5 (12,000)	6 (22,000)	7 (12,000)	8 (22,000)	INFLUENT (700)
15-11	42	564.	9352.	–	27312.	9748.	17920.	10916.	17880.	544.
17-11	44	500.	9528.	17828.	28172.	9716.	18676.	10756.	17704.	460.
19-11	46	564.	9700.	18092.	29156.	9820.	19252.	11472.	20416.	652.
27-11	53	464.	9232.	17308.	26676.	9344.	18384.	9320.	19748.	668.
1-12	58	450.	8800.	17180.	23016.	9028.	19416.	9860.	–	860.
6-12	63	512.	9660.	18752.	25428.	9612.	21084.	10308.	20544.	912.
11-12	68	468.	9396.	18944.	30708.	10404.	20056.	9844.	20204.	892.
16-12	73	396.	9364.	18480.	29824.	9676.	19064.	9808.	19292.	464.
MEAN		491.	9379.	18083.	27537.	9669.	19232.	10286.	19398.	682.
MAXIMUM		564.	9700.	18944.	30708.	10404.	21084.	11472.	20544.	912.
MINIMUM		396.	8800.	17180.	23016.	9028.	17920.	9320.	17704.	460.
STD. DEV.		53.	265.	636.	2345.	369.	930.	667.	1089.	175.

Table A-10. Tracer study results.

DAY	Reactor 1, 2 l/d C	DT	D	Reactor 2, 3 l/d C	DT	D	Reactor 3, 5 l/d C	DT	D
1	51.	1.0	0.0	56.	1.0	0.0	100.	1.0	0.0
2	194.	1.8	0.03	106.	1.7	0.04	740.	1.9	0.01
3	443.	2.6	0.03	356.	2.6	0.03	6753.	2.9	0.01
4	1030.	3.4	0.03	1589.	3.7	0.02	8348.	3.5	0.02
5	2456.	4.4	0.02	5253.	4.6	0.01	7157.	3.9	0.02
6	6658.	5.4	0.02	8051.	5.3	0.01	6658.	4.4	0.03
7	6711.	6.0	0.02	7998.	5.9	0.01	5349.	4.8	0.04
8	7011.	6.6	0.02	8098.	6.4	0.02	4380.	5.2	0.05
9	7032.	7.1	0.02	7859.	7.0	0.02	3304.	5.4	0.06
10	7230.	7.6	0.03	6544.	7.4	0.03	2404.	5.7	0.07
11	7077.	8.2	0.03	6023.	7.8	0.03	2268.	5.9	0.08
12	7136.	8.7	0.04	5753.	8.2	0.04	2093.	6.2	0.10
13	7077.	9.2	0.04	5178.	8.6	0.05	1601.	6.4	0.11
14	6795.	9.7	0.04	4302.	9.0	0.05	969.	6.6	0.12
15	6354	10.1	0.05	3833.	9.3	0.06	906.	6.7	0.13
16	6211.	10.6	0.05	3833.	9.3	0.06	699.	6.8	0.13
17	5939.	11.0	0.06	2706.	9.9	0.07	496.	6.9	0.14
18	5658.	11.5	0.06	2320.	10.1	0.07	335.	7.0	0.15
19	5295.	11.9	0.06	2018.	10.3	0.08	281.	7.0	0.16
20	4859.	12.3	0.07	1561.	10.5	0.08	241.	7.1	0.16
21	4278.	12.6	0.07	1261.	10.7	0.09	188.	7.1	0.17
22	3728.	13.0	0.08	996.	10.8	0.09	161.	7.2	0.17
23	3433.	13.3	0.08	845.	10.9	0.10	140.	7.2	0.18
24	3195.	13.6	0.08	714.	11.0	0.10	119.	7.3	0.19
25	2848.	13.8	0.09	548.	11.1	0.10	104.	7.3	0.19
26	2458.	14.1	0.09	385.	11.2	0.11	86.	7.3	0.20
27	2141.	14.3	0.09	357.	11.2	0.11	78.	7.4	0.20
28	1869.	14.5	0.10	326.	11.3	0.11			
29	1542.	14.7	0.10	293.	11.4	0.12			
30	1308.	14.8	0.10	268.	11.4	0.12			
31	1154.	15.0	0.11	242.	11.5	0.12			
32	953.	15.1	0.11	225.	11.5	0.13			
33	756.	15.2	0.11	201.	11.6	0.13			
34	696	15.3	0.12	198.	11.6	0.13			
35	642.	15.4	0.12	170.	11.7	0.14			
36	616.	15.5	0.12	160.	11.7	0.14			
37	542.	15.6	0.13	133.	11.7	0.14			
38	514.	15.7	0.13	125.	11.8	0.15			
39	487.	15.8	0.13	116.	11.8	0.15			
40	471.	15.8	0.13	110.	11.8	0.16			
41	449.	15.9	0.14	105.	11.9	0.16			
42	435.	16.0	0.14	98.	11.9	0.16			

where: C = effluent specific conductivity, umho/cm
DT = reactor detention time from Eq. 6, days
D = dispersion number from Eq. 7 and 8.