

# WASTEWATER STABILIZATION PONDS ON THE MEXICO-USA BORDER PERFORMANCE AND UPGRADING POTENTIAL

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# TABLE OF CONTENTS

ACKNOWLE	DGEMENTSi
CHAPTER 1	INTRODUCTION1-1
	SUMMARY AND CONCLUSIONS
CHAPTER 2	DESIGN OF POND SYSTEMS
	PRELIMINARY TREATMENT 2-4
	FACULTATIVE PONDS
	PARTIAL-MIX AERATED PONDS 2-13 Partial mix design model 2-13 Selection of reaction rate constants 2-14 Temperature effects 2-14 Pond configuration 2-15 Mixing and aeration 2-16 Example 2-17 Solution 2-17
	COMPLETE MIX AERATED POND
	CONTROLLED DISCHARGE PONDS 2-25
	COMPLETE RETENTION PONDS
	COMBINED SYSTEMS
	ANAEROBIC PONDS
1	PATHOGEN REMOVAL

	SUSPENDED SOLIDS REMOVAL
. •	NITROGEN REMOVAL
	PHOSPHORUS REMOVAL
	PHYSICAL DESIGN AND CONSTRUCTION
	STORAGE PONDS FOR LAND TREATMENT SYSTEMS 2-41
CHAPTER 3	MEXICALI, BAJA CALIFORNIA NORTE (BCN) MEXICO WASTEWATER TREATMENT SYSTEM
	SYSTEM DESCRIPTION 3-1
	PERFORMANCE OF SYSTEM
	PREDICTED PERFORMANCE
	REDESIGN OF MEXICALI BAJA CALIFORNIA NORTE (BCN)I SYSTEM

<b>CHAPTER 4</b>	NOGALES INTERNATIONAL WASTEWATER TREATMENT
	PLANT
	SYSTEM DESCRIPTION
	DESIGN ANALYSIS
	Design Assumptions
	Design Predictions
	20 °C
	11 °C
	Oxygen Requirements
	Complete Mix Cell
	Partial Mix Cells 4-12
	PERFORMANCE OF SYSTEM
<b>CHAPTER 5</b>	REYNOSA, TAMAULIPAS, MEXICO WASTEWATER TREATMENT
	SYSTEM
<b>CHAPTER 6</b>	OTHER UPGRADING TECHNOLOGIES 6-1
	CONSTRUCTED WETLANDS
	LAND APPLICATION OF LAGOON EFFLUENT 6-2
	DUCKWEED COVER 6-3
	SYNOPSIS OF PERFORMANCE EXPECTATIONS 6-3
REFERENCE	S
APPENDIX A	
	PLANT MONTHLY REPORTS

# CHAPTER 1 INTRODUCTION

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#### **CHAPTER 1**

#### INTRODUCTION

Stabilization ponds are one of the most widely used wastewater treatment technologies. Facultative ponds have been used worldwide for over 3,000 years under the full wide range of climatic conditions. Stabilization ponds have been in use in the USA since the early 1900s. Today there are approximately 7,000 pond systems in use in the USA. Variations of the basic stabilization form of biological treatment also have been developed to attempt to increase the loading rates, reduce detention times, decrease water loss through evaporation and to limit discharges to surface waters. For the most part, these variations are fully proven and widely used in many parts of the USA and Mexico. In addition to the proven modifications, several innovative technologies (e.g. Advanced Integrated Pond Systems, Lemna duckweed systems and rock filters) have great potential for retrofitting basic stabilization ponds and enhancing their performance along the US/Mexican border.

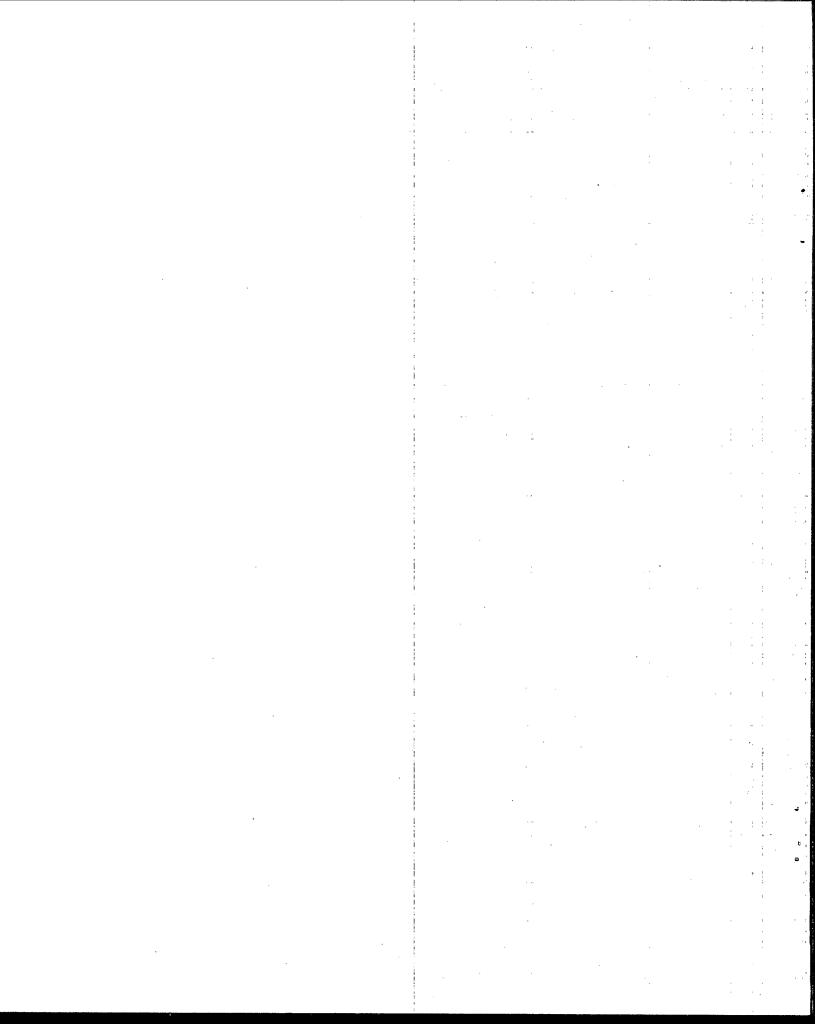
At the present time there are significant needs along the border for effective, low cost, low maintenance, wastewater treatment concepts. Pond systems, either new facilities or upgrading existing ponds, offer the potential for resolution of these needs in many locations.

Under Contract No. 68-C2-0102, Work Assignment 2-16, Parsons Engineering Science, Inc. (Parsons ES) in conjunction with E. Joe Middlebrooks and Sherwood C. Reed of Environmental Engineering Consultants of Norwich, Vermont has been requested to provide an evaluation of the use of pond systems for treating wastewater along the US/Mexican border. The objective of this work assignment is to provide useful information and guidance for planners and engineers for use in considering the potential application of pond systems and in preparing preliminary project designs for application along the US/Mexican border area.

### SUMMARY AND CONCLUSIONS

- 1. The wastewater stabilization ponds evaluated along the Mexican-USA border were not constructed as designed (i.e., aeration omitted). Consequently the systems are not performing at a satisfactory level; however, all of the systems visited appear to be upgradable to efficient and reliable facilities.
- 2. By converting the systems to combinations of complete and partial mix aerated lagoons, the systems can be expected to produce effluent BOD concentrations of less than 10 mg/l and process two to three times the current flow rate.
- 3. Installing multiple outlets and multiple drawoff depths in the final cells of all three systems would materially improve the effluent TSS quality.
- 4. Improvements in the hydraulic characteristics of the lagoon systems would have a significant impact on performance.
- 5. Regardless of the modifications made to the systems, to achieve the maximum efficiency from the lagoon systems it is essential that proper operation be provided. For example, it will do little good if multiple depth drawoff outlets are installed but not used.
- 6. Existing wastewater stabilization lagoons can be economically upgraded to produce an effluent of practically any quality desired by the utilization of many processes, many of which are briefly discussed in Chapter 2. Design of Pond Systems (intermittent sand filtration, rock filters, microstrainers, dissolved air flotation, centrifugation, etc.).
- 7. In addition to the upgrade methods referred to above, wetlands, duckweed, and land treatment appear to be viable upgrade alternatives for lagoon effluents, but additional study is needed to confirm their viability.
- 8. Two of three systems (Mexicali, Baja California Norte (BCN), and Reynosa) visited are grossly overloaded, but in view of the operational effort and inadequate design, the systems perform remarkably well.

- 9. The Nogales system is relatively new, well operated, and produces an excellent quality effluent. Relatively simple modifications in the lagoon system would double or triple the flow rate that could be processed by the lagoon system. Modifications in the solids removal activities would be necessary if higher flow rates are to be processed by the facility.
- Less than optimal solids removal processes have been employed at the Nogales facility, and modifications are needed to maximize the efficiency of the units.
- 11. At the Nogales facility, reductions in the hydraulic residence time in the final settling cells probably would reduce the algae load going to the automatic backwash filters. Normal granular media filtration has been shown to be capable of removing only one-third of the solids in lagoon effluents without chemical addition. Consideration should be given to some type addition to improve the filter efficiency.



# CHAPTER 2 DESIGN OF POND SYSTEMS

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#### **CHAPTER 2**

## **DESIGN OF POND SYSTEMS**

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, over 7000 pond systems are used in the U.S. for the treatment of municipal and industrial wastewaters (USEPA, 1980), under a wide range of weather conditions ranging from tropical to Arctic. Large numbers of pond systems are used throughout the world (WHO, 1987). Many types of pond systems are utilized on both sides along the border between Mexico and the USA. Three pond systems located along the Mexico-USA border were visited and evaluated. The results of these evaluations are presented in Part II. Pond systems in Mexicali BCN and Reynosa, Mexico, and Nogales, Arizona, USA were visited. These pond systems are used alone or in combination with other wastewater treatment processes.

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 most basic classification depends on the dominant biological reactions occurring in the pond, and the four principal types are:

- o Facultative (aerobic-anaerobic) ponds
- o Aerated ponds (Complete and Partial Mix)
- o Aerobic Ponds
- o Anaerobic ponds

All four types depend on the interaction of the in-situ biological components for treatment and can be considered to be "natural treatment systems." General design features and performance expectations for pond systems and a modification of pond systems using water hyacinths are presented in Table 2-1.

The most common type is the facultative pond. Other terms which are commonly applied are oxidation pond, sewage 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. The 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

Table 2-1
Design Features and Expected Performance for Aquatic Treat, ment Units (Bastian and Reed, 1979; Middlebrooks et al., 1981; and USEPA, 1983)

	Effluent Characteristics mg/L	BOD 20-40 TSS 80-140	BOD 30-40 TSS 40-100	BOD 30-40 TSS 30-60	BOD 10-30 TSS 10-40	BOD <30 TSS <30	BOD < 10 TSS < 10 TP < 5
	Organic Loading kg/na-day	40-120	22-67	50-200	1st Cell = Facultative or Aerated Pond	<30	<50
	Depth meters	1-1.5	1.5-2.5	2-6	3-5	<1.5	V
Typical Criteria	Detention Time days	10-40	25-180	7-20	100-200	30-50	9<
	Climate Needs	Warm	None	None	None	Warm	Warm
	Treatment Goals	Secondary	Secondary	Secondary, Polishing	Secondary, Storage, Polishing	Secondary	AWT, with Secondary Input
	Unit	Oxidation Pond	Facultative Pond	Partial-Mix Aerated Pond	Storage, and Controlled-Discharge Ponds	Hyacinth Ponds	Hyacinth Ponds
	í	J	<b>H</b>	PH 1H	2-2	H	

organic material in the upper layer. The algae are necessary for oxygen production, but their presence in the final effluent represents one of the most serious performance problems associated with facultative ponds.

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 detention times, and the effluent is discharged once or twice per year when the effluent quality and stream conditions are satisfactory. A variation of the controlled discharge pond, used in the southern U.S. is called a hydrograph controlled release lagoon. The pond discharge is matched to periods of high flow in the receiving stream, using the stream hydrograph as the control.

In an aerated pond, oxygen is supplied mainly through mechanical or diffused aeration. 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. Aerated ponds can be designed as complete mix reactors or as partial mix reactors. In the former case sufficient energy must be used to keep the pond contents in suspension at all times. The basic design of a complete mix reactor is similar to that of an activated sludge system without sludge recycle.

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. Aerobic ponds are limited to warm sunny climates and are used infrequently in the United States.

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 1 to 50 days depending upon the environment in which they function. The principal biological reactions occurring are acid formation and methane fermentation. 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. They do not have wide application to the treatment of municipal wastewater in the USA, but are used successfully in many areas of the world.

#### 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. Design criteria and examples for preliminary treatment components can be found in a number of references, as well as in equipment manufacturer's catalogs (Al-Layla et al., 1980; Metcalf and Eddy, 1991; Ten State Standards, 1978; Wallace, 1978; and Water Pollution Control Federation, 1977). 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.

#### **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, the thermal overturn of the pond contents can result in significant quantities of benthic 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 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 these ponds have been developed. These include the ideal plug flow and complete mix models, as well as models proposed by Fritz, et al. (1979); Gloyna (1971); Larson (1974), Marais (1970); McGarry and Pescod (1970), Oswald et al. (1970); and Thirumurthi (1974). Several produce satisfactory results, but the use of some may be limited because of the difficulty in evaluating coefficients or by the complexity of the model.

# Areal loading rate method

Canter and Englande (1970) reported that most states have design criteria for organic loading and/or hydraulic detention time for facultative ponds.

These criteria are assumed to ensure satisfactory performance; however, repeated violations of effluent standards by pond systems that meet state design criteria indicate the inadequacy of the criteria. A summary of the state design criteria for each location and actual design values for organic loading and hydraulic detention time for four facultative pond systems evaluated by the Environmental Protection Agency (Middlebrooks, 1987 and USEPA, 1981) are shown in Table 2-2. Also included is a list of the months the federal effluent standards for  $BOD_5$  were exceeded. The actual organic loading for the four systems is nearly equal, but the system in Corinne, UT consistently satisfied the federal effluent standard. This may be a function of the larger number of cells in the Corinne system; seven as compared to three for the others. More hydraulic short-circuiting is likely to occur in the three cell systems, resulting in an actual detention time which was shorter than exists in the Corinne system. The detention time may also be affected by the location of the pond cell inlet and outlet structures.

Based on many years of experience, the following loading rates for various climatic conditions are recommended for use in designing facultative pond systems. For average winter air temperatures above 15°C (59°F), a BOD $_5$  loading rate range of 45 to 90 kg/ha-d (40-80 lb/ac-d) is recommended. When the average winter air temperature ranges between 0° and 15°C (32° to 59°F) the organic loading rate should range between 22 and 45 kg/ha-d (20-40 lb/ac-d). For average winter temperatures below 0°C (32°F) the organic loading should range from 11 to 22 kg/ha-d (10-20 lb/ac-d).

The BOD loading rate in the first cell is usually limited to 40 kg/ha-d (35 lb/ac/d) or less, and the total hydraulic detention time in the system is 120 to 180 days in climates where the average air temperature is below 0°C (32°F). In mild climates where the air temperature is greater than 15°C (59°F), loadings on the primary cell can be 100 kg/ha-d (89 lb/ac-d).

### Gloyna equation

Gloyna (1971) has proposed the following empirical equation for the design of facultative wastewater stabilization ponds:

$$V = (3.5 \times 10^{-5})(Q)(La)[\theta^{(35-T)}](f)(f')$$
 (1)

where

 $V = pond volume, m^3$ 

Q = influent flow rate, L/d.

La = ultimate influent BOD or COD, mg/l

EPA, 1983)		Months	BOD Exceeded	Tigur SS	Oct., Feb., Mar., Apr.	Nov., July	Mar., Apr., Aug.	None
, 1982; USI	n Time		Actual		107	214	231	70 88 (c)
brooks et al.	Theoretical Detention Time		Design		57	79	47	180
rabie 2-2 nance Data from EPA Pond Studies (Middlebrooks et al., 1982; USEPA, 1983)	Theoreti	State	Design Standard		None	None	None	180
rabie 2-2 om EPA Pond 9	Loading (kg BOD/ha-day)		Actual 1974-1975		16.2	17.5	18.8	29.7 (a) 14.6 (b)
nce Data fr	ading (kg B		Design		19.6	43	38.1	36.2
and Performa	1.1	State	Design Standard		39.3	56.2	38.1	45
Summary of Design and Perform			Location		Peterborough, NH	Kilmichael, MS	Eudora, KS	Corinne, UT

 $\theta$  = temperature correction coefficient = 1.085

T = pond temperature, °C.

f = algal toxicity factor.

f' = sulfide oxygen demand.

The  $BOD_5$  removal efficiency is projected to be 80 to 90 percent based on unfiltered influent samples and filtered effluent samples. A pond depth of 1.5 m (5 ft) is suggested for systems with significant seasonal variations in temperature and major fluctuations in daily flow. The surface area design using Eq. 1 should always be based on a 1 m (3 ft) depth. The algal toxicity factor (f) is 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 sulfate equivalent ion concentration of less than 500 mg/l. The design temperature is usually selected as the average pond temperature in the coldest month. Sunlight is not considered to be critical in pond design, but can be incorporated into Eq. 1 by multiplying the pond volume by the ratio of sunlight at the design location to the average found in the southwestern United States.

The Gloyna method was evaluated using the data referenced in Table 2-2. The equation giving the best fit of the data is shown below as Eq. 2. There was considerable scatter to the data, but the relationship is statistically significant.

$$V = 0.035Q(BOD)(1.099)^{L/GHT} (35-7)/250$$
 (2)

where

BOD = BOD<sub>5</sub> in the system influent, mg/l.

LIGHT = solar radiation in langleys.

 $V = pond volume, m^3$ 

Q = influent flow rate, m<sup>3</sup>/day

T = pond temperature, °C

# Complete-mix model

The Marias & Shaw (1961) equation is based on a complete mix-model and first order kinetics. The basic relationship is shown in Eq. 3.

$$\frac{C_n}{C_0} = \left[\frac{1}{1 + k_c t_n}\right]^n \tag{3}$$

where

 $C_n = effluent BOD_5 concentration, mg/l.$ 

 $C_o$  = influent BOD<sub>5</sub> concentration, mg/l.

 $k_c$  = complete mix first order reaction rate, d-1.

 $t_n$  = hydraulic residence time in each cell, d.

n = number of equal sized pond cells in series.

The proposed upper limit for the  $BOD_5$  concentration  $(C_e)_{max}$  in the primary cells is 55 mg/l to avoid anaerobic conditions and odors. The permissible depth of the pond, d in meters, was found to be related to  $(C_e)_{max}$  as follows:

$$(C_e)_{\text{max}} = \frac{700}{1.9d + 8} \tag{4}$$

where  $(C_e)_{max}$  is the maximum effluent BOD, 55 mg/l, and d is the design depth of the pond in meters.

The influence of water temperature on the reaction rate is estimated using Eq. 5.

$$k_{cT} = k_{c35} (1.085)^{T-35} (5)$$

where

 $k_{cT}$  = reaction rate at water temperature T, day<sup>-1</sup>

 $k_{c35}$  = reaction rate at 35°C

 $= 1.2 \text{ days}^{-1}$ 

T = operating water temperature, °C.

# Plug flow model

The basic equation for the plug-flow model is:

$$\frac{C_e}{C_o} = \exp\left[-k_\rho t\right] \tag{6}$$

where

 $C_e$  = effluent BOD<sub>5</sub> concentration, mg/l.

 $C_o = influent BOD_5 concentration, mg/l.$ 

 $k_p$  = plug flow first order reaction rate, days<sup>-1</sup>

t = hydraulic residence time, days

The reaction rate  $(k_p)$  varies with the BOD loading rate as shown in Table 2-3.

Table 2-3
Variation of the Plug-Flow Reaction Rate Constant with
Organic Loading Rate (Neel et al., 1961)

Organic Loading Rate kg/ha-day <sup>a</sup>	k <sub>p</sub> b days <sup>-1</sup>
22	0.045
45	0.071
67	0.083
90	0.096
112	0.129

 $<sup>^{</sup>a}$  kg/ha-d x 0.8907 = Ib/ac-d.

The influence of water temperature on the reaction rate constant can be determined with Eq. 6a.

$$k_{pT} = k_{p20} (1.09)^{T-20}$$
 (6a)

<sup>&</sup>lt;sup>b</sup> reaction rate constant at 20°C.

where  $k_{pT}$  = reaction rate at temperature T, days<sup>-1</sup>

 $k_{p20}$  = reaction rate at 20°C, days<sup>-1</sup>

T = operating water temperature, °C.

# Wehner Wilhelm equation

Thirumurthi (1974) found that the flow pattern in facultative ponds is somewhere between ideal plug-flow and complete-mix, and he recommended the use of the following chemical reactor equation developed by Wehner and Wilhelm (1956) for chemical reactor design.

$$\frac{C_e}{C_0} = \frac{4ae^{1/(2D)}}{(1+a)^2(e^{a/2D}) - (1-a)^2(e^{-a/2D})}$$
(7)

where  $C_0$  = influent BOD concentration, mg/l

C<sub>e</sub> = effluent BOD concentration, mg/l

e = base of natural logarithms, 2.7183

 $a = (1 + 4ktD)^{0.5}$ 

k = 1st order reaction rate constant, days<sup>-1</sup>

t = hydraulic residence time, days

D = dimensionless dispersion number

 $= H/vL = H_{\bullet}/L^{2}$ 

H = axial dispersion coefficient, area per unit time

v = fluid velocity, length per unit time

L = length of travel path of a typical particle

Thirumurthi (1974) prepared the chart shown in Fig.2-1 to facilitate the use of Eq.7. The dimensionless term kt is plotted versus the percentage of BOD remaining for dispersion numbers ranging from zero for an ideal plug flow

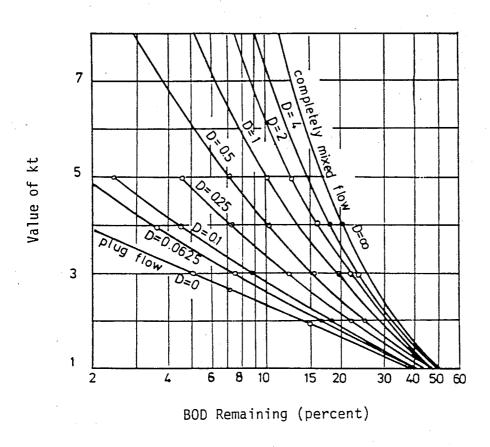


FIGURE 2-1 WEHNER AND WILHELM EQUATION CHART (THIRUMURTHI, 1974).

unit to infinity for a completely mixed unit. Dispersion numbers measured in wastewater ponds range from 0.1 to 2.0 with most values less than 1.0. 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 can have an equal effect. If the chart in Fig. 2-1 is not used, Eq. 7 can be solved on a trial and error basis.

To improve on the selection of a D value for use in Eq. 7, Polprasert and Bhattarai (1985) developed Eq. 8 based on data from pilot and full scale pond systems.

$$D = \frac{0.184[tv(W+2d]^{0.489}(W)^{1.511}}{(Ld)^{1.489}}$$
(8)

where

D = dimensionless dispersion number

t = hydraulic residence time, days

 $v = \text{kinematic viscosity, } m^2/\text{day}$ 

d = liquid depth of pond, m

W = width of pond, m

L = length of pond, m

The hydraulic residence time used to derive Eq. 8 was determined by tracer studies; therefore, it is still difficult to estimate the value of D to use in Eq. 7. A good approximation is to assume that the actual hydraulic residence time is half that of the theoretical hydraulic residence time.

The variation of the reaction rate constant k in Eq. 7 with the water temperature is determined with Eq. 9.

$$k_T = k_{20} (1.09)^{T-20} \tag{9}$$

where  $k_T$  = reaction rate at water temperature T, days<sup>-1</sup>

 $k_{20}$  = reaction rate at 20°C = 0.15 days<sup>-1</sup>

T = operating water temperature, °C

# Comparison of facultative pond design models

Because of the many approaches to the design of facultative ponds it is not possible to recommend the "best" procedure. An evaluation of the design methods presented above, with operational data referenced in Table 2-2, failed to show that any of the models are superior to the others in terms of predicting the performance of facultative pond systems (Middlebrooks et al., 1978; USEPA, 1981; and Reed et al., 1995).

#### PARTIAL-MIX AERATED PONDS

In the partial mix aerated pond system the aeration serves only to provide an adequate oxygen supply, and there is no attempt to keep all of the solids in suspension in the pond as is done with complete mix and activated sludge systems. Some mixing obviously occurs and keeps portions of the solids suspended; however, an anaerobic degradation of the organic matter that settles does occur. The system is sometimes referred to as a facultative aerated pond system.

Even though the pond is only partially mixed, it is conventional to estimate the BOD removal using a complete mix model and first order reaction kinetics. Recent studies (Middlebrooks, 1987) have shown that a plug flow model and first order kinetics more closely predict the performance of these ponds when either surface or diffused aeration is used. However, most of the ponds evaluated in this study were lightly loaded and the reaction rates calculated are very conservative because the rate decreases as the organic loading decreases (Neel et al., 1961). Because of the lack of better design reaction rates, it is still necessary to design partial mix ponds using complete mix kinetics.

### Partial mix design model

The design model using first order kinetics and operating "n" number of equal sized cells in series is given by Eq. 10.

$$\frac{C_n}{C_0} = \frac{1}{[1 + (kt/n)]^n}$$
 (10)

where  $C_n$  = effluent BOD concentration in cell n, mg/l

 $C_o$  = influent BOD concentration, mg/l

k = first order reaction rate constant, days<sup>-1</sup>

= 0.276 day<sup>-1</sup> at 20°C (assumed to be constant in all cells).

t = total hydraulic residence time in pond system, days

n = number of cells in the series.

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

$$\frac{C_n}{C_0} = \left(\frac{1}{1 + k_1 t_1}\right) \left(\frac{1}{1 + k_2 t_2}\right) \bullet \bullet \bullet \left(\frac{1}{1 + k_n t_n}\right) \tag{11}$$

where  $k_1$ ,  $k_2$ ,... $k_n$  are the reaction rates in cells 1 through n (all usually assumed equal for lack of better information) and  $t_1$ ,  $t_2$ ,... $t_n$  are the hydraulic residence times in the respective cells.

It has been shown (Mara, 1975) that a number of equal volume reactors in series is more efficient than unequal volumes; however, due to site topography or other factors there may be cases where it is necessary to construct cells of unequal volume.

Selection of reaction rate constants. The selection of the k value is the critical decision in the design of any pond system. A design value of 0.276 day is recommended by the *Ten States Standards (1978)* at 20°C and 0.138 day at 1°C. Using these values to calculate the temperature coefficient yields a value of 1.036. Boulier and Atchinson (1975) recommended values of k 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 are used in the calculation. Reid (1970) suggested a k value of 0.28 at 20°C and 0.14 at 0.5°C based on research with partial mix ponds aerated with perforated tubing in central Alaska. These values are essentially identical to the *Ten States Standards* recommendations.

**Temperature effects.** The influence of temperature on the reaction rate is defined by Eq. 13.

$$k_T = k_{20} \theta^{T_{\rm w}-20} \tag{13}$$

where  $k_T$  = reaction rate at temperature T, days<sup>-1</sup>

 $k_{20}$  = reaction rate at 20°C, days<sup>-1</sup>

⊖ = temperature coefficient = 1.036

 $T_w$  = temperature of pond water, °C.

The pond water temperature  $(T_w)$  can be estimated using the following equation developed by Mancini and Barnhart (1976).

$$T_{w} = \frac{AfT_{a} + QT_{i}}{Af + Q} \tag{14}$$

where

T<sub>w</sub> = pond water temperature, °C

T<sub>a</sub> = ambient air temperature, °C

A = surface area of pond, m<sup>2</sup>

f = proportionality factor = 0.5.

Q = wastewater flow rate, m<sup>3</sup>/day

An estimate of the surface area is made based on Eq.12, corrected for temperature, and then the temperature is calculated using Eq. 14. After several iterations, when the water temperature used to correct the reaction rate coefficient agrees with the value calculated with Eq. 14, the selection of the detention time in the system is completed.

# Pond configuration

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 cells 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 condition. The dimensions of the cells can be calculated by Eq.15.

$$V = [LW + (L-2sd)(W-2sd) + 4(L-sd)(W-sd)] \frac{d}{6}$$
(15)

where  $V = \text{volume of pond or cell, m}^3$  (ft<sup>3</sup>).

L = length of pond or cell at water surface, m (ft).

W = width of pond or cell at water surface, m (ft).

s = slope factor (i.e.; 3:1 slope, s = 3)

d = depth of pond, m (ft).

# Mixing and aeration

In most municipal systems, the oxygen requirements control the power input required for partial mix pond systems. A complete-mix system can require approximately ten times the power as a system designed to satisfy the oxygen requirements only. There are several rational equations available to estimate the oxygen requirements for pond systems (Al-Layla et al., 1980; Benefield and Randall, 1980; Gloyna, 1976; and Metcalf and Eddy, 1991). In most cases partial mix system design is based on the BOD entering the system to estimate the biological oxygen requirements. After calculating the required rate of oxygen transfer, equipment manufacturers' catalogs should be used to determine the zone of complete oxygen dispersion by surface, helical, or air gun aerators or the proper spacing of perforated tubing. Equation 16 is used to estimate oxygen transfer rates.

$$N = \frac{N_a}{\alpha \left[ \frac{C_{SW} - C_L}{C_S} \right] 1.025^{T_w - 20}}$$
(16)

where

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

 $N_a$  = oxygen required to treat the wastewater, kg/hr (usually taken as 1.5 x the organic loading entering the cell)

 $\alpha$  = (oxygen transfer in wastewater)/(oxygen transfer in tap water) = 0.9

C<sub>L</sub> = minimum DO concentration to be maintained in the wastewater, assume 2 mg/l  $C_s$  = oxygen saturation value of tap water at 20°C and one atmosphere pressure

$$= 9.17 \text{ mg/l}.$$

 $T_w$  = wastewater temperature, °C.

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

 $\beta$  = (wastewater saturation value)/tap water oxygen saturation value) = 0.9

Css = tap water oxygen saturation value at temperature  $T_w$ , (see oxygen saturation tables in Standard Methods or many textbooks).

P = ratio of barometric pressure at the pond site to barometric pressure at sea level. (assume 1.0 for an elevation of 100 m)

Eq. 14 can be used to estimate the water temperature in the pond during the summer months which will be the critical period for design. The use of the partial-mix design procedure is illustrated by the following example.

**Example.** Design a four cell partial-mix aerated pond for the following environmental conditions and wastewater characteristics.  $Q = 151,400 \, \mathrm{m}^3/\mathrm{day}$ ,  $C_o = 300 \, \mathrm{mg/l}$ ,  $C_e$  from fourth cell = 30 mg/l,  $k_{20} = 0.276 \, \mathrm{day}^{-1}$ , winter air temperature =  $10^{\circ}\mathrm{C}$ , summer air temperature =  $40^{\circ}\mathrm{C}$ , influent water temperature =  $15^{\circ}\mathrm{C}$ , elevation =  $100 \, \mathrm{m}$ , maintain a minimum DO concentration of 2 mg/l in all cells, use a pond depth of 4 m.

# Solution

1. Assume a winter pond water temperature of 10°C and calculate the volume of a cell in the pond system.

$$k = (0.276)(1.036)^{(10-20)} = 0.194 \text{ day}^{-1}$$

$$t = \frac{4}{0.194} \left[ \left( \frac{300}{30} \right)^{1/4} - 1 \right] = 16.0 \ days$$

$$t_1 = t_2 = t_3 = t_4 = \frac{16.0}{4} = 4.0 \ days$$

 $V_1 = (4.0)(151,400 \text{ m}^3/\text{day}) = 605,600 \text{ m}^3$ , use three trains,  $V_{11} = 202,867 \text{ m}^3$ 

2. Assuming that the pond cells have a length to width ratio of 4:1, calculate the dimensions of the cell using Eq. 15.

$$V\left(\frac{6}{4}\right) = 4W \times W + (4W - 2 \times 3 \times 4)(W - 2 \times 3 \times 4) + 4(4W - 3 \times 4)(W - 3 \times 4)$$

$$1.5V = 24W^2 - 360W + 1152$$

or: 
$$W^2 - 15W = (1.5/24)(V) - 48 = 0.0625(202,867) - 48 = 12,631$$

Solve the quadratic equation by completing the square:

$$W^2 - 15W + 56.25 = 12,631 + 56.25$$
  
 $(W - 7.5)^2 = 12,687.25$   
 $W - 7.5 = 112.6$   
 $W = 120.1 \text{ m}$   
 $L = (120.1)(4) = 480.4 \text{ m}$ 

Surface Area A = 
$$(120.1)(480.4) = 57,696 \text{ m}^2$$

3. Check the pond temperature using the calculated cell area of 57,696 m<sup>2</sup> and the other known characteristics in Eq. 14.

$$T_w = \frac{AfT_a + QT_i}{Af + Q} = \frac{(57,696)(0.5)(10) + (50,467)(15)}{(57,696)(0.5) + 50,467} = 13.2^{\circ}C$$

A temperature of 10°C was assumed, so another iteration is necessary.

4. For the second iteration assume 14°C.

$$k = (0.276)(1.036)^{(14-20)} = 0.223 \text{ days}^{-1}$$

Using Eq. 12, the total detention time for the four cell system is 14.0 days, or 3.5 days/cell.

$$V_1 = (3.5)(151,400) = 529,900 \text{ m}^{3.7}_{11} = 176,633 \text{ m}^3$$
  
 $W^2 - 15W = 0.0625V - 48$ 

$$(W - 7.5)^{2} = 10,992$$

$$W = 112.3 \text{ m}$$

$$L = (112.3)(4) = 449.2 \text{ m}$$

$$A = (112.3)(449.2) = 50,445 \text{ m}^{2}$$

$$T_{w} = \frac{(50,445)(0.5)(14) + (50,467)(15)}{(50,445)(0.5) + 50.467} = 14.7^{\circ}C$$

This is close enough to the assumed value of 14°C; therefore, adopt the detention time and cell dimensions calculated in this iteration. Add a freeboard allowance of 0.6 m. This will increase the cell dimensions at the top of the inside of the dike to 115.9 m by 449.2 m. Using only 2 cells instead of 4 will increase the detention time by about 50 percent and increase the surface area and volume by a factor of about 3. This would be undesirable in cold climates because of the enhanced potential for ice formation and in all locations because of the additional costs for construction.

5. Determine the oxygen requirements for this pond system based on the organic loading in each cell and by using Eq. 16. The maximum oxygen requirements will occur in the summer months.

Use Eq. 14 to estimate pond temperatures

$$T_w = \frac{(50,445)(0.5)(40) + (50,467)(15)}{(50,445)(0.5) + 50,467} = 23.3^{\circ}C$$

At 23°C the tap water oxygen saturation value ( $C_{ss}$ ) is 8.68 mg/l (see tables in Standard Methods or standard text books for other values).

The organic load in the influent wastewater is:

$$(C_o)(Q) = (300 \text{ g/m}^3)(50,467 \text{ m}^3/\text{day})(\text{day}/24 \text{ hr})(\text{kg}/1000\text{g}) = 631 \text{ kg/h}$$
  
 $k_p = 0.276 (1.036)^{23-20} = 0.307$ 

The effluent BOD from the first cell can be calculated with Eq. 10.

$$\frac{C_1}{C_0} = \frac{1}{\left\lceil \frac{k_c t}{1} + 1 \right\rceil^{-1}} = \frac{1}{(0.307)(3.5) + 1} = 0.482$$

$$C1 = 300(0.482) = 145 \text{ mg/l}$$

Therefore the organic loading on the second cell is:

 $145 \text{ mg/l}(50,467 \text{ m}^3/\text{d})(1000 \text{ L/m}^3)(1\text{d}/24 \text{ h})(1 \text{ kg}/1000\text{g})(1 \text{ g}/1000\text{mg}) = 305 \text{ kg/h}$  Similarly,

BOD in cell 2 effluent = 70 mg/l

Organic loading on cell 3 = 147 kg/hr

BOD in cell 3 effluent = 34 mg/l

Organic loading on cell 4 = 71 kg/hr

The oxygen demand is assumed to be 1.5 times the organic loading, hence

$$N_{a1} = (1.5)(631 \text{ kg/hr}) = 947 \text{ kg/h}$$

Similarly,  $N_{a2}=458$  kg/hr,  $N_{a3}=221$  kg/hr,  $N_{a4}=107$  kg/hr. Use Eq. 16 to calculate equivalent oxygen transfer.

$$N = \frac{N_a}{\alpha \left[ \frac{C_{SW} - C_L}{C_S} \right] 1.025^{T_w - 20}}$$

 $C_{sw} = (\beta)(C_{ss})(P) = (0.9)(8.68 \text{ mg/l})(1.0) = 7.81 \text{ mg/l}$ 

$$N_1 = \frac{947}{0.9 \left[\frac{7.81 - 2.0}{9.17}\right] (1.025)^{23-20}} = 1542kg/h \text{ of } O_2$$

Similarly,

$$N_2 = 746 \text{ kg/h of } O_2$$

$$N_3 = 360 \text{ kg /h of } O_2$$

$$N_4 = 174 \text{ kg/h of } O_2$$

6. Evaluate both surface and diffused air aeration equipment. A value of 1.9 kg  $O_2$ /kWh (1.4kg/hp/h) is recommended for estimating power requirements for surface aerators. A value of 2.7 kg  $O_2$ /kWh (2 kg/hp/h) is recommended by the manufacturers of this equipment. The gas transfer rate must be verified for the equipment selected.

The total power for surface aeration is:

Cell 1: 
$$(1542 \text{ kg/h of } O_2)/(1.9 \text{ kg/kWh of } O_2) = 812 \text{ kW } (1,089 \text{ hp})$$

Similarly,

The total power for diffused aeration is:

Cell 1: 
$$(1542 \text{ kg/h of } O_2)/(2.7 \text{ kg/kWh of } O_2) = 571 \text{ kW } (766 \text{ hp})$$

Similarly

Cell 
$$2 = 276 \text{ kW} (370 \text{ hp})$$

Cell 
$$3 = 133 \text{ kW } (178 \text{ hp})$$

Cell 
$$4 = 64 \text{ kW} (86 \text{ hp})$$

These surface or diffused aerator power requirements must be corrected for gearing and blower efficiency. Assuming 90 percent efficiency for both gearing and blowers, the total power for surface aerators in cell 1 would be 812 kW/ 0.9 = 902 kW (1210 hp). The total power needs are about 1,651 kW for the surface aerators and 1,160 kW for the diffused aerators. These are approximate values and are used for the preliminary selection of aeration

equipment. The actual power requirement using surface aeration will be determined by using the zone of complete oxygen dispersion reported by the equipment manufacturers along with the calculated power estimates. The distribution of the two types of aeration equipment are illustrated in Figs. 2-2 and 2-3. Surface aeration equipment is subjected to potential icing problems in cold climates, and use of the fine bubble perforated tubing requires that a diligent maintenance program be established. A number of communities have experienced clogging of the perforations, particularly in hard water areas. Corrective action requires purging with HCl gas.

The final element recommended in this partial-mix aerated pond system is a settling cell with a 2-day detention time.

## **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. 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 (Metcalf and Eddy, 1991). 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 (Metcalf and Eddy, 1991; and Benefield and Randall, 1980). 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 (Malina et al., 1972). Temperature effects are incorporated into the BOD removal equations. Solids removal will be accomplished by installing a

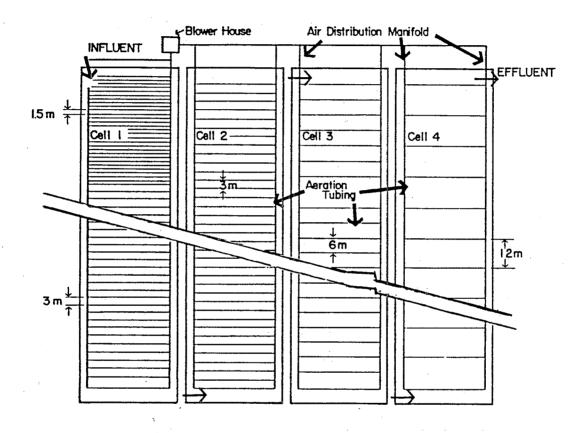


FIGURE 2-2 LAYOUT OF AERATION SYSTEM FOR PARTIAL MIX DIFFUSED AIR AERATED POND SYSTEM.

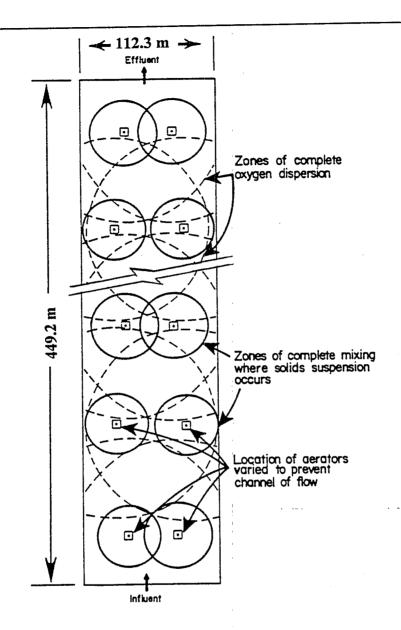


FIGURE 2-3 LAYOUT OF SURFACE AERATORS IN FIRST CELL OF PARTIAL MIX SYSTEM.

settling pond with a two day detention time. If a higher quality effluent is required, the solids removal devices described herein should be evaluated and one selected to produce an acceptable effluent quality.

# Complete Mix Model

The complete mix model using first order kinetics and operating in a series with n equal volume ponds is the same as Equation 10 used for the partial mix pond system. The differences in using Equation 10 with complete mix systems is the change in  $k_c$  (becomes 2.0 per day) and  $\theta$  in Equation 13 (becomes 1.085) .

$$\frac{C_n}{C_o} = \frac{1}{[1 + (kt/n)]^n} \tag{10}$$

where  $C_n = effluent BOD concentration in cell n, mg/l$ 

C<sub>o</sub> = influent BOD concentration, mg/l

 $k = first order reaction rate constant, days^{-1}$ 

= 2.0 day 1 at 20°C (assumed to be constant in all cells).

t = total hydraulic residence time in pond system, days

n = number of cells in the series.

The design example shown for a partial mix aerated lagoon can be followed to design a complete mix system by substituting the values of  $k_{\rm c}$  and  $\theta$  mentioned above.

#### CONTROLLED DISCHARGE PONDS

No rational or empirical design model exists specifically for the design of controlled discharge wastewater ponds as utilized in the northern U.S. and Canada. However, the facultative pond design models may also be applied to the design of controlled discharge ponds provided allowance is made for the required larger storage volumes. The plug-flow model for facultative ponds can be applied to the controlled discharge type if the hydraulic residence time is less than 120 days. A study of 49 controlled discharge ponds in Michigan indicated that residence times were 120 days or greater and discharge periods ranged from 5 days to 30 days for each occurrence. Ponds of this type have been

successfully operated in the north central United States using the following criteria:

- Overall organic loading: 22-28 kg BOD/ha-day (20-25 lb BOD/ac-day).
- Liquid depth: not more than 2 m (6 ft) in first cell, not more than 2.5 m (8 ft)in subsequent cells.
- Hydraulic detention: at least 6 months 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 interconnected piping 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 the discharge period, and that the receiving watercourses can accommodate the discharge rate from the pond. The design must also develop 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 discharge period.

- The following steps are usually taken for discharge from all systems:
- Isolate the cell to be discharged, usually the final one in series, by valving off the inlet line from the preceding cell.
- Analyze cell contents for parameters of concern in the discharge permit.
- Plan activities to spend full time on control of discharge during the entire discharge period.
- Monitor conditions in receiving stream and request approval from regulatory agency for discharge.
- Commence discharge when approval is received and continue as long as weather is favorable, and dissolved oxygen levels and turbidity are below limits. Typically the last two cells in the series are sequentially

isolated and drawn down. Then discharge is interrupted for a week or more while raw wastewater is diverted to one of the cells which has been drawn down. The purpose here is to isolate the first cell prior to its discharge. When the first cell is drawn down to about 60 cm (24 in) depth, the usual internal series flow pattern, without discharge is resumed.

• During the discharge periods, samples are taken at least three times daily near the discharge pipe for immediate dissolved oxygen analysis. Additional testing may be required for SS and other parameters.

Experience with the operational concept listed above is limited to northern states with seasonal and climatic constraints on performance. A continuous ice cover on a facultative pond will lower performance and little better than primary effluent will result if discharge is permitted during such periods. Stringent limits on SS may also limit discharge during the seasonal algal bloom periods. The concept will be guite effective for BOD removal in any location. The process will also work with a more frequent than semi-annual discharge cycle, depending on receiving water conditions and requirements.

The hydrograph-controlled release (HCR) pond is a variation of this concept, which was developed for use in the southern United States, but can be effectively used in most areas of the country. In this case the discharge periods are controlled by a gauging station in the receiving stream and are allowed to occur during high flow periods. During low flow periods the effluent is stored in the HCR pond. The process design uses conventional facultative or aerated ponds for the basic treatment, followed by the HCR cell for storage/discharge. No treatment allowances are made during design for the residence time in the HCR cell; its sole function is storage. Depending on stream flow conditions, the storage needs may range from 30 to 120 days. The design maximum water level in the HCR cell is typically about 2.4 m (8 ft) with the minimum water level at 0.6 m (2 ft). Other physical elements are similar to conventional pond systems. The major advantage for HCR systems is the possibility of utilizing lower discharge standards during high flow conditions as compared to a system designed for very stringent low flow requirements and then operated in that mode on a continuous basis.

#### COMPLETE RETENTION PONDS

In areas of the world 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 if low cost land is available. The pond 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 available elsewhere (USEPA, 1983).

#### 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 described above, and the predicted effluent quality from this unit would be the influent quality for the facultative polishing pond. Further details on combined pond systems can be found elsewhere (Boulier and Atchinson, 1980; Gloyna, 1976; and Rich, 1982). Oswald (1970) has developed the Advanced Integrated Pond system (AIP) which consists of four basic types of ponds in series (USDOE, 1993). A facultative pond with a "digester pit" is followed by a high-rate pond, a settling pond, and a maturation pond(s). Systems have been built in several locations in California and several countries throughout the world. The best know facility of this type is located in St. Helena, California.

#### ANAEROBIC PONDS

There is no agreement on the best approach to the design of anaerobic stabilization ponds. Systems are designed on the basis of surface loading rate, volumetric loading rate and hydraulic detention time. Although done frequently, design on the basis of surface loading rate probably is inaccurate. Proper design should be based on the volumetric loading rate, temperature of the liquid, and the hydraulic detention time.

In climates where the temperature exceeds 22°C, the following design criteria should yield a  $BOD_5$  removal of 50 % or better (WHO, 1987).

- Volumetric loading up to 300 g BOD<sub>5</sub>/m<sup>3</sup>-day
- Hydraulic detention time of approximately 5 days
- Depth between 2.5 and 5 meters

In cold climates, detention times as great as 50 days and volumetric loading rate as low as 40 g  $BOD_5/m^3$ -day may be required to achieve 50% reduction in  $BOD_5$ . The relationship between temperature, detention time, and BOD reduction is shown in Tables 2-4 and 2-5.

Table 2-4
Five-day BOD Reduction as a Function of Detention Time
for Temperatures Greater Than Twenty Degrees Celcius (WHO, 1987)

Detention Time	BOD₅ Reduction
(days)	(%)
1	50
2.5	60
5	70

Table 2-5
Five-day BOD Reduction as a Function of Detention Time, and
Temperature (WHO, 1987)

Temperature (°C)	Detention Time (days)	BOD Reduction (%)	
10	5	0-10	
10-15	4-5	30-40	
15-20	2-3	40-50	
20-25	1-2	40-60	•
25-30	1-2	60-80	

#### PATHOGEN REMOVAL

Bacteria, parasite, and virus removal is very effective in multiple-cell wastewater stabilization ponds with suitable detention times (Reed, 1985). A minimum of three cells is recommended. It is expected that the normal detention time provided for BOD removal in most pond systems may be sufficient to satisfy most regulatory requirements for bacteria and virus removal without additional disinfection; however, a 20-day minimum detention time is suggested.

#### SUSPENDED SOLIDS REMOVAL

The occasional high concentration of SS, which can exceed 100 mg/l, in the effluent is the major disadvantage of pond systems. The solids are primarily composed of algae and other pond detritus, not wastewater solids. These high concentrations are usually limited to 2 to 4 months during the year. Several options, discussed in the sections to follow, are available for improving system performance. Further details can be found in Middlebrooks et al. (1982); USEPA (1973); USEPA (1974).

# Intermittent sand filtration

Intermittent sand filtration is capable of polishing pond effluents at relatively low cost. It is similar to the practice of slow sand filtration in potable water treatment or the slow sand filtration of raw sewage which was practiced during the early 1900s. Intermittent sand filtration of pond effluents is the application of pond effluent on a periodic or intermittent basis to a sand filter bed. As the wastewater passes through the bed, suspended solids and other organic matter are removed through a combination of physical straining and biological degradation processes. The particulate matter collects in the top 5 to 8 cm (2 to 3 in) of the filter bed, and this accumulation eventually clogs the surface and prevents effective infiltration of additional effluent. When this happens, the bed is taken out of service, the top layer of clogged sand removed, and the unit is put back into service. The removed sand can be washed and reused or discarded.

The effluent quality is almost totally a function of the sand gradation used. When BOD and SS below 30 mg/l will satisfy requirements, a single-stage filter with medium sand will produce a reasonable filter run. If better effluent quality is necessary, a two-stage filtration system should be used, with finer sand in the second stage.

Typical hydraulic loading rates on a single stage filter range from 0.37 to 0.56 m<sup>3</sup>/m<sup>2</sup>-day (0.4 to 0.6 million gallons/ac-day). If the SS in the influent to the filter will routinely exceed 50 mg/l, the hydraulic loading rate should be reduced to 0.19 to 0.37 m<sup>3</sup>/m<sup>2</sup>-day (0.2 to 0.4 million gallons/ac-day) to increase the filter run. In cold weather locations, the lower end of the range is recommended during winter operations to avoid the possible need for bed cleaning during the winter months. The total filter area required for a singlestage operation is obtained by dividing the anticipated influent flow rate by the hydraulic loading rate selected for the system. One spare filter unit should be included to permit continuous operation since the cleaning operation may require several days. An alternate approach is to provide temporary storage in the pond units. Three filter beds are the preferred arrangement to permit maximum flexibility. In small systems that depend on manual cleaning, the individual bed should not be bigger than about 90 m<sup>2</sup> (1000 ft<sup>2</sup>). Larger systems with mechanical cleaning equipment might have individual filter beds up to 5000 m<sup>2</sup> (55,000 ft<sup>2</sup>) in area.

Selected sand is usually used as the filter media. These are generally described by their effective size (e.s.) and uniformity coefficient (u). The e.s. is the 10 percentile size, i.e., only 10 percent of the filter sand, by weight, is smaller than that size. The uniformity coefficient is the ratio of the 60 percentile size to the 10 percentile size. The sand for single-stage filters should have an e.s. ranging from 0.20 to 0.30 mm and a u of less than 7.0, with less than 1 percent of the sand smaller than 0.1 mm. The u value has little effect on performance, and values ranging from 1.5 to 7.0 are acceptable. In the general case clean, pit-run concrete sand is suitable for use in intermittent sand filters providing the e.s., u, and minimum sand size are suitable.

The design depth of sand in the bed should be at least 45 cm (18 in) plus a sufficient depth for at least 1 year of cleaning cycles. A single cleaning operation may remove 2.5 to 5 cm (1 to 2 in) of sand. A 30-day filter run would then require an additional 30 cm (12 in) of sand. In the typical case an initial bed depth of about 90 cm (36 in) of sand is usually provided. A graded gravel layer 30 to 45 cm (12 to 18 in) separates the sand layer from the underdrains. The bottom layer is graded so that its e.s. is four times as great as the openings in the underdrain piping. The successive layers of gravel are progressively finer to prevent intrusion of sand. An alternative is to use gravel around the underdrain piping and then a permeable geotextile membrane to separate the sand from the gravel. Further details on design and performance of these systems can be found in Middlebrooks et al. (1982), Russell (1980), and USEPA (1983).

## Microstrainers

Early experiments with microstrainers to remove algae from pond effluents were largely unsuccessful. This was generally attributed to the algae being smaller than the mesh size of the microstrainers tested. A polyester fabric with a 1-µm mesh size has since been developed, and it appears that microstrainers equipped with this fabric are capable of producing an effluent with BOD and SS concentrations less than 30 mg/l.

Microscreen manufacturers are promoting the use of the 1- $\mu$ m screen with the return of the filtered algae to the pond. Short-term experience indicates that the return of filtered algae does not cause problems; however, the potential exists for the filtered material to accumulate and eventually cause overloading of the screen. The effects of solids recycle through the pond system should be monitored in newly constructed microscreen systems. The first full-scale microstrainer application to pond effluent, a 7,200 m<sup>3</sup>/day (1.9 million gallon/day) unit was placed in operation in Camden, South Carolina in December 1981 (Harrelson and Cravens, 1982). Typical design criteria include surface loading rates of 90 to 120 m<sup>3</sup>/m<sup>2</sup>-day (1.5 to 2.0 gpm/ft<sup>2</sup>) and head losses up to 60 cm (2 ft). Other process variables include drum speed, backwash rate and pressure; these are normally determined on the basis of influent quality and effluent expectations. The service life of the screen is reported to be about 1 1/2 years, which is considerably less than the manufacturer's prediction of 5 years. Difficulty with screen binding and short run times was experienced with the Camden system. Before designing a microscreen for pond polishing, careful study is recommended.

#### Rock filters

A rock filter operates by allowing pond effluent to travel through a submerged porous rock bed, causing algae to settle out on the rock surfaces as the liquid flows through the void spaces. The accumulated algae are then biologically degraded. Algae removal with rock filters has been studied extensively at Eudora, Kansas; California, Missouri; and Veneta, Oregon (Swanson and Williamson, 1980; and USEPA, 1983). Many rock filters have been installed throughout the United States and the world, and performance has varied (Middlebrooks, 1988).

The principal advantages of the rock filter are its relatively low construction cost and simple operation. Odor problems can occur, and the design life for the filters and the cleaning procedures have not yet been firmly established. However, several units have operated successfully for 10 to 15 years.

# Other solids removal techniques

A detailed discussion of normal granular media filtration, dissolved air flotation, autoflocculation, phase isolation, centrifugation, and coagulation-flocculation is presented in Middlebrooks et al. (1982) and USEPA (1983). These techniques are used infrequently, but the designer should be aware of their potential.

#### **NITROGEN REMOVAL**

The BOD and SS removal capability of pond systems has been reasonably well-documented, and reliable designs are possible; however, the nitrogen removal capability of wastewater ponds is given little consideration in most system designs. Nitrogen removal can be critical in many situations, since ammonia nitrogen in low concentrations can adversely affect some young fish in receiving waters. In addition, nitrogen is often the controlling parameter for design of land treatment systems. Any nitrogen removal in the preliminary pond units can result in a very significant savings in the land area required and therefore the costs for land treatment (Reed et al., 1995).

Nitrogen loss from streams, lakes, impoundments, and wastewater ponds has been observed for many years. Data on nitrogen losses have been insufficient for a comprehensive analysis, and there has been no agreement on the removal mechanisms. Various investigators have suggested: algal uptake, sludge deposition, adsorption by bottom soils, nitrification/denitrification and loss of ammonia as a gas to the atmosphere (volatilization). Several evaluations suggest that a combination of factors may be responsible, with the dominant mechanism under favorable conditions being losses to the atmosphere (Pano and Middlebrooks, 1982; Reed, 1984; USEPA, 1983).

The EPA sponsored comprehensive studies of wastewater pond systems in the late 1970s. These results provided absolute verification that significant nitrogen removal does occur in pond systems. Table 2-6 summarizes the key findings from these studies, which confirm that nitrogen removal is in some way related to pH, detention time, and temperature in the pond system. The pH fluctuates as a result of the algae-carbonate interactions in the pond, so wastewater alkalinity is important. Under ideal conditions, up to 95 percent nitrogen removal can be achieved in wastewater stabilization ponds.

Table 2-6
Data Summary From EPA Pond Studies (USEPA, 1983)

Location	Detention Time (days)	Water Temperature (°C)	pH (median)	Alkalinity (mg/l)	Influent Nitrogen (mg/l)	Removal (%)
Peterborough, NH 3 cells	107	11	7.1	85	17.8	43
Kilmichael, MS 3 cells	214	18.4	8.2	116	35.9	80
Eudora, KS 3 cells	231	14.7	8.4	284	50.8	823
Corinne, UT 1st 3 cells	42	10	9.4	555	14.0	46

# Design models

Data were collected on a frequent schedule from every cell at all of the pond systems listed in Table 2-6 for at least a full annual cycle. This large body of data allowed quantitative analysis with all major variables included, and two design models were independently developed. These have been validated using the same data from sources not used in the model development. The two models are summarized in Tables 2-7 and 2-8; details on development of Model 1 can be found in Reed (1984) and details of Model 2 can be found in Pano and Middlebrooks (1982).

Table 2-7
Design Model Number 1 (Reed, 1984)

$$N_e = N_0 \exp \left\{ -k_t \left[ t + 60.6(pH - 6.6) \right] \right\}$$
 (17)

where  $N_e$  = effluent total nitrogen, mg/l

 $N_o = influent total nitrogen, mg/l$ 

 $k_T$  = temperature dependent, rate constant, days<sup>-1</sup>, pH<sup>-1</sup> =  $k_{20}(\theta)^{T-20}$ 

 $\theta = 1.039$ 

T = water temperature, (use Eq. 13)

$$k_{20} = 0.0064 \text{ day}^{-1}$$

See Reed (1984) or USEPA (1983) for typical pH values or estimate with:

 $pH = 7.3 \exp[0.0005(ALK)]$ 

ALK = expected influent alkalinity, mg/l, (derived from data in Reed, 1984 and USEPA, 1983)

# Table 2-8

Design Model Number 2 (Pano and Middlebrooks, 1982 and USEPA, 1983)

$$N_e = N_0 \left[ \frac{1}{1 + t(0.000576T - 0.00028) \exp[(1.080 - 0.042T)(pH - 6.6)]} \right]$$
(18)

All terms defined in Table 2-7.

Both are first order models, and both depend on pH, temperature, and detention time in the system. Although they both predict the removal of total nitrogen, it is implied in the development of each that volatilization of ammonia is the major pathway for nitrogen removal from wastewater stabilization ponds. Figure 2-4 demonstrates the application of the two models and compares the predicted total nitrogen in the effluent to the actual monthly average values measured at Peterborough, New Hampshire.

Both of these models are written in terms of total nitrogen, and they should not be confused with the still valid equations in Pano and Middlebrooks (1982) and USEPA (1983), which are limited to only the ammonia fraction. Calculations and predictions based on total nitrogen should be even more conservative than those earlier models.

The high-rate ammonia removal by air stripping in advanced wastewater treatment depends on high (>10) chemically adjusted pH. The algal-carbonate interactions in wastewater ponds can elevate the pH to similar levels for brief periods. At other times, at moderate pH levels the rate of nitrogen removal may be low, but the long detention time in the pond compensates.

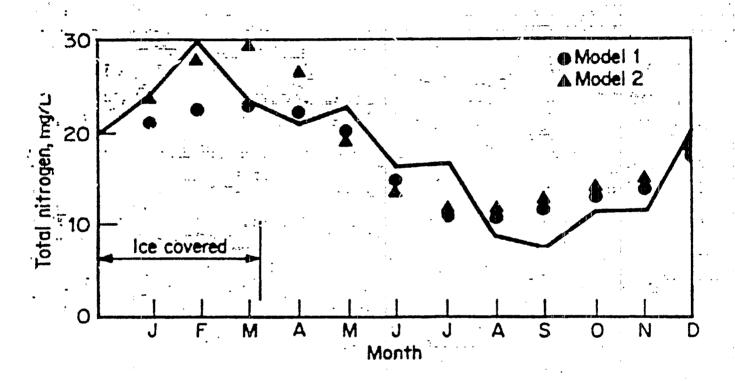


FIGURE 2-4 PREDICTED VERSUS ACTUAL EFFLUENT NITROGEN, PETERBOROUGH, N.H.

# Application

These models should be useful for new or existing wastewater ponds when nitrogen removal and/or ammonia conversion is required. The design of new systems would typically base detention time on the BOD removal requirements. The nitrogen removal that will occur during that time can then be calculated with either model. It is prudent to assume that the remaining nitrogen in the effluent will be ammonia, and to then design any further removal /conversion for that amount. If additional land area is available, a final step can be a comparison of the costs of providing additional detention time in the pond for nitrogen removal with the costs for other removal alternatives.

#### PHOSPHORUS REMOVAL

The need for phosphorus removal can occur when eutrophication is an issue. In general, phosphorus removal is not often required for wastewaters receiving stabilization pond treatment, but there are a number of exceptions for systems in the north central United States and Canada (Reed et al., 1995).

#### Batch chemical treatment

In order to meet a phosphorus requirement of 1 mg/l for discharge to the Great Lakes an approach using in-pond chemical treatment in controlled discharge ponds was developed in Canada. Alum, ferric chloride and lime were all tested by using a motor boat for distribution and mixing of the chemical. A typical alum dosage might be 150 mg/l, and this should produce an effluent from the controlled discharge pond that contains less than 1 mg/l of phosphorus and less than 20 mg/l BOD and SS. The sludge build-up from the additional chemicals is insignificant and would allow years of operation before requiring cleaning. The costs for this method were very reasonable and much less than conventional phosphorus removal methods (Graham and Hunsinger, undated).

# Continuous overflow chemical treatment

Studies of in-pond precipitation of phosphorus, BOD, and SS were conducted over a 2 year period in Ontario, Canada (Graham and Hunsinger, 1977). 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 2-year period. Hydrated lime, at dosages up to 400 mg/l, was not effective in consistently reducing phosphorus below 1 mg/l (1 to 3 mg/l was

achieved) and produced no BOD reduction while slightly increasing the SS concentration. Ferric chloride reduced effluent BOD from 17 to 11 mg/l and SS from 28 to 21 mg/l; alum produced no BOD reduction and a slight SS reduction (from 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, Maryland, was modified to operate as two three-cell units in parallel (Engel and Schwing, 1980). One system was used as a control and alum 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 to 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, the effluent total phosphorus concentration averaged 2.5 mg/l, with the control units 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.

#### PHYSICAL DESIGN AND CONSTRUCTION

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, will determine the actual efficiency achieved (Middlebrooks et al., 1982; USEPA, 1983).

Length to width ratios are determined according to the design model used. Complete-mix ponds should have a length to width ratio of approximately 1:1; whereas, plug-flow ponds require a ratio of 3:1 or greater.

The danger of groundwater contamination may impose seepage restrictions, necessitating lining or sealing the pond. Reuse of pond effluents 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 attacks, etc. Transfer structure

placement and size affect flow patterns within the pond and determine operational capabilities in controlling the water level and discharge rate.

#### 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.

Erosion protection is necessary 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 extend from at least 0.3 m (1 ft) below the minimum water level to at least 0.3 m (1 ft) above the maximum water surface. Asphalt, concrete, fabric, low grasses, and riprap have all been used to provide protection form wave action. The use of rip rap, however, can make weed and rodent control more difficult. In some cases when fabric liners are used, a covering of rip rap is also used to protect the plastic materials from damaging ultraviolet radiation from the sun. Rodent control can be achieved with earthen dikes by periodically changing the water levels to flood the burrows. The selection of proper soils and compaction during construction can render an earthen dike essentially impermeable. Seepage collars should be provided around any pipe penetrating the dike; these collars should extend a minimum of 0.6 m (2 ft) from the pipe.

## Pond sealing

The primary motive for sealing ponds is to prevent seepage, which can pollute groundwaters and affect treatment performance by causing fluctuations in the water depth. Sealing methods can be grouped in three categories:

- Synthetic and rubber liners
- Compacted earth or soil cement liners
- Natural and chemical treatment liners

Within each category also exists a wide variety of application characteristics. Choosing the appropriate lining for a specific site is a critical factor in pond design and seepage control. Seepage rates range from 0.003 cm/day (0.001 in/day) for synthetic membranes to about 10 cm/day (4 in/day) for soil cement liners (USEPA, 1983). Detailed information is available from

manufacturers and in other publications (Kays, 1986; and Middlebrooks, et al., 1978).

# Pond hydraulics

In the past, the majority of ponds were designed to receive influent wastewater through a single pipe, usually located toward the center of the first cell in the system. Hydraulic and performance studies <sup>7,9,19,20</sup> have shown that the center discharge point is not the most efficient method of introducing wastewater to a pond (Finney and Middlebrooks, 1980; George, 1973; Mangelson, 1971, 1972). Multiple inlet arrangements are preferred even in small ponds [<0.5 ha (<1.2 ac)]. The inlet points should be as far apart as possible and the water should preferably be introduced by means of a long diffuser or multiple inlet structure. The inlets and outlets should be placed so that flow through the pond is uniform between successive inlets and outlets.

Single inlets can be used successfully if the inlet is located the greatest distance possible from the outlet structure and is baffled, or the flow is otherwise 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 to reduce the potential impact of algae and other surface detritus on effluent quality.

Analysis of performance data from selected aerated and facultative ponds indicates that four cells in series are desirable to give the best BOD and fecal coliform removals for ponds designed as plug-flow systems. Good performance can also be obtained with a smaller number of cells if baffles or dikes are used to optimize the hydraulic characteristics of the system.

Better treatment is obtained when the flow is guided more carefully through the pond. In addition to treatment efficiency, economics and aesthetics play an important role in deciding whether or not baffling is desirable. In general, the more baffling that is used, the better the flow control and treatment efficiency. The lateral spacing and length of the baffle should be specified so that the cross-sectional area of flow is as close to a constant as possible.

Wind generates a circulatory flow in bodies of water. To minimize short circuiting due to wind, the pond inlet-outlet axis should be aligned perpendicular to the prevailing wind direction if possible. If this is not possible, baffling can be used to control to some extent the wind-induced circulation. In a constant-depth pond the surface current will be in the direction of the wind, and the return flow will be in the upwind direction along the bottom.

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 is that the effective treatment 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.

# STORAGE PONDS FOR LAND TREATMENT SYSTEMS

Seasonal effluent storage ponds are sometimes required for land treatment systems. Storage is necessary for all nonoperational periods at the land treatment system and is desirable for flow equalization and emergency system back-up. Nonoperating periods may be due to climate, planting or harvesting and other maintenance operations. The design storage volume is determined from a calculated water balance during design (Reed et al., 1995).

The storage pond may follow other conventional treatment units or may be the final cell in a stabilization pond system. The storage cell is usually deeper than typical treatment pond cells and can range from 3 to 6 m (9 to 18 ft) in depth. Credit should be taken during design for the additional treatment which will occur in this storage pond, using the methods presented herein. Calculation of nitrogen removal using either Eq. 17 or 18 is particularly important. Nitrogen is often the limiting design factor for land treatment systems, directly affecting the land area required for treatment. Any nitrogen removal in the storage pond will reduce the final treatment area and the costs. Similarly, the pathogen removal in the pond can often satisfy requirements without further disinfection.

The operation of the storage pond will depend on the type of land treatment system in use. Storage is usually only provided for emergencies in rapid infiltration systems so the pond is drained as soon as it is possible to do so. Since overland flow systems are not very effective for algae removal, storage ponds for these systems are bypassed during algal bloom periods, and the ponds drawn down when algae concentrations are low. Algae are not a concern for slow rate land treatment, so the storage pond may stay on line continuously. This is necessary if nitrogen or pathogen removal is expected in the storage cell. In this case, treated wastewater flow into the cell should continue on a year-round basis, and the withdrawals should be scheduled for attainment of the specified water depth at the end of the operating season for the land treatment component.

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# CHAPTER 3

MEXICALI, BAJA CALIFORNIA NORTE (BCN) MEXICO WASTEWATER TREATMENT SYSTEM

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#### **CHAPTER 3**

# MEXICALI, BAJA CALIFORNIA NORTE (BCN), MEXICO WASTEWATER TREATMENT SYSTEM

Mexicali, Mexico is located in the Northwest part of the state of Baja, California, and the New River flows through the city. Effluent from the two wastewater stabilization pond systems in Mexicali discharge into the New River as well as the effluent from industrial operations and agricultural drains.

The sewer system is divided into two sections, Mexicali I, the older more established section of the city, and Mexicali II which is still developing in conjunction with industrial operations. The current population of the city is 539,000 inhabitants, and the population is expected to grow to 941,000 by the year 2015. Wastewater flow rates are expected to be 1,645 liters per second (37.6 MGD) in 2015. Parts of Mexicali II currently are not sewered, and when the current flow rate into the Mexicali I pond system is considered, the 2015 design flow rate is currently being approached and likely will increase beyond the projections.

Wastewater from Mexicali II is partially treated in the Gonzalez Ortega pond system which is heavily overloaded. The Mexicali BCN Lagoon System I (MLSI) receives wastewater from the Zaragoza neighborhood, and the system is less stressed but still overloaded.

## SYSTEM DESCRIPTION

A plan view of the Mexicali BCN I lagoon system is shown in Figure 2-1, and a simplified flow diagram of the system is shown in Figure 2-2. The system consists of three anaerobic ponds (designed as aerated ponds) followed by two trains of facultative ponds. Train A consists of six facultative ponds receiving one-half of the combined flow from the three primary (anaerobic) ponds, and train B has four facultative ponds receiving the remaining half of the flow from the three anaerobic ponds. All three primary ponds have equal volumes and surface areas. Although shaped differently, all ten of the facultative ponds have the same volume and surface area. The sizes of the primary and facultative ponds are presented in Figure 2-2. The volume of the cells, hydraulic detention time in each cell, and the total detention time for the system are shown in Table 2-1. These data were used to develop the relationships presented in the following sections.

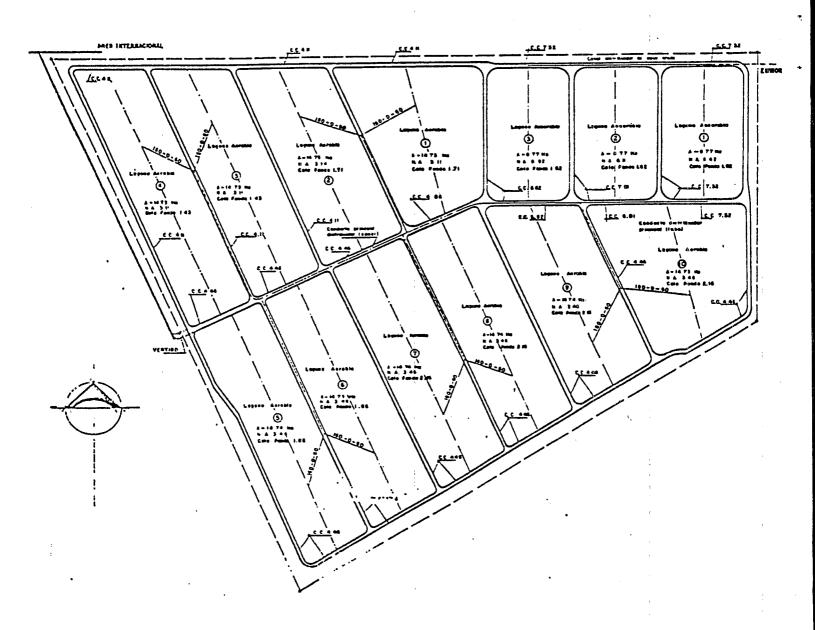


FIGURE 3-1 PLAN VIEW OF MEXICALI, MEXICO WASTEWATER STABILIZATION POND SYSTEM (MEXICALI I)

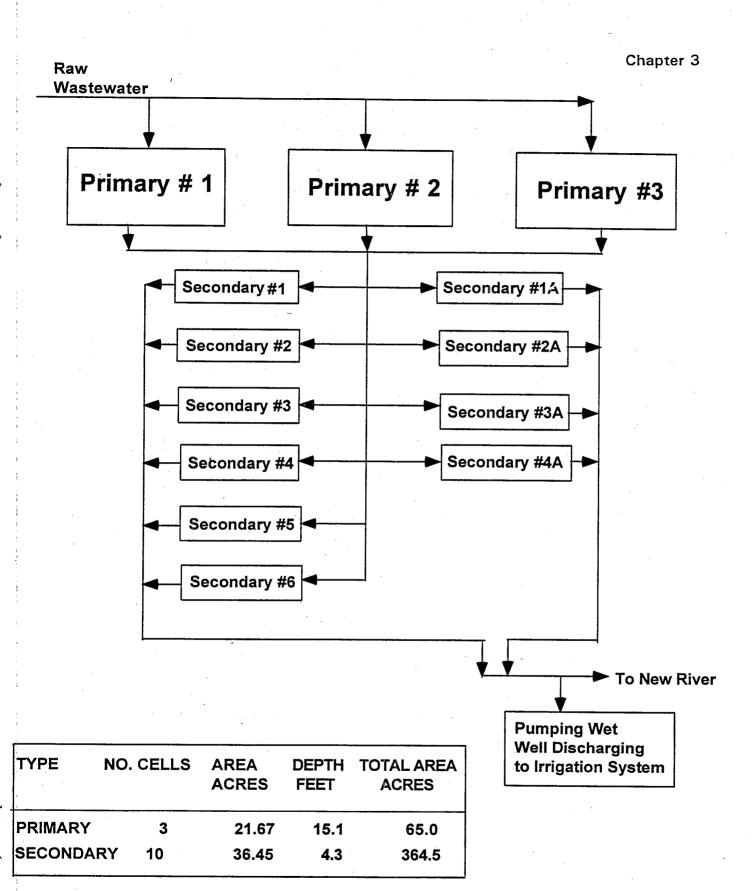


FIGURE 3-2 SCHEMATIC OF MEXICALI BCN, MEXICO WASTEWATER STABILIZATION LAGOON SYSTEM

TABLE 3-1 Mexicali BCN, Mexico Lagoon System

Total Hyd. Detention Time, days	8 4 8 4 8 4 8 4 8 4 8 7 7 7 7 8 8 8 8 7 7 7 8 8 8 8	•
Effluent BOD mg/L	2 2 2 2 1 1 1 1 1 2 1 3 3 4 5 3 3 4 3 3 4 3 3 4 3 3 4 3 3 4 3 3 3 4 3 3 3 4 3	3
Hydraulic Detention Time in, days Secondary Cell Train B (4 cells)	22.1. 18.1. 22.1. 22.1. 23.3. 24.1. 25.3. 26.1. 27.1.	0.01
Hydraulic Detention Time in, days Secondary Cell Train A (6 cells)	31.6 27.1 31.6 31.6 31.6 31.6 31.6 31.6 31.6 31	
Hydraulic Detention Time, days Primary Cell	16.5 16.5 16.5 16.5 16.5 16.5 16.5 16.5	16.4
Volume in One Primary Cell cubic feet	14,253,573 14,253,573	616,662,41
Volume in One Secondary Cell cubic feet	6,827,377 6,827,377	0,041,511
Water Temperature degrees celsius	2 E Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z	77
Air Temperature degrees celsius	5 7 5 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8 8	ΛC
Flow	88888888888888888888888888888888888888	}
Time of day	09:20 10:45 10:05 10:05 10:05 09:35 09:30 09:45 10:10 09:45 10:05 09:30 09:30 09:50 09:50 09:50 09:50 09:50	CV.70
Date	12/03/92 01/14/93 02/03/93 02/11/93 04/06/93 05/10/93 06/09/93 11/01/93 01/05/94 02/02/94 02/02/94 02/02/94 02/02/94 02/02/94 02/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94 06/02/94	04/2/193

#### PERFORMANCE OF SYSTEM

# **Design Limitations**

The MLSI was designed as an aerated pond system, but power was not supplied to the aerators; therefore, the system operates as an anaerobic pond system followed by facultative ponds. Had the ponds been operated as designed, the system would produce an excellent quality effluent in terms of BOD removal. Proper design, location and control of a settling basin would have resulted in a far superior quality effluent overall.

# **Performance of Anaerobic Ponds**

The performance of anaerobic ponds has not been documented as well as it has for aerated or facultative ponds; however, there are enough data available to make a reasonable estimate of the expected performance at various water temperatures. The variation in BOD removal with temperature shown in Figure 3-3 was taken from a World Health Organization document (1987), and the line of best fit was determined to be the following exponential curve.

 $Temperature = e^{0.0165(BOD\ Removal) + 2.146}$ 

or

% BOD Removal=60.61(In Temperature)-130.06

Using the above equations and the observed water temperatures in the Mexicali BCN I pond system, the performance of the anaerobic ponds was estimated and the results are shown in Table 3-2.

#### **Effluent Characteristics**

A summary of the performance data and the effluent characteristics for the Mexicali BCN I lagoon system is shown in Table 3-3. Until January 1994 the pond system appeared to be consistently producing an effluent BOD of less than 30 mg/l. Since then the BOD effluent quality has varied considerably, and the BOD concentration has frequently approached or exceeded 40 mg/l. The variations in BOD, COD, and water temperature are shown in Figure 3-4. Fecal coliform numbers also have tended to increase during this period while the dissolved oxygen in the effluent has declined (Figure 3-5 and Figure 3-6). These changes are probably attributable to the increase in influent flow rate caused by population growth and additional sewer connections.

20

9

20

BOD Removal, %

20

PERFORMANCE OF ANAEROBIC LAGOONS (1) FIGURE 3-3 **Temperature** degrees C 3-6

Temp. = exp[0.0165(BOD Removal) + 2.146] R Squared = 0.904

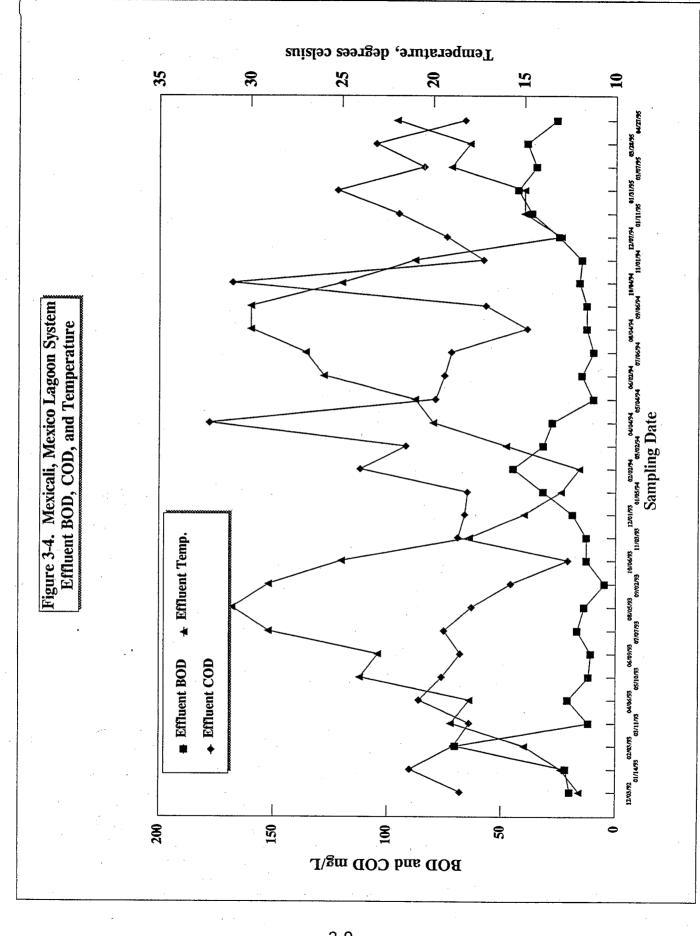
(1) Taken from: World Health Organization Wastewater Stabilization Ponds, Principles of Planning and Practice, WHO Technical Publication 10, Regional Office for the Eastern Mediterranean, Alexandria, 1987.

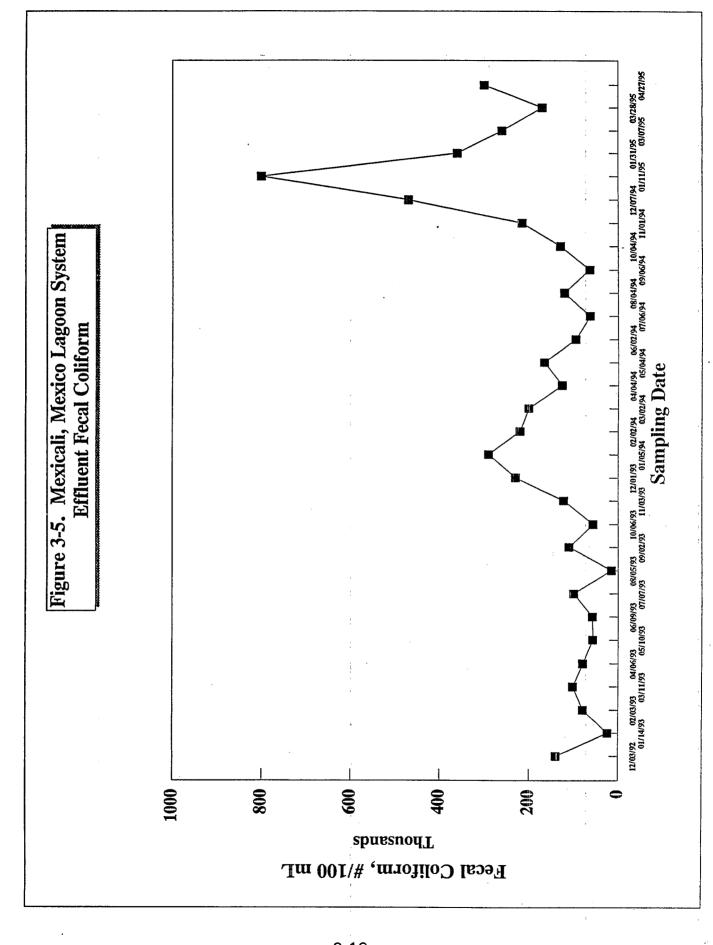
TABLE 3-2 Mexicali BCN, Mexico Lagoon System

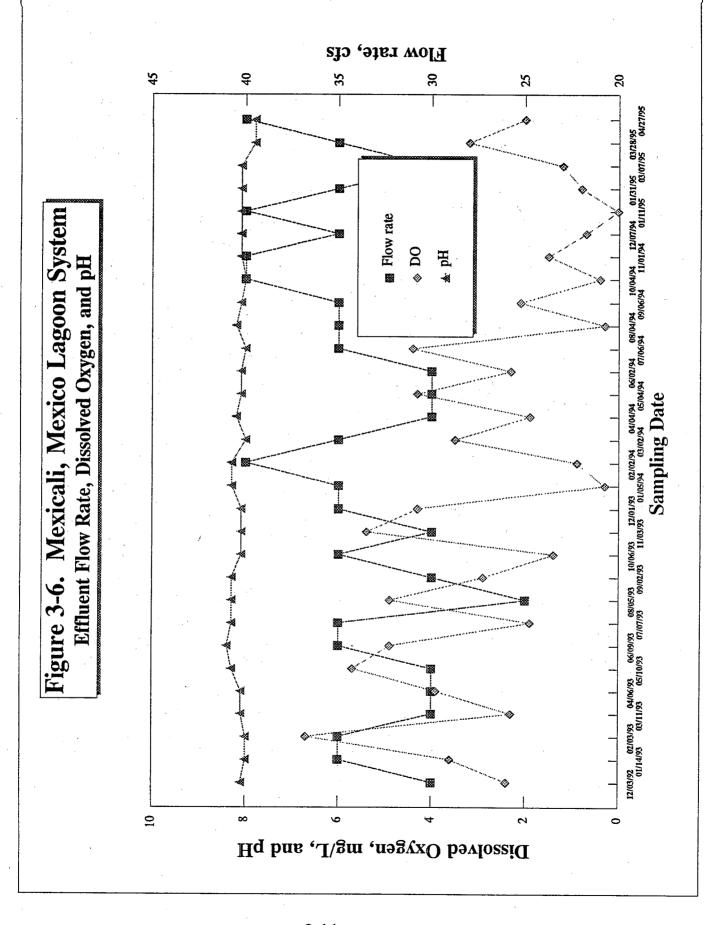
BOD loading rate lbs/ac/day Secondary Cells Train B (4 Cells)	\$2 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5
BOD loading on Secondary Cells lbs/day Train B (4 Cells)	3,214 3,521 2,087 2,220 1,514 1,889 1,226 1,650 2,220 3,111 3,521 4,285 2,927 1,961 1,129 1,129 1,129 1,129 1,129 1,129 2,456 3,521 3,521 2,456 3,530 2,590
Effluent BOD mg/L	20 20 20 21 21 21 21 21 21 21 21 21 21 21 21 21
BOD loading rate, lbs/ac/day Secondary Cells Train A (6 Cells)	\$2 4 5 8 1 4 5 5 5 4 5 5 5 5 5 5 5 5 5 5 5 5 5 5
BOD loading on Secondary Cells lbs/day Train A (6 Cells)	2,143 2,347 2,074 1,391 1,480 1,010 1,259 817 493 700 1,100 1,480 2,074 2,347 2,857 1,951 1,228 879 953 752 752 752 752 1,257 1,637 2,347 2,347 2,347 2,347 2,347 2,347 2,347 1,536 1,726
% BOD Removed in Primary	22 4 8 4 8 6 9 4 8 4 8 6 4 8 7 8 8 2 7 7 5 5 5 5 7 8 8 7 8 8 7 7 8 8 7 8
BOD loading rate, lbs/ac/day Primary Cell	871 871 871 747 747 747 871 871 871 871 871 871 871 871 871 87
BOD loading lbs/day	48,541 56,632 48,541 48,541 48,541 48,541 56,632 56,632 64,722 56,632 56
BOD corr. for Temp.	28 33 4 5 7 5 8 3 7 8 8 3 7 8 8 8 8 8 8 8 8 8 8 8 8 8
Date	12/03/92 01/14/93 02/03/93 02/11/93 04/06/93 05/10/93 05/02/93 11/03/93 11/03/93 11/03/93 11/03/94 02/02/94 02/02/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94 05/04/94

Table 3-3 New River Water Quality - Mexicalli, Mexico Oxidation Pond Effluent

COD mg/L	89 68	<u> </u>	98	9/	89	75	63	46	21	9	99	65	112	92	178	79	75	72	39	57	168	58	74	95	122	84	105	99
BOD mg/L	8 2 8	2 2	21	12	11	17	14	5	13	13	19	32	45	32	78	10	15	10	13	13	16	15	25	37	43	35	33	56
Fecal Coliform no./100 mL	24000	102000	80000	2,1000	28000	100000	15000	110000	2000	122000	230000	290000	220000	200000	125000	165000	92000	63000	120000	64000	130000	215000	470000	800000	360000	260000	170000	300000
Conductance	2500	2040	1700	1850	1700	1600	1900	1100	2050	2120	2200	2450	2500	2210	2050	2000	1970	2000	1950	1550	1870	2120	2300	2150	2250	2050	2050	2050
Hd	8.1	8.1	8.1	8.3	8.4	8.3	8.3	8.3	8.1	8.1	8.1	8.3	8.3	<b>∞</b>	8.2	8.1	8.1	∞	8.2	8.1	∞	8.1	8.1	8.1	8.1	8.1	7.8	7.8
DO mg/L	3.6	0. <i>1</i> 2.3	3.9	5.7	4.9	6.1	4.9	2.9	1.4	5.4	4.3	0.3	6.0	3.5	1.9	4.3	2.3	4.4	0.3	2.1	0.4	1.5	0.7	0.02	8.0	1.2	3.2	2
Water Temperature	12 13	J 61	18	24	23	50	31	29	25	18	15	13	12	16	20	21	26	27	30	30	25	21	13	15	15	61	. 81	22
Air Temperature	10	25 25	29	32	34	38	40	38	33	25	25	. 15	14	21	26	38	38	38	39	44	27	26	23	18	25	28	23	30
Flow CFS	35	3 2	30	30	35	35	22	30	35	30	35	35	40	35	30	30	30	35	35	35	40	40	35	4	. 35	30	35	40
Time of day	09:20 11:30	10:05	11:10	09:35	09:55	10:20	10:30	09:50	06:60	09:40	09:50	10:10	09:15	08:20	10:35	10:20	09:45	10:10	09:25	09:45	10:05	06:30	06:60	09:55	09:50	06:30	09:45	09:02
Date	12/03/92 01/14/93	03/11/93	04/06/93	05/10/93	06/09/93	07/07/93	08/05/93	09/02/93	10/06/93	11/03/93	12/01/93	01/05/94	02/02/94	03/02/94	04/04/94	05/04/94	06/02/94	07/06/94	08/04/94	09/06/94	10/04/94	11/01/94	12/07/94	01/11/95	01/31/95	03/01/95	03/28/95	04/27/95







The variations in flow rate, dissolved oxygen and Ph value are shown in Figure 3-6. The Ph value varied very little over the 30 months, but dissolved oxygen tended to decrease as the pond system matured. This drop in dissolved oxygen was caused by the increasing load being applied to the system.

# **BOD Loadings and Loading Rates**

BOD loadings and loading rates for the primary cells and Train A and B are shown in Table 3-2. BOD loading rates on all components of the system are high and the system is performing as well as can be expected. The loading rate on each of the six facultative ponds is approximately two-thirds of that applied to the four facultative ponds; however, observers of the system report that at most times the effluent from Train B is clearer than that coming from Train A. Based upon the characteristic of the two trains, there does not appear to be any reason to expect the quality to differ unless the detention time in Train B ponds is such that the algae production is less. This does not appear to be the case, because both detention times are adequate to produce considerable algae concentrations.

# **BOD vs Hydraulic Detention Time**

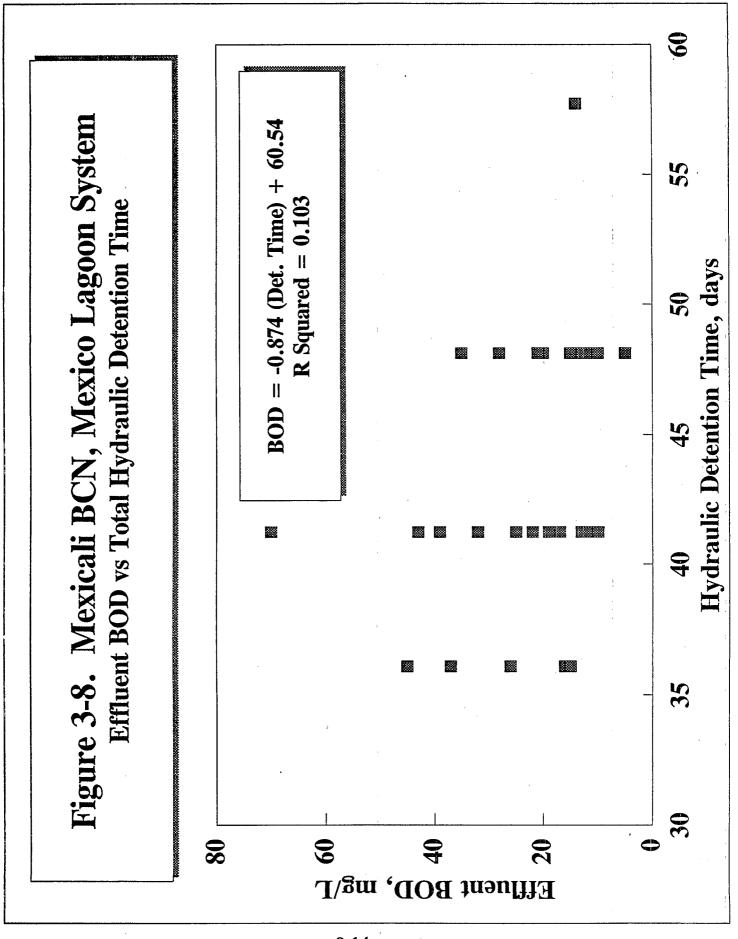
Variations in the effluent BOD concentration, system hydraulic detention time, and the water temperature for the 30 months that performance data were available are shown in Figure 3-7. In general, the effluent BOD improved with an increase in water temperature; however, there was no significant change in BOD with changes in hydraulic detention time.

The relationship between effluent BOD and the system hydraulic detention time is shown in Figure 3-8. There is a downward trend in the effluent BOD as the hydraulic detention time increases; however, the coefficient of determination ( $\mathbb{R}^2$ ) is only 0.103 which is not statistically significant. This result is not surprising because the influent flow rate is only a rough estimate of the actual flow rate BOD versus BOD Loading Rate

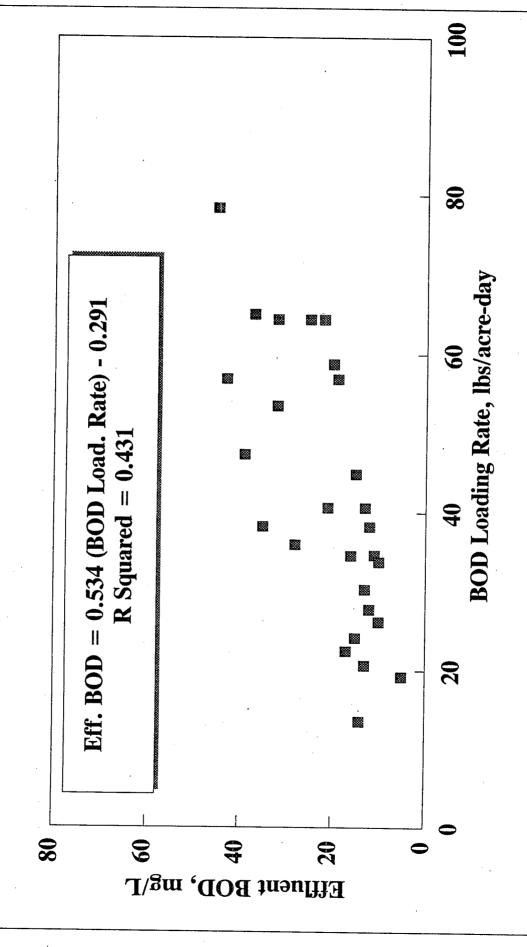
The variations in the effluent BOD concentration with the BOD loading rate on the secondary (facultative) ponds, Train A and B, are shown in Figures 3-9 and 3-10. The relationships show an increase in the effluent BOD concentration as the BOD loading rate increases, and the relationships are statistically significant at the 95 % confidence level.

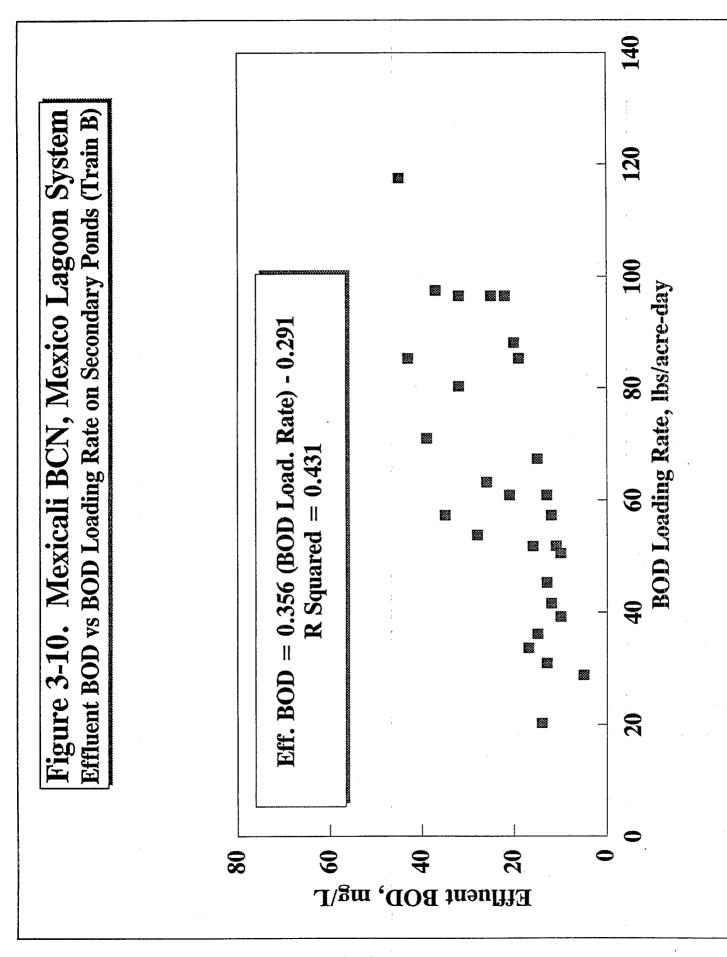
Water Temperature, degrees celsius 35 30 15 10 25 12/03/92 02/03/93 04/06/93 06/09/93 08/05/93 10/06/93 12/01/93 02/02/94 04/04/94 06/02/94 08/04/94 10/04/94 12/07/94 01/31/95 03/28/95 01/14/93 03/11/93 05/10/93 07/07/93 11/03/93 11/03/93 01/05/94 03/02/94 05/04/94 07/06/94 11/01/94 01/11/95 03/07/95 04/27/95 04/27/95 Prine, days Hydraulic Detention Time 8 8 \$ Effluent BOD, mg/L and Detention Time, days

Figure 3-7. Mexicali BCN, Mexico Lagoon System Effluent BOD, Hydr. Det. Time in System, and Water Temperature









# **BOD-Temperature**

When influent and effluent BOD values and water temperature are available, it is possible to develop an equation showing a rate relationship between the BOD and temperature. Unfortunately, influent BOD values were unavailable, but if it is assumed that the influent BOD is relatively constant and that the effluent BOD is related to the water temperature, predictive relationships can be developed that are similar to those developed from the Arrhenius equation. A relationship can be developed from either a simple plot of the natural logarithm of the effluent BOD versus water temperature (Figure 3-11) or a quasi-Arrhenius plot of effluent BOD divided by the BOD at 20 degrees celsius versus the water temperature minus 20 degrees celsius (Figure 3-12). Both relationships yield a statistically significant ( $R^2 = 0.460$ , 95% confidence level) relationship and yield the same answer when using either equation because the numbers are divided by a constant to yield the quasi-Arrhenius plot.

Although these relationships lack precision, the results are as good or superior to those developed from numerous other sets of pond performance data. There are too many variables affecting the performance of pond systems for one to expect the relationship between effluent BOD and temperature to be perfectly correlated (Middlebrooks et al., 1982, USEPA, 1983).

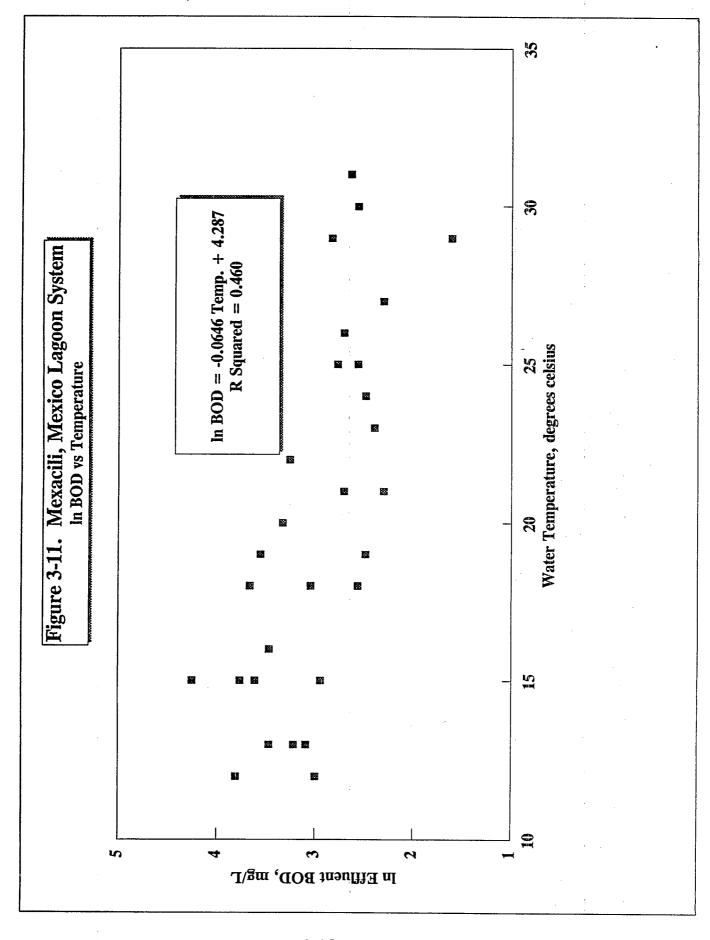
# Temperature Corrected BOD and Detention Time

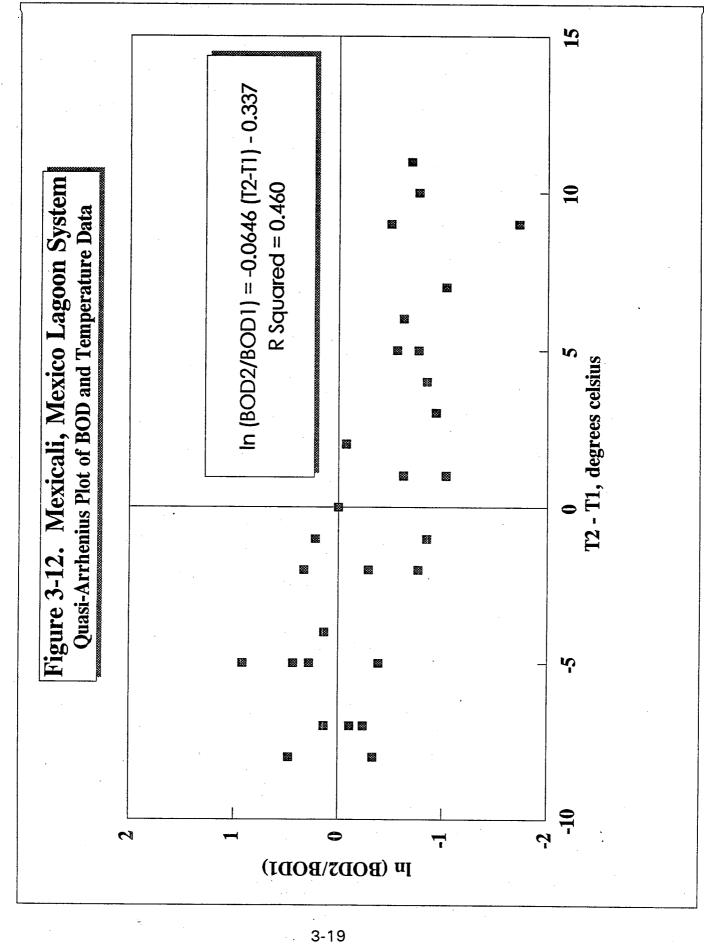
Using the relationships developed between effluent BOD and water temperature, a corrected effluent BOD was calculated and plotted versus the hydraulic detention time (Figure 3-13). As shown in Figure 3-13, the relationship between the two variables was poorer than that observed in Figure 3-8 when the effluent BOD was uncorrected. Again, this is not surprising because similar results have been observed when analyzing pond performance data (Middlebrooks et al., 1982, USEPA, 1983).

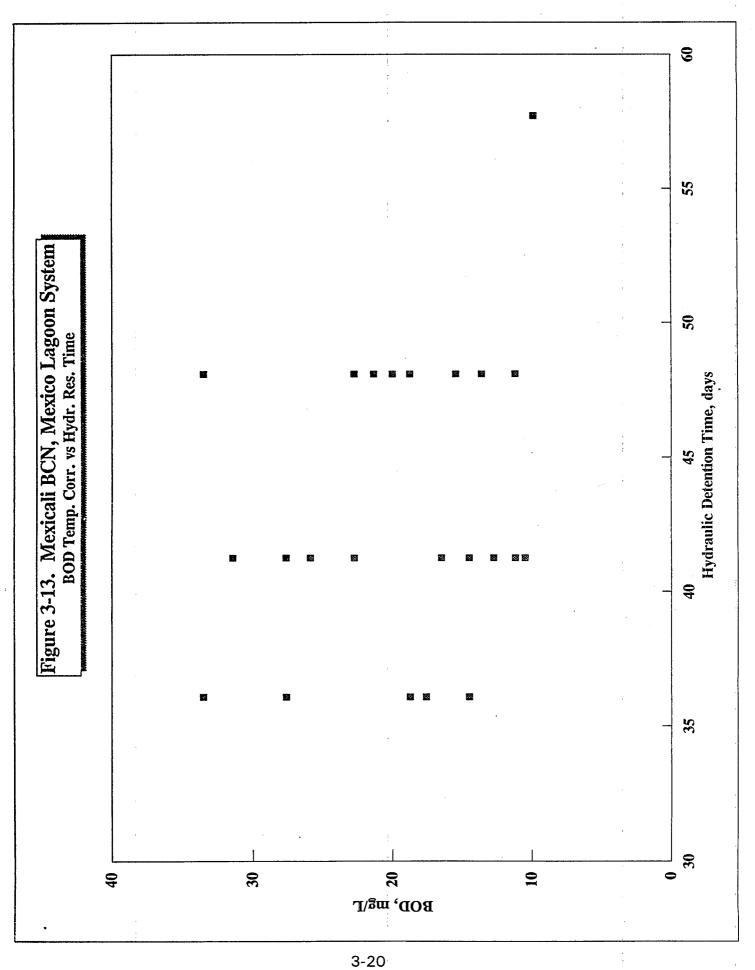
# PREDICTED PERFORMANCE

In an effort to predict the performance of the Mexicali BCN I pond system, a linear correlation was conducted with the effluent BOD as the independent variable and the BOD loading rate on the facultative ponds, the water temperature, and the BOD loading rate on the anaerobic ponds as the dependent variables. The following relationship was obtained when the outlier value of 71 mg/I was excluded.

Eff.BOD=1.732(BOD Load.Rate on Sec.)+3.149(Temp.)-0.054(BOD Load. Rate on Pri.)







A plot of the predicted values versus the measured values is shown in Figure 3-14. Although less precise than desired, the points lie around the 45 degree line, and the equation would be expected to yield a reasonable estimate of the performance of the Mexicali BCN I system in view of the limitations of the flow measurements.

# REDESIGN OF MEXICALI BAJA CALIFORNIA NORTE (BCN) I SYSTEM

Calculations were made using Equations 10 and 11 taken from the body of the report to determine the performance of the Mexicali BCN I pond system when converting the primary and secondary ponds to complete mix or partial mix or a combination of the two types.

$$\frac{C_n}{C_0} = \frac{1}{\left[1 + (kt/n)\right]^n} \tag{10}$$

where

C<sub>n</sub> = effluent BOD concentration in cell n, mg/l

C<sub>o</sub> = influent BOD concentration, mg/l

 $k_p$  and  $k_c$  = first order reaction rate constant, days<sup>-1</sup>

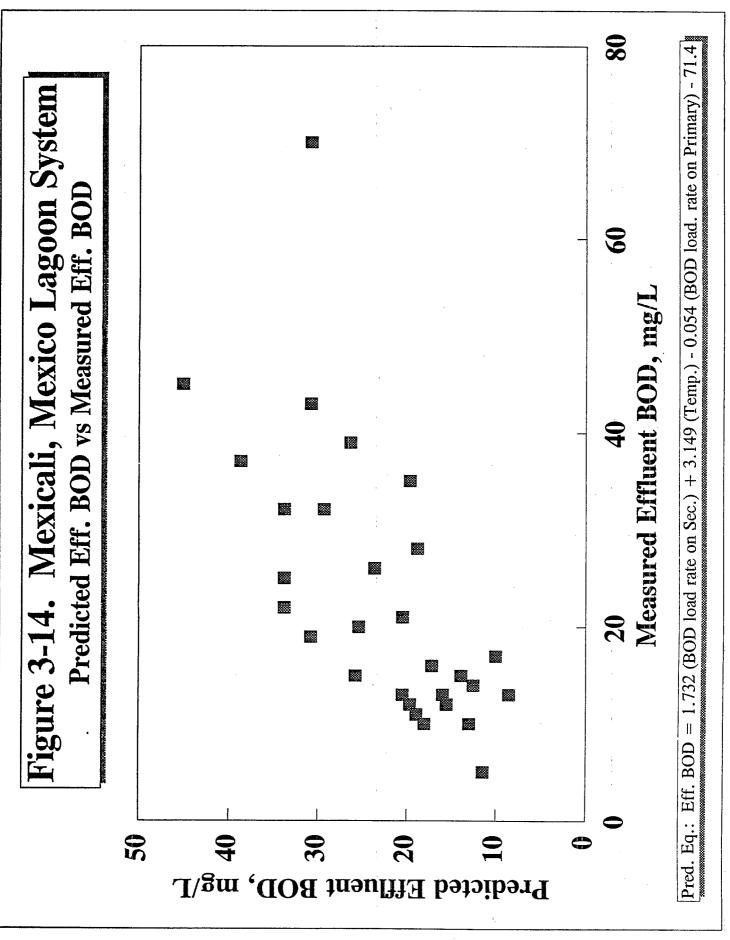
 $k_p = 0.276 \ day^{-1}$  at 20° C (assumed to be constant in all cells)  $k_c = 2.0 \ day^{-1}$  at 20° C (assumed to be constant in all cells.)

t = total hydraulic residence time in pond system, days

n = number of cells in the series.

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

$$\frac{C_n}{C_0} = \left[\frac{I}{I + k_1 t_1}\right] \left(\frac{I}{I + k_2 t_2}\right) \bullet \bullet \bullet \left(\frac{I}{I + k_n t_n}\right) \tag{11}$$



The predicted performances based on the design temperature of 12° C, an influent BOD of 300 mg/l, and the current estimated flow rates for several combinations of complete mix and partial mix pond systems are presented in Table 234. Converting the primary cells (anaerobic) to complete mix ponds would produce an effluent BOD of less than 25 mg/l throughout the 30 months of record. When converting the primary cells to partial mix ponds, an effluent BOD ranging from 59 to 84 mg/l would be produced. Converting the ponds to two partial mix cells of equal volume in series would yield an effluent BOD concentration of 11 to 23 mg/l. The remaining volume would provide an excess of settling time, and it would be necessary to account for this excess volume. The effluent BOD would range between 7 and 20 mg/l when operating the primary cells and the two (Trains A and B) secondary trains as partial mix pond systems. Converting the primary cells to complete mix and the Train A (six cells) cells to partial mix would result in an effluent BOD concentration of two to five mg/l.

Table 3-5 contains the results for the same combinations presented in Table 3-4, but the flow rate has been increased to twice that of the current flow rates. If the objective is to produce an effluent with a BOD concentration of 30 mg/l or less at twice the current flow rate, the only combination that will yield the desired quality is to convert the primary cells to complete mix and the secondary cells to partial mix pond systems.

Increasing the flow rate to 1.5 times the current flow rates for the same conditions described for Table 3-4 and 3-5, the system would be able to produce an effluent satisfying a 30 mg/l BOD standard with the conversion of the primary and secondary cells to partial mix pond systems and the use of a complete mix pond in the primary followed by partial mix cells in the secondary ponds (Table 3-6).

To control the solids concentrations in the effluent, it will be necessary to redesign the effluent structures to provide multiple draw-off structures with multiple draw-off depths. It may be necessary to improve the hydraulics of the system with floating baffles or dikes. It was impossible to determine if the flow was evenly distributed to the three primary cells; therefore, it probably will be necessary to construct new inlet structures as well.

Based upon the standard design equations shown above, it is possible to make relatively simple conversions to the Mexicali BCN I pond system that would result in a good quality effluent at a relatively small cost.

Table 3-4
Predicted Performance of Mexicali, Mexico Lagoon System If Converted to Combinations of Partial Mix and Complete Mix Aerated Lagoon Systems

			•		Predictions Based on Design	Predictions Based on Design Temperature = 12 degrees Colsies and Current Flow Rate	dates and Correct Now Rate				
Date Flow	Hydraulos Desectos Tines, days no One Primary Cadi	Hydradio Hydradio Datadios Datadios Tata, days Tine, days I Ose Secondary Cellistrus A (6 cells) Cellistrus B (4 cells)	Hydralio Daerdies Tine, drys Ose Scorodury Call inTrais B (4 cells)	Efficient BOD whole Drienty Call converted to Complete Mix, Int D = 2.0 per day T = 12-dg cellin Int I = Ind Ort 100 JT-20 Int I = 1.04 per day	Billesez BOD whee Printary Cal converted to Partial Mis. 1920 = 0.276 per day T = 12-05 codius 1912 = 192011.050 T20 1912 = 0.206 per day	Elibert BOD whes Coewelig One 2nd Call to Partial Mix Call Equal to Volune of Pineary Call Reubling in Two Call is Series. Excess settling in (6 or 4 Call) sydem.	Efficient BOD when Converting One 2nd Cell and One Primary Cell to Partial Mix System (6 Cell)	Efficient BOD whee Converting One 2nd Cult and One Priessry Cult to Purist Mix System(4 Cult)	Effect BOD whee Princey Cal converted to Complete Nist. Enc. = 2.0 per day T = 12 day, colline 1e12 = 1e20*1.(65)*T-20 Ini2 = 1.04 per day	Effect BOD when Primary CAI converted to Complete Mit followed by Partial Mix is Secondary CAI (6 CAI) in 12 = 1,04 per day in 12 = 0,356 per day	Effect DOD who Primary Call converted to Complete Mix followed by Parial Mix is Scondary Call (t Call hall = 1.04 per day hall = 0.205 per day
12/49/92 30	591 0	31.6	21.1	17	8	15	Ø	13	1.1	74	ю
		27.1	18.1	13	76	19	==	91	61	60	4
	_	27.1	18.1	19	76	19	=	16	2	60	4
		31.6	21.1	17	88	SI	Φ.	23	11	73	<b>60</b>
64/66/93 30	•	31.6	21.1	11	88	15	۵	ដ	11	<b>~</b>	eo (
	0 16.5	31.6	21.1	11	83	53	6	23	12	<b>64</b>	ю ·
		27.1	18.1	19	26	81	=	16	61	<b>с</b> Э .	❤ ·
	5 14.1	27.1	18.1	13	92	19	=	16	51	<b>с</b> Э .	•
	5 19.8	37.9	25.3	7	29	=	1	•	7	r• ·	P4 (
99/02/93 3(		31.6	21.1	17	8	15	Φ.	<b>13</b>	11	P4 (	
	5 14.1	27.1	18.1	19	26	19	=	<b>9</b> 2	<b>S</b>	e3 (	₹ (
		31.6	21.1	11	88	15	•	<b>13</b>	<b>1</b> 2	P4 :	ю·
		27.1	18.1	19	26	19	=	16	<b>S</b>	eo .	4.
91/16/94 35		27.1	18.1	a	26	61	=	91	<b>2</b> 1	ю.	4.
02/02/94 4(	0 12.4	23.7	15.8	22	<b>3</b> 5	ន	7	29	23	4	'n
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eurouse 30	0 16.5	31.6	21.1	. 11	88	15	•		11	<b>N</b>	<b>.</b>
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	5 14.1	27.1	18.1	61	76	19		92	6	m .	₹ .
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11/01/94 40	0 12.4	23.7	15.8	22	<b>3</b> 5	ន	Z	22	77	4	ın ·
	5 14.1	27.1	18.1	19	26	19	=	16	61	<b>с</b> э .	₹ '
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	5 14.1	27.1	18.1	19	22	19	=	91	19	т.	₹ '
03/07/95 3(	0 16.5	31.6	21.1	17	8	51	σ.	ជ	11	~	. co
35 35		27.1	18.1	19	76	19	=	91	61	ω,	4 1
04/27/95 4	0 12.4	23.7	15.8	<b>11</b>	<b>3</b> 5	ឌ	14	70	23	4	'n

Table 3-5
Predicted Performance of Mexicali, Mexico Lagoon System If Converted to Combinations of Partial Mix and Complete Mix Aerated Lagoon Systems Predictions Based on Design Temperature  $\approx 12$  degrees/Calsus and Flow Rate  $\approx 2$  x Current Flow Rate

Efficient BOD when Primary Call converted to Congles Mir fallowed by Partial Mir Mis Scoodeny Call (4 Call) boll = 1.04 per day kpl = 6.205 per day Plow Rate = 2.x Freeed	\$	2 \$	2 5	21 :	2 \$	2 \$	2 £	<b>3</b> £	4 6	- 5	2 2	1 9	2 22	1 2	<b>S</b>	121	10	9	10	21	21	12	12	51		; ¥	2 2	1 5	2 2	1 ;
Effined BOD when Primary Cell converted to Compiles Mix followed by Partial Mix in Secondary Cell (C.C.II) kell 2 = 1.04 per day kpl 2 = 0.205 per day Filow Rata = 2 x Present	•	~ 6	<b>N</b> •	<b>&gt;</b> t	~ •	- 1	- 6			,	- 0				22	•	1	7	7	•	•	6	77	12	•	. 22	10	· i-	- 0	٠;
Efflued BOD whee Primary Cell converted to Complete Mix. Ex20 = 2.0 pur day T = 12 day, odition in 12 = in 2074; (055) T-20 in 12 = in 4 pur day in 12 = in 2074; (055) T-20 in 12 = in 2 x Freeset	5	: 2	ያ አ	ያ ድ	7 5	7 7	<b>5</b> 8	3 %	3 5	: F	: > <del>:</del>	3 5	8	×	4	%	Æ	3	31	æ	æ	36	4	8	8	4	: <b>&gt;</b>	8 25	: ×	) •
Hillson BOD when Converting the 2 del Call to and the Primary Call to Partial Mix System (4 Call Flow Rate = 2 x Present	×	<b>3 \$</b>	<b>;</b>	; ; ;	3 %	3 2	3 6	: 4	7.7	<b>S</b>	4	: £	4	#	S	4	32	35	32	43	4	#	20	20	4	20	. 4	. X	4	! {
Effloor BOD when Conventing 1992 2d Cell and Dose Primary Cell to Partial Mix System (6 Cell) Flow Rate = 2 x Present	56	3 €	\$ 8	3 %	3 %	2 %	2 23	8	20 2	92	25	792	32	33	38	33	92	92	92	33	32	. 32	38	38	32	38	33	56	32	: 6
Effloret BOD whee Corwating too 2nd Call to Parial Mix Call Equal to Volume of Primary Call Reuthing in Two Calls in Series. Excess settling in (6 or 4 Call) system. Flow Rate = 2 1 Pratog	41	: 8	<b>.</b>	<del>, 1</del>	: 4	. 4	<b>.</b>	\$	33	4	\$	41	\$	43	S.	\$	4	4	#	49	<del>\$</del>	49	S	21	49	S	49	14	49	
Efflored BOD when Primary Call converted to Partial Mix. px20 = 0.216 per day T = 12 deg. celsius px12 = px20*11.056/T-20 hp12 = 0.205 per day How Rate = 2 x Freeen	110	. 171	121	i 🖴	110	110	121	171	<b>8</b>	110	121	110	121	121	131	121	911	91	91	121	121	121	131	5	121	131	121	110	121	
Ellitout BOD when Primary Call converted to Complete Mat, hazil = 2.0 per day T = 12 dag, colaius hazil = hazil + 2.0 per day hazil = 1.04 per day How Rate = 2 x Present	31	36	<b>%</b>	æ	33	31	36	36	11	31	36	31	36	36	4	36	ਜ਼ <sub>ੇ</sub>	ਲ :	<b>.</b>	% %	<b>8</b> }	<b>%</b>	<b>3</b>	\$	36	<del>\$</del>	36	31	36	ę
Hydraulio Detection Time, days One Secondary Call inTrain B (4 cells)	21.1	18.1	18.1	21.1	21.1	21.1	18.1	18.1	25.3	21.1	18.1	21.1	18.1	18.1	15.8		21.1	21.1	21.1	18.1	18.1	18.1	15.8	15.8	18.1	15.8	18.1	21.1	18.1	15.8
nyerenes Privatelo Detention Detection Time, days Time, days One Secondary Cell inTrain A (6 cells) Cell inTrain B (4 cells)	31.6	27.1	27.1	31.6	31.6	31.6	27.1	27.1	37.9	31.6	27.1	31.6	27.1	77.1	73.7	17.1	31.6	31.6	97.0	1.77	1.17	1.77	7:53	1.57	27.1	23.7	27.1	31.6	27.1	23.7
Internation Defection Time , days One Primary Cell	16.5	14.1	14.1	16.5	16.5	16.5	14.1	14.1	19.8	16.5	14.1	16.5	14.1	14.1	12.4	14.1	10.5 7	19:5	ro.,	1.4.1	7.7	1.5	4 F	4.7	14.1	12.4	14.1	16.5	14.1	12.4
Flow CPS	33	32	35	30	30	30	35	32	25	30	32	e :	33	S :	3 ;	ð :	3 8	3 8	2 5	3 5	3 6	3 5	₽ \$	3 5	g :	<del>\$</del> ;	35	೯ :	£ :	<del>\$</del>
å .	12/03/92	01/14/93	62/03/33	63/11/53	04/06/93	65/10/93	66/60/99	81/01/93	68/02/93	69/02/93	10/06/93	11/43/93	12/01/93	11/08/34	274275	3/02/94	No.	N/10/0	1677	K/90//A		66/Delika	10/04/V	1/10/1/2E	2/07/94	91/11/95	1/31/95	56/10/50	63/28/95	94727/95

Table 3-6 Predicted Performance of Mexicali, Mexico Lagoon System If Converted to Combinations of Partial Mix and Complete Mix Aerated Lagoon Systems

		1	,	11 11 11 11	Prediction of the Code of the	Probations have dus Design Temperature = 12 degrees Caddus and Flow East = 1.5 x Correct Flow East.  Probation = 1.5 x Correct Flow East.  Probati	ture = 12 degrees Caldus no.	d Flore Rate = 1.5 x Cerrent   Prilliand ROD when	Flore Rade Professor ROD and an	PMane ROD acknes Disease	Honey Bob when Primery	Effect ROD whee Primary	
	Data Plow CFS	Hydracio Detection Time, day w One Prinary Call FS	Diversition Diversition Titles, days Ose Secondary Cell inTrais A (6 cells)	riporanao riporanao Denesies Denesies Denesies Denesies Time, days Time, days Ose Secondary Ose Secondary Call inTrais A (6 cells) Call inTrais B (4 cells)	-	callows Doubles Treaty Call Convented to Partial Hix. hp30 = 0.276 per day T = 12 deg. coldin T = 12 deg. coldin tp12 = 40.200 per day p13 = 0.200 per day Flow Rate = 1.3 x Freest	Converting One Date of Converting One Date of Call figure of Primary Call French of French of Calls in Series. Excess settling in (6 or 4 Call) system. Flow Rate = 1,5 x French Flow Rate = 1,5 x French Call or 4 Call) system.	Linear Loy was Coverting One And Call and Ose Prinary Call to Partial Mix Sydom (6 Call) Flow Rate = 1.3 x Present	Liloner DOD Weel Converting Doe Jud Call and Oos Princity Call to Pertial Mix System (4 Call) Flow Rate = 1.5 x Present	Call converted to Compile A Mix. In:20 = 2.0 per day T = 12 day, coldina In:12 = In:20 + 10 per day In:13 = In:20 + 10 per day Mix. In:20 + 10 per day Mix. In:20 + 10 per day	Call converted to Complete Mix followed by Partial Mix is Secondary Call (6 Call) is 12 = 1,00 per day jull = 6,200 per day jull = 6,200 per day	Cell converted to Complete Mix followed by Pertial Mix is Scondary Cell (4 Cell) helt = 1.04 de dy helt = 0.208 per day helt = 0.208 per day helt = 0.208 per day	
2	12/05/92 30	0 16.5	31.6	21.1	z	26	82	11	ដ	*	4	9	
ð			27.1	18.1	78	101	ਝ	77	83	83	v	œ	
8		5 14.1	27.1	18.1	28	101	ਲ	77	23	82	•	œ	
8	_		31.6	21.1	শ্ব	ĸ	78	11	ដ	z	4	•	
ó			31.6	21.1	*	16	83	11	ដ	z	4	9	
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¥		5 19.8	37.9	25.3	20	8	21	ដ	18	8	<b>6</b> 0	s.	
			31.6	21.1	*	ಕ	28	11	ឌ	র	~	v	
· <del>-</del>		5 14.1	27.1	18.1	78	101	×	77	53	83	•	œ	
			31.6	21.1	*	16	<b>5</b> 2	11	ដ	*	4	9	
-		5 14.1	27.1	18.1	28	101	ਲ •	71	23	83	•	œ	
-			27.1	18.1	83	101	ੜ	71	೩	8	<b>'</b>	œ	
*			13.7	15.8	31	110	4	92	35	31	-	2	
*			27.1	18.1	78	101	ਲ	17	23	83	•	<b>co</b>	
6			31.6	21.1	*	25	<b>58</b>	11	ដ	z	4	•	
ಕ			31.6	21.1	*	55	82	11	ដ	z	4	9	
8		0 16.5	31.6	21.1	7	25	83	. 11	ដ	*	4	•	
8			27.1	18.1	28	101	ਲ	21	જ	83	•	<b>∞</b>	
8			27.1	18.1	82	101	×	77	29	83	•	∞ '	
8	35 16/90/60		27.1	18.1	82	101	ਝ	21	23	88	•	90	
<b>=</b>			13.7	15.8	31	911	4	92	35	E	-	9	
-			23.7	15.8	31	011	41	92	35	31	7	2	
- 11			27.1	18.1	78	101	ਝ	17	53	87	•	∞	
6			13.7	15.8	31	011	41	92	35	31	7	2	
o		5 14.1	27.1	18.1	28	101	ਲ	71	53	82	9	∞	
8			31.6	21.1	*	8	78	11	ន	ጃ	4	9	
8		35 14.1	27.1	18.1	78	101	ੜ	17	53	82	۰	<b>90</b>	
6	04/27/95 4(		23.7	15.8	. 31	110	41	78	35	31	1	01	

Tables 3-7 through 3-9 were developed for the same flow patterns used in Tables 3-4 through 3-6, but the predictions are based on the measured water temperature and the current flow rates, two times the current flow rate, and 1.5 times the current flow rate. Because the design temperature was the lowest temperature reported for the 30 months of sampling, the results from the various combinations of partial and complete mix ponds for the observed operating temperatures were equal to or superior to those reported in Tables 3-4 through 3-6.

Table 3-7 Predicted Performance of Mexicali, Mexico Lagoon System If Converted to Combinations of Partial Mix and Complete Mix Aerated Lagoon Systems

25 15 14.1 7/1 18.1 15 16 16 19 14
000008 30 19 16.5 31.6 21.1 10 . 56 10 6 8 10 1 1 1

Table 3-8

Predicted Performance of Mexicali, Mexico Lagoon System If Converted to Combinations of Partial Mix and Complete Mix Aerated Lagoon Systems
Predicted Performance of Mexicali, Mexico Lagoon System Indicates Bard & Normal Wave Tappens and Partial Mix and Complete Mix Aerated Lagoon Systems

-																																
	Efflore BOD when Primary Cell converted to Complete	Mat matemat by Pathel Mat is Secondary Cell (4 Cell)	01	=		, M	**	•		~	-	. 64	. 60	• •••	•	Ħ	15	**	•	•	**	<b>6</b>	.4	64	•	•	=	=	•	40	4.	•
	Efficient BOD when Primary Cell converted to Complete	in Secretary Cell (6 Cell) in Secretary Cell (6 Cell) in T = in 20° (1.085) T-30 in T = ip 20° (1.085) T-20	•	•	. ~	•	•	- 74	ю	64	-		14	*	1	•	21	•			64	74	-		60	S	•	•	-	•	L/S	*
	Efflored BOD when Primery Cell converted to Complete	leT = 1:20*(1.063)*T-20	. 16	8	8	2	ន	23	16	2	vo	œ	12	8	2	æ	\$	27	17	. 16	=	21	•	•	<b>3</b>	21	33	æ	82	19	ន	
	Efflored BOD where Converting One 2nd Coll	Partial Miss Systems (4 Cell)	35	አ	ដ	*	15	=	*	10	S	•	<b>1</b>	15	ជ	*	31	ដ	13	13	20	21	91	2	91	20	7	21	Ħ	4		61
2 x Cerrent Flow Rate	Efficient BOD when Converting One 2nd Cell	Partial Mix System (6 Cell)	%	30	88	81	61	*	18	2	7	2	19	<b>£</b>	**	8	88	8	11	16	21	15	13	22	ឧ	<b>.</b> (2	33	33	88	18	ጽ	ន
rredictions based on Meanwed Woter Temperature and Flow Rate - 2 x Current Flow Rate	Efflore BOD when Converting One 2nd Coll to Periol Mir. Coll Francis.	Volume of Primary Cell Resulting in Two Cells in Series. Bucoss settling in (6 or 4 Cell) system.	7	47	\$	82	31	ผ	30	ដ	2	18	77	31	\$	41	21	¥	88	12	77	≴	77	71	33	39	41	21	\$	29	38	37
ons Dured on Monamed Water	Efflored BOD whom Primary Cell converted to Partial Min. In:20 = 0.235 not day	мт = h <sub>2</sub> 30°(1.056)-Т-20	110	611	114	*	*	8	æ	퓮	8	ĸ	8	*	114	119	131	Ħ	8	&	79	38	2	5	8.	<b>38</b>	119	83	114	¥	901	106
Tradica	Efficient BOD when Primary Cell converted to Complete Mir. Jury = 20 per day	lef = le20*(1.063)*T-20	31	33	83	19	ន	13	16	92	•	90	13	8	ន	æ	\$	. 12	17	91	Ξ	21	•	•	15	ĸ	33	33	ଅ	19	ដ	19
	Bydonsko Detention Time, days	S : 5	21.1	18.1	18.1	21.1	21.1	21.1	18.1	18.1	25.3	21.17	18.1	21.1	18.1	18.1	15.8	18.1	21.1	21.1	21.1	18.1	18.1	18.1	15.8	15.8	18.1	15.8	18.1	21.1	18.1	15.8
	Bydomiko Detection Time: days	One Secundary Cell in Train A (6 orlis)	31.6	27.1	27.1	31.6	31.6	31.6	27.1	27.1	37.9	31.6	27.1	31.6	27.1	27.1	23.7	27.1	31.6	31.6	31.6	27.1	27.1	27.1	23.7	23.7	27.1	23.7	27.1	31.6	27.1	23.7
	Hydradio Detection Time, days	Ose Primary Cell	16.5	14.1	14.1	16.5	16.5	16.5	14.1	14.1	19.8	16.5	14.1	16.5	14.1	14.1	7.7	14.1	16.5	16.5	16.5	14.1	14.1	14.1	12.4	12.4	14.1	12.4	14.1	16.5	14.1	12.4
	West	Temperature deg. echies	21	13	<b>S</b>	19	18	*	ន	53	31	8	52	18	15	13	2	16	8	77	8	27	30	8	ห	21	13	15	15	19	81	ដ
		§ 80	30	35	32	30	2	30	32	32	53	30	35	30	32	32	\$	32	30	30	30	32	32	35	<del>\$</del>	各	32	\$	32	9	33	\$
		å	12,00,62	01/14/35	62,007.00	88/11/20	04/06/38	05/10/30	06/09/38	03/10/10	06/50/90	09/07/98	10/06/93	11/03/90	12/01/98	1,0594	<b>62/02/94</b>	60,702.94	<b>64</b> /0494	95/04/94	<b>94</b> /02/94	#470/LB	04/04/94	96/90/60	10/04/94	11,61,64	12/07/94	91/11/95	61/31/95	63,677.95	\$6,121,00	04/Z7/95

Table 3-9
Predicted Performance of Mexicali, Mexico Lagoon System If Converted to Combinations of Partial Mix and Complete Mix Aerated Lagoon Systems

				•	•	Profest	Profestons Bused on Monarced Water Transporature and Flow Rate 5.5 x Overest Flow Rate	Temperature and Flow Rude	1.5 x Oursel Ther Bate				
,			Hydra Tine	Hydradio Describe Time, days		Efficient BOD when Primary Call converted to Complete Min. 1420 = 20 per day	Efficient BOD when Primary CLB converted to Pertial Mis. 1920 = 0.276 per day	Efficient BOD when Conversing One 2nd Cell to Partial Mis Cell Repuil to	Effect 800 when Converting One 2nd Cell and One Primary Cell to	Efficie 300 when Covering On 2nd Call and On Primary Call to	Efficient BOD when Primary Call convented to Complete Witz. 1420 = 2.0 per day	Efficient BOD when Primary Call converted to Complete Mix followed by Period Mix	Milmet BOD when Primary Call maverale to Complete Mix followed by Perial Mix.
Ž	<b>1</b> 8	Temperature deg. oclains	3	One Secondary Cell inTrain A (6 cells)	Oce Securing Cell in Train B (4 cells)	leT = 1.20*(1.005)*T-30	14T = 1420*(1.0)6)*T-20	Volume of Primary Cell Rembing in Two Cells in Series. Excess settling in (6 or 4 Cell) system.	Parisal Mat System (6 Cct)	Partial Min Symbon (4 Cod)	0;·1.(30:1).05% - 1=	In Secondary Cod (6 Cest) For = Incor(1,015)·T-20 For = Incor(1,036)·T-20	In Secondary Col. (4 Col.)
12,40,92	30	12	16.5	31.6	21.1	7	91	8	17	ន	7	*	vo
91745	32	51	14.1	27.1	18.1	%	&	æ	ឧ	88	ጽ	v.	1
92,003,50	32	<b>3</b>	14.1	27.1	18.1	ដ	¥	ጽ	18	ង	ដ	<b>~</b>	<b>1</b> 0
60/11/50	8	61	16.5	31.6	21.1	7	24	13	21	16	*	64	€0
8406/38	30	18	16.5	31.6	21.1	15	æ	ន	ជ	11	15	64	**
05/10/50	8	ጸ	16.5	31.6	21.1	2	<i>19</i>	15	•	ជ	10	-	64
<b>86.09/33</b>	35	ន	14.1	27.1	18.1	12	4	ន	ដ	16	11	м	••
SQUAL	35	ક્ષ	14.1	27.1	18.1	7	*	*	œ	11	1	=	-
\$6,05/35	•	31	19.8	37.9	25.3	ĸ	- 14	-	•	•	<b>5</b>	•	-
89/00/98		8	16.5	31.6	21.1	vo	88	=	•	•	•	-	-
10/06/95		53	14.1	27.1	18.1	2	Ę	18	=	21	9	-	64
11,40,75		18	16.5	31.6	21.1	<b>21</b>	æ	ន	22	11,	15	ч	60
12.01/30	35	15	14.1	27.1	18.1	ដ	¥	æ	82	ĸ	ដ	•	vo
P6/50/10	•	53	14.1	27.1	18.1	×	8	33	8	88	32	s.	1
#570378#	-	21	12.4	23.7	15.8	31	91	4	8	35	ж.	1	92
MS/ED/SM		97	14.1	27.1	18.1	77	8	88	17	ጽ	ដ	· · · · · · · · · · · · · · · · · · ·	<b>50</b>
H010H	30	ន	16.5	31.6	21.1	22	74	81	11	15	23	~	60
<b>BS/04/94</b>		ដ	16.5	31.6	21.1	12	Ę	11	2	<b>*</b>	21	74	74
P6/20/94		8	16.5	31.6	21.1	90	8	13	<b>00</b>	=	œ	_	
#7.06/94		21	14.1	27.1	18.1	6	69	91	•	<b>13</b>	•	-	<b>6</b> 4
1670494	32	8	14.1	27.1	18.1	7	\$	7	œ	=	7	-	1
9670760	•	8	14.1	27.1	18.1	7	2	*	<b>S</b>	=	7	-	
10.04.94	-	23	12.4	23.7	15.8	21	8	ដ	13	<b>5</b> 2	11	64	60
11,01/94	4	77	12.4	23.7	15.8	91	86	23	16	ដ	16	m	•
12,07/94	35	<b>51</b>	14.1	27.1	18.1	*	8	33	ន	88	25	50	1
91/1/95	\$	15	124	23.7	15.8	X1	103	35	ដ	30	જ	Ŋ	1
6131/95	35	15	14.1	27.1	18.1	Ħ	¥	30	18	<b>53</b>	ដ	•	•
\$6100.00	30	19	16.5	31.6	21.1	7	76	19	21	91	Z	7	<b>6</b> 3
\$6/28/95	32	18	14.1	27.1	18.1	81	<b>8</b> 8	ጽ	16	77	<b>\$</b>	~	<b>~</b>
84127/95	•	ដ	12.4	23.7	15.8	15	84	ĸ	21	12	15	6	4

# CHAPTER 4

NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

## **CHAPTER 4**

# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

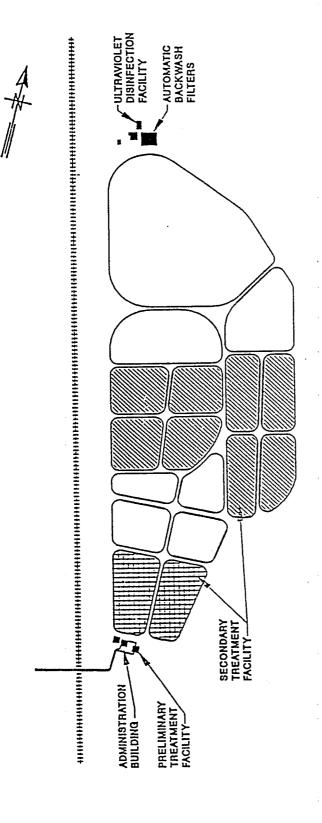
The Nogales International Wastewater Treatment Plant (NIWTP) is located on the USA side of the border, and receives approximately two-thirds of the plant influent flow from the Mexican side of the border. The wastewater from the Mexican side of the border is a combination of storm runoff and domestic and industrial wastewater. The wastewater being treated by the NIWTP is a relatively weak sewage because of the dilution from the stormwater events and groundwater infiltration.

The Nogales plant was designed to treat an annual mean flow rate of 15.75 MGD and a peak flow of 28 MGD. The ultimate mean design flow is 17.2 MGD with the addition of aerators. The system currently is treating an annual mean flow rate of 14.9 MGD. Flow from Nogales, Sonora is estimated to increase to 32.3 MGD by the year 2020 with relatively small increases projected for Nogales, Arizona.

# SYSTEM DESCRIPTION

A plan view of the NIWTP is shown is Figure 4-1. The NIWTP has extensive preliminary treatment consisting of screening and degritting. Preliminary treatment is followed by two parallel trains of secondary treatment with each train having a complete mix lagoon followed by four partial mix lagoons. Only three of the partial mix lagoons are in service with the fourth used to dry settled solids for removal. The lagoon system is followed by five dual media automatic backwash filters for solids removal prior to disinfection with an ultraviolet facility.

The preliminary treatment was added after the plant was constructed because of very large concentrations of grit in the wastewater. Based on the average daily flow rate of 15.75 MGD, the plant produces about 2 cubic yards of grit each day, and the total volume of grit and screenings is approximately 4 cubic yards per day.



# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

# LEGEND:

- AREA FOR FUTURE EXPANSION
- COMPLETE MIX LAGOON
- PARTIAL MIX LAGOON

# FIGURE 4-1 SITE PLAN

# **DESIGN ANALYSIS Design Assumptions**

When using any of the conventional formulas and removal rate constants for the design of the Nogales plant, it is not possible to obtain the effluent BOD concentration predicted in the operations manual. It appears that the designer made an incorrect assumption that having two trains of equal volume would double the detention time or it was assumed that a removal rate constant of 6.0 per day for the complete mix lagoon was acceptable. In either case, the quality of the effluent was over estimated.

Based on conventional formulas and rate constants between 2.0 and 3.0 and using Equation 10, the effluent BOD concentration will range between 53 and 72 mg/l. Calculations are shown below for rate constants ranging from 2.0 to 6.0 per day.

$$\frac{C_n}{C_0} = \frac{1}{\left[1 + (kt/n)\right]^n} \tag{10}$$

where

 $C_n$  = effluent BOD concentration in cell n, mg/l

 $C_o$  = influent BOD concentration, mg/l

k = first order reaction rate constant, days<sup>-1</sup>

= 2.0 day<sup>-1</sup> at 20°C (assumed to be constant in all cells).

t = total hydraulic residence time in pond system, days

n = number of cells in the series.

When  $k_c = 2.0$  per day:

$$C_e = \frac{260}{(1 + k_c t)} = \frac{260}{(1 + 2(1.3))} = 72 \text{ mg//}$$

When  $k_c = 2.5$  per day:

$$C_e = \frac{260}{(1+k_c t)} = \frac{260}{(1+2.5(1.3))} = 61 \, mg/I$$

When  $k_c = 3.0$  per day:

$$C_e = \frac{260}{(1+k_c t)} = \frac{260}{(1+3.0(1.3))} = 53 \, mg/I$$

When  $k_c = 4.0$  per day:

$$C_e = \frac{260}{(1+k_c t)} = \frac{260}{(1+4.0(1.3))} = 42 \, mg/I$$

When  $k_c = 5.0$  per day:

$$C_e = \frac{260}{(1+k_c t)} = \frac{260}{(1+5.0(1.3))} = 35 \, mg/I$$

When  $k_c = 6.0$  per day:

$$C_e = \frac{260}{(1 + k_c t)} = \frac{260}{(1 + 6.0(1.3))} = 30 \, mg/I$$

# **Design Predictions**

The NIWTP was designed for an annual mean flow rate of 15.75 MGD and an ultimate mean flow of 17.2 MGD and an influent and effluent BOD of 260 and 30 mg/l, respectively. The following calculations were made to predict the performance for various combinations of complete and partial mix aerated pond systems at 20 °C and 11 °C.

20 °C. At the design BOD concentration of 260 mg/l and a flow rate of 15.75 MGD, the following results were obtained.

$$C_{ec1} = \frac{C_o}{\left[1 + \frac{k_c t}{n}\right]^n} = \frac{260}{\left[1 + 2(1.3/1)\right]^1} = 72 \, mg/l$$

$$C_{ec2} = \frac{72}{\left[1 + 2\left(\frac{1}{1}\right)\right]^{1}} = 24 \, mg/I$$

The two complete mix cells are followed by two partial mix cells.

$$C_{ep2} = \frac{24}{\left[1 + 0.276 \left(\frac{2}{2}\right)\right]^2} = 15 \, mg/I$$

With three complete mix cells in series (the original CM cell and the conversion of two of the partial mix cells to CM) at the above conditions, the following was obtained.

$$C_{ec3} = \frac{72}{\left[1 + 2\left(\frac{2}{2}\right)\right]^2} = 8 \, mg/I$$

At 20 °C the design will produce an effluent that will satisfy the effluent standard of 30 mg/l of BOD. Better solids control can be exercised by better design of overflow structures and a reduced hydraulic residence time in the settling basins.

11 °C. The following effluent BOD concentrations were calculated for a design flow rate of 15.75 MGD and a design BOD concentration of 260 mg/l.

$$C_{ec1} = \frac{C_o}{\left[1 + \frac{k_c t}{n}\right]^n} = \frac{260}{\left[1 + 0.96(1.3/1)\right]^1} = 116 \ mg/l$$

$$C_{ec2} = \frac{116}{\left[1 + 0.96 \left(\frac{1}{1}\right)\right]^{1}} = 59 \ mg/I$$

The two complete mix cells are followed by two partial mix cells.

$$C_{ep2} = \frac{59}{\left[1 + 0.201 \left(\frac{2}{2}\right)\right]^2} = 41 \ mg/l$$

If three partial mix cells are converted to three complete mix cells in series, the effluent quality will improve to 15 mg/l, and one "partial mix" cell will remain for settling. If the remaining "partial mix" were converted to a true partial mix cell rather than a settling cell, the effluent BOD concentration would be 21 mg/l.

Similar calculations are shown for actual operating conditions and increased flow rates in Tables 4-1 through 4-4. The physical characteristics of the NIWTP lagoons are shown in Table 4-1 along with the influent and effluent characteristics for the system. Table 4-2 shows the predicted performance for various combinations of partial and complete mix cells at 11 degrees celsius and the measured temperature and flow rate. The predicted effluent from the existing complete mix aerated lagoon shows that the system cannot meet the predicted effluent quality as specified in the operations manual. By modifying the system to various combinations of complete and partial mix aerated lagoons, the system will be able to produce an excellent quality effluent in terms of BOD.

	· · · · · · · · · · · · · · · · · · ·	
Total HDT in System days	2. 4 4 8 8 8 9 8 9 8 9 8 9 8 9 9 9 9 9 9 9	8.4. 8.5.
Hydraulic Detertion Time in Orn PM Cell days	128 128 128 128 128 128 128 128 128 128	1.02
Hydraulic Detention Time in One CM Cell days	2. 2. 2. 2. 2. 2. 2. 2. 2. 2. 2. 2. 2. 2	<u> </u>
Volume in One PM Cell cubic meters	31007 31007	31037 31037
Volume in One CM Cell cubic meters	9341.5 9341.5	37471.5 37471.5
¥ #	Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z Z	21 23
Temperature Efflu degree		
T Influent degrees F		ដដ
gen Effluent mg/L	100 100 100 100 100 100 100 100	7.9
Dissolved Oxygen CM2 mg/L		4.8
CM1 mg/L	, \$2.5\$21\$6725888888888888888888888888888888888888	5.0 4.6
Efflert pH	8824886661116148888886668888888888888888	7.2
Induent pH	2,4,4,4,4,4,4,4,4,4,4,4,4,4,4,4,4,4,4,4	7.0
Feel Coliform Influent pH Hifturnt pH no./100 mL	g g s s s s s s s s s s s s s s s s s s	<del>1</del> 4 E
Effluent TSS mg/L	であけまななりな。 ガロけいけははいけんないけん はいけいけいけん はいけん はいけん けんしゅ にらける にっぱん はっぱっぱん はっぱん はっぱん はっぱん はっぱん はっぱん はっぱん	8 Z
Influent TSS mg/L	13	157
Effluent BOD mg/L	*	21 18
Influent BOD mg/L		Z Z1
Effluent Flow Rate Usec	8 6 6 6 5 5 4 4 4 5 5 5 5 5 5 5 5 5 5 5 5	203 203
Influent Flow Rate Usec	\$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$ \$	70/ 699
Date	Teb 91  March 91  March 91  May 10  Juny 10  Ang 10  A	May 95

Table 62

Neples international Winderstrational Winderster Tradement Final Predicted Performance If Converted to Combinations of Period Pilit and Compilete Mits Arraical Lagous Systems Predictions Essed on Design Temperstate or II degrees Colous and Missaured Temperstates and Filor Pale

Efficient DOD from CM Coll and Convertion of Three Parkeds Mills Colls to CM (it = i.2 i, il = ii.4) Milvered by Saurita ender, PM Cell (Stalling Coll, it = 1.0) at measured FR & Temp.	<b>~</b>	<b>?</b> E	0.38	0.26	97'0	0.01	29.0	70.0		· eo	en	•	- 66	0.28	0.31	0.14	9.02	0.22	0.38		•	n en	60	7	-	0.28	0.10	0.27	} <b>-</b>	**	<b>64</b> (	Ν =		0.21	0.18	0.13	0.15	0.24	-	, ,	<b>.</b>	n r	. ·		۱ 🗝	
Efferent BOD from CAI Cell and Converten of Two "bracks MAY Colds to CAI (It = 1.24, Ca = 1.84, d. 1.8 t.) Milwood by adding PM Call Scaling Cell; i. = 1.0, at measured FA. & Temp.	Va (	<b>10</b> 04		. —	0.3	0.1	 		- e-	• •	-	•		· •		-	0.2	-	_	4	r	~ 10	7	<b>v</b> o	74	<b>-</b>	٦,	<b>-</b> 6	1 (**)	4	₹ .	er e	° 6			-	-	1	<b>с</b> о .	<b>v</b> (	× \$	3 +	~ <i>V</i>	v	) eo	
Effects (200 frees Complete Mit Call and Converted of One "Bartial Mit" Call to a Complete Mit Call (11st 12 d, Cat. 18 d) fallowed by calling PM Cells at some IR & I tomp.	<b>Z</b> (	15	. 4	· w	2	-	⊷ •	- 7		· <b>2</b>	23	•	۰ د	n <b>v</b>	· w	. 60	-	S	<b>.</b>	2	¥	3 23	<b>52</b>	22	Φ	<b>'</b>	<b>.</b>	<b>+ V</b>	. 00	۵	Φ,	<b>5</b>	<b>0</b> 0¢	<b>4</b>	- 4	4	*	S	10	<b>2</b> /	= \$	2 :	<u>o</u> e	3 2	<b>:</b>	
Effernt BOD from Georgiete Mix Celi and Conversion of Tartial Mar's Celi to True Partial Mix Band on Missaved Torp, and Time Rase (FM) kp = kp34(1.456) T-26	E:	3 5	i rv	, <b>x</b> o	٣		~ ~	٦ ٥	^ <b>=</b>	: 82	11			e ve		- 4	~	œ	7	ដ	ō	98	ន	Ħ	14	Φ.	4 1	~ o	, <u>e</u>	H	= :	= :	<b>=</b> =	i w	. ~	. 7	7	٥	14	19	2 2	3 5	<u> </u>	2 2	31	
Effect ECO free Complete Max Call and Convertes of Factor Max Califor True Partial Max kpil = 4,776(450,9 kpil = 4,776(450,9 kpil = 6,24) per day	ង៖	3 4	; a	ដ	6	9	œ <b>-</b>	۲ ک	9 6	#	n	;		. 2	: 12	. 21	, vo	Ħ	17	ដ	J.	3 23	37	4	37	<b>8</b> 2 :	2 8	2, <u>s</u>	12	16	<b>S</b> 3 :	<b>2</b> 9	3 2	1 52	177	53	87	23	31	<b>8</b> 8	7 1	a :	<b>,</b> 5	Ę <b>Ş</b>	5 E	
Efforce BOD from Complete Mix Cell K-24 = 2.0 per day T = 11 dry, cedden kell = k-241 desy T-20 kell = 4.940 per day	<b>F</b> 8	3 8	: 61	6	11	<b>2</b>	2 5	2 5	8 8	. XS	4	\$	9.7	78 T	32	<b>33</b>	01	\$\$	32	4	**	<b>.</b> 3	88	8	£	ස :	25 55	S 55	8 8	8	<b>8</b> 2 :	E Y	8 <b>5</b>	÷ ==	: ×s	8	88	88	S	S ;	ደ ነ	ጸኔ	ጽ ፡	; v	3 3	
Total HDT in System days	5.04	4.26	5,53	5.98	22.9	89.9	6.97	C7 5	6.51	5.72	5.34		2.5	5.27	5.81	5.69	5.19	5.37	5.70	5.32	4 10	4.59	4.70	5.01	5.29	88.	8.68	4. 4 6. 8	¥.	5.09	5.16	2, 2	5 C	5.40	25.	5.84	5.68	5.34	5.04	5.52	17.4	8 8	79.5	3.65	£.7.	
Hydraulic Detenden Time ta One PM Cell days	1.20	\$ 5	1.32	1,42	9:1	1.59	8 9	6. 5	55.1	1.36	1,27	•	2:	1.25	1.38	1.35	1.23	1.28	1.36	1,26	8	1.09	1.12	1.19	1.26	<b>5.</b> ;	1.35	2 : I	1.17	1.21	1.23	1.17	1.23	78	5	1.39	1.35	1.27	1.20	<b></b>	70.7	6.9	, y	? =	1:1	
Hydraulic Detration Time is Ose CM Cell days	1.45	27.1	1.59	1.73	1.93	23	2.00	? 5	18.	<u>2</u>	1.53	5	75.	5.1	1.67	1.	1.49	. 1.54	<u>1.6</u>	1.53	1 30	133	1.35	1.4	1.52	9.	3. i	<u> </u>	1.42	1.46	1.48	1.41	. e	55	1.62	97.	1.63	1.53	1.45	1.30	7. T	9 5	31.1	2 2	136	
	4 ;	ī 7≅	18	22	ষ	78	8 %	3 5	1 52	2	= :	<b>13</b>	<b>3</b> \$	2 F	7	ន	*	7	20	51 t	3 2	<b>.</b> 7	<b>3</b> 2	61	77	<b>n</b> :	S 5	\$ 2	22	7	<b>E</b>	: E	<u> </u>	: 2	8	92	58	ষ	<b>S</b>	<b>S</b>	3 5	3 7	9 2	2 2	77	
Temperatore Influent Effluent degrees C degrees C	11	¥ #	<b>.</b>	**	S3	% ?	2 2	3 %	រ ន	18	9 ;	L :	2 5	ដ ដ	2	*	z	*	77	ន :	2 =	: 11	92	19	IJ	<b>3</b>	2 3	3 2	2	11	<b>9</b> 9	2 5	3 2	: 3	77	7.7	11	92	22	នុះ	<u> </u>	2 5	7 7	7 7	: ::	
lottern BOD rught	112	3 2 <u>3</u>	\$	178	z	<u>چ</u> ا	56	8 5	27	4	S 2	æ ¥	<b>8</b> 6	3 8	<b>.</b>	ន	z	110	8	≅ 5	7 7	: <u>S</u>	157	8	179	<u>4</u>	<b>3</b> 8	8 2	<b>2</b> 2	ፎ	ଚ (	2 8	201	76	143	157	148	143	142	EE :	3 3	114	801	142	12	
Influent Flow Rate Linc	8	£ 55	35	206	450	<b>2</b>	<b>3</b> 8	\$ \$	\$ \$	675	200	747	6 5	<b>1 1 3</b>	025	z	285	563	23	269	7,	§ §	8	509	ZJ.	514	750	8 8 8	612	293	286	\$ 6 5	8	839	536	218	23	200	<b>S</b> S :	§ §	2 8	7 5	7.0	202	639	
ž	Jan 91	March 91	April 91	May 91	June 91	14 91 12 5	Aug 91	865.51 04.91	Nov 91	Dec 91	Jan 92	Feb 92	Miarch %	May 92	June 92	July 92	Aug 92	Sept 92	0d %	New 20	Len 93	Feb 23	March 93	April 93	May 93	June 93	S Ainf	S 55 S 55 S 55 S 55 S 55 S 55 S 55 S 55	Nov 93	Dec 93	ar i	28 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2 2	Apr	May 94	Jun 94	ᄷᅖ	Aug 94	Sep SE	ह्य इ.स.	X &	rec X	C Tall So	Mar 95	Anr 95	May 95	

Table 4.3

Nogales international Nogales international Windowsker Freatment Plant
Predicted Performance If Converted to Combinations of Partial May and Complete MIX Acrabed Legend Systems
Predictions Research Discon-Temperature at 14 decrees Codings and Measured Temperature and 2 to Measured Prince 26.

Efficient BOD from CM Cell and Converting of Three "Parish Mrv Cells to CM (1 = 1,2 d, 2 = 1 e, 2 + 1 e, 2 d) followed by fourth cells. PM Cell (8-e) followed by fourth cells. PM 2 e, 2 e	9 9	3 5	3 ~	• ~	50	0.1	0.2	0.2	~ 4	. 2	<b>12</b> ,		4.	• •	. ~	-	0.4	~		۲	12	1 =	21	•	₹ (	۰ وم	٠,	· 100	•	-	r- r	- <b>t</b> ri	• ◄	_	1			. 0	=	14	17	22	<b>.</b>	, v	,
Efficient 300 from CM Cell and Conversion of Two "Partial Mrr Cell to CM (11=1.2 d, Cn=1.0 d, Cn=1.3 d) followed by catalong PM Cell (Sealing Cell, t= 1.0) at measured F. & Temp. 2 x Measured Flow Eats	82 5	2 %	₫ ◄	<b>.</b>			-	⊶ `	• :	20 21	70		<b>-</b> -	+ ¬	r sen	(1)		¥s	•	<b>E</b> 1	9	<b>.</b> 22	n	61	ο, 1		4 V	• •	01	21	21 22	1 2	, a	4	च	<b>.</b>	<b>4</b> u	. 2	70	22	76	61 :	Σ <u>i</u>	2 2	:
Efficient BOD from Complete Mix Cell and Conversion of One "Partial Mix Cell to a Complete Mix Cell (11=1.2 d, 12=1.0 d) (allowed by estiting PM, Cell as Inose, FT & Temp. 2 x Measured Flow Easte.	88	<b>8</b> 3	•	. 4	i va	7	€ .	~ ;	<b>2</b> £	: X	33	;	<b>5</b> F	. =	: 53	7	es	<b>23</b>	នា	z	ş	8	\$	39	<b>z</b> :	9 6	- 2	១	18	50	£1 £2	20 2	2	91	ន	3 5	2 2	3 X3	8	36	38	33	17 :	2 X	ì
Efficient BOD from Complete Mix Celi and Convexion of "Partial Mix Celis to True Partial Mix Bard on Measured Temp, and Flow Rate (TR), kp = kp2/k1/260, T.28, 21 Measured Flow Rate	36	\$ 2	\$ ≅	2	<b>00</b>	4	•	4 ;	3 E	: 4	36	;	El e	, <del>2</del>	<b>2</b>	11	₹	70	<u>s</u>	53	<u> </u>	38	20	ដ	٤ څ	<b>8</b> 5	3 %	61	22	ន	3 2	3 X2	23	14	77	7 7	77	33	41	4	4	38	£ :	7 ½	;
Effuent BOD from Complete Mix Cell and Conversion of Partial May Cellita True Partial Mix Ppill = kp201.009/T-29 kpill = 0.201 per day 2 x Measured Flow Rata	₩.	3 6	2	5	77	23	<b>8</b> 2 ·	o [	<u>ک</u> ک	83 E	4	;	12 ¥	<b>2</b>	ঃ	z	10	\$	≯:	4	45	\$	8	<b>&amp;</b> i	۲ I	S	3 00	38	R	<b>8</b>	8 5	: ×	8	8	<b>8</b> 2 8	វន	ñ 5	<b>.</b> 8	8	SI	SS :	<b>%</b> 1	នៈ	8 G	ļ
Effluent BOD frant Compilete Mix Call kc20 = 2.0 per day T = 11 day, codius T = 11 day, codius kc11 = 4.950 per day 2 x Meaured Florr Rale	<b>38</b> 8	8 5	<b>:</b> 2	;	32	20	53	<b>7</b> 8	<b>2</b> 6	8	ន	•	% ¢	3	8	35	2	8	ب ا	ឌ	8	8	83	<b>21</b> :	S 2	8 5	F 64	\$	4	<b>4</b>	<del>3</del>	· 55	19	4	≅ 8	≥ 8	3 &	: : 3	83	76	<b>2</b> 7	37 E	D/ 60	ે કે	
Total HDT in System days	5.04	4.26	5.53	5.98	6.72	89.9	6.97	6.25	8.5	5.72	5.34	;	20.4	5.27	5.81	5.69	5.19	5.37	5.70	5.32	4.18	4.59	4.70	5.01	5.29	80.4	4.76	4.99	¥.	5.03	5. F	5.04	5.19	5.40	2. S	\$ °	8 7	5.04	4.52	4.27	3.68	3.82	4.03	£. 4.	:
Hydraulic Detention Time in One PM Cell days	1.20	<u> </u>	1.32	1.42	1.60	1.59	99 :	1.49	85.1 25.1	1.36	1.27	;	2 2	1.25	1.38	1.35	1.33	1.28	1.36	1.26	0.99	1.09	1.12	<u></u>	1.26	8.1 8.2	1.13	1.19	1.17	1.21	3.1	1.20	1.23	1.28	<u> </u>	1.39	1.27	1.20	1.07	1.02	0.87	0.91	£ :	7 27	
Hydraulic Detention Time in One CM Cell days	1.45	27.	8	1.1	1.93	1.92	8:3	. i.	181	2.	1.53		7 1	1.51	1.67	1.63	1.49	<b>3</b>	\$ (	1.55	1.20	1.32	1.35	<b>4</b> (	75.	9 5	137	1.43	1.42	2.49 54.50	8. T	1.45	1.49	1.55	1.62	8 E	3 15	1.45	1.30	1.23	9.	1.10	e :	1.36	
	<u> </u>	3 72	2 22	ដ	*	78	82 3	ន រុ	7 5	2	= :	3 ;	<u>a</u> 2	: 7	77	ដ	ጸ	য় :	2 ;	S 12	4	4	92	<u>s</u>	3 5	3 X	3	.07	15	₹ :	3 22	12	18	2	X X	3 %	3 3	19	15	22	£ ;	9 9	<u>e</u> 2	21	
Temperature Influent Effluent degress C degress C	11	<u> </u>	6	**	23	23	8 3	<b>a</b> 5	3 8	<b>.</b>	9 :	<b>≥</b> \$	2 P	: 2	12	শ্ৰ	23	র :	2 2	2 2	=	11	<b>82</b>	≘ {	3 2	\$ \$	র	ដ	92	<u> </u>	2 82	61	71	<b>3</b> !	3 5	3 5	3 X	ដ	22	10	<u>e</u> :	<b>7</b> 7	7 7	: 2	
8	112	<u> </u>	\$	128	79	æ !	25	9 5	127	<b>∓</b>	<u>8</u>	£ ¥	<b>₽</b> ₹	8	25	ន	z	01 01 1	₹ ?	122	X	103	157	<u>8</u> į	173	<u> </u>	8	8	<b>%</b>	F	3 6	18	<u>\$</u>	92 ?	₹ Ē	148	£ 5	142	133	121	¥ ;	<u> </u>	001 CF1	142	
influent Flow Rate L/sec	8	8 6	<b>₹</b>	206	450	452	<b>\$</b> 8	§ ź	<u> </u>	25	200		g 5	ST.	220	231	283	S	3 5	S S S S S S S S S S S S S S S S S S S	22	65)	<b>₹</b>	g i	7 2	23.	SS	<b>50</b> 5	219	233	614	009	283	229	518 518	8	200	299	699	707	<del>2</del> 2	187	vc.)	8	
Date	Jan 91 Ear 01	March 91	April 91	May 91	June 91	July 91	Aug 91	Sept 2	Nov 91	Dec 91	Jan 92	160 %	Anril 92	May 92	June 92	July 92	Aug 92	Sept 92	7 S 2 2	Nov 52 Dec 92	Jan 93	Feb 93	March 93	April 93	May %3	July 93	Sept 93	Oct 33	Nov 93	Dec 93	Feb 25	Mar 94	Apr 94	May 94	t a	Ang 94	Sep 24	<b>8</b> 50	Nov 94	Dec 24	Jan 95	Mor 95	Anr 95	May 95	

Table 44

Negel et internation | Negel et international Windowsker Treatment Flust
Frederich Performance | I Commercial Combinations of Partial Pits and Computed Pitz Arrainal Laguess Systems
Frederiches Raund on Deriges Temperature = | II degreet Calous and Measured Temperature and 3 E Measured Flust

Elitent 100 from CA Call and Correction of Three "Partial Mar Call to CA (it = 1.3 4, G = G = (4 = 1.8) Call Goottle Call ( = 1.8) Call Goottle Call ( = 1.8) A treasured Fit & Temp. A Variented Fit & Temp.	90	\$ ¥	3 %	\$ <b>~</b>	n <b>v</b>		40	; <del>-</del>		8	<b>=</b> 1	<b>3</b> 2	1	90	4	4	us.	ю ·		* 4	, <b>2</b> 3		77	ឧ	<b>3</b> 8	3 0	\ <b>\</b>	14	to	<b>•</b> ;	= 52	<b>13</b>	23	= :	2 •	P 99	· 65	₹.	en (	2 t	3 K	3 23	ឌ	11	61	21
Efform 100 from CH Cell and Correction of Tro- *Parish Mar Cellon CH (R=1.24, C=1.04, C=1.04) followed by anticipy Cellomic Cellomic St Frontine Train Trop. St Frontine Train Tax. St Manufol Train Train St Manufol Train Train St Manufol Train Train St Manufol St	ę	3 %	3 =	F &	• =	; ~	, —	• ••	-	27	2 3	<b>3</b> E	•	21	9	•	9	۰ ص	rı \$	2 =	ឧ		33	S	8 8	ţ 2	2	40	•	<b>1</b> 1	÷ \$	19	<b>S</b>	<b>2</b> 2 9	. •	<b>.</b> 0	90	σ.	= 8	21 22	3 ኢ	3 8	31	26	30	ដ
Efferent 200 from Correlate Mac Coll and Converted of One "Partiel Mac Coll to a Complete Mix Coll to ((1=1.54 for 1.04) fellowed by calcing TM Coll to a mac. Ft & Tomp.		3 6	; 5	2 2	24	ς α	o 4	• •	4	78	<b>X</b> :	<b>4</b> 2	;	18	10	91	70	23 '	w <del>;</del>	3 5	38		41	<b>4</b>	S &	9 8	3 23	23	19	7 7	<b>8</b> 85	77	78	8 3	62 5	2 22	77	7	<b>X</b>	<b>8</b> 8 <b>9</b> 8	\$ \$	£ 55		39	87	39
Efficient 200 from Complete Ms. Coll med Complete Ms. Coll in Their Ms. Ms. Coll in True Partial Ms. Beard of Pleasand True, and Frow Late (F. M. Mannered From Ms. Mannered Ms. 3 x Mentered From Ms. 2 x Mentered From Ms.		\$ 5	3 5	2 5	2	: =	<b>1</b> ~	. 01	•	4	4 t	7 94 8	<b>;</b>	21	<b>3</b>	11	11	<u>.</u>	۲.	. %	4		4	<b>.</b>	3 8	ŧ 7	; <del>\$</del>	23	11	F 7	25	31	33	£ 1	3 23	3 8	ಸ	ੜ	.37	<del>\$</del> [	ัง ถ	3 25	8 68	47	88	ĸ
Elformé 100 frances de la complete M1 Cell and Complete M1 Cell and Tort Partie M2 Cell to		5 2	÷ 5		16	38 8	2 82	23	11	<b>2</b> 2	ક ક	2 G	i	76	81	. A	\$	33	<b>2</b> 2	8 %	રા દ		<b>3</b> 2	S 63	<b>2</b> 2	3 34	76	4	<b>3</b>	<del>র</del> হ	3 %	37	€	<del>4</del> ,	ጽ 等	3 E	82	27	ኤ የ	F ¥	۶ ۶	: 4	; t	æ	8	79
Effort BOD hen Complete Ms Cal Eds = 24 per day T = 11 deg. (45 day Ed 11 = £2041, 1865) T.D Ed 12 = 55 ber day 3 x Measured From Rat.	Y.	2 5	: =	æ	: S	25	* *	x	11	10 <b>6</b>	<b>?</b> 3	<b>;</b> E	!	æ	ដ	4	<b>S</b>	<b>;</b>	2 2	2 23	£		<b>%</b> :	2 5	130	120	2 2	¥	<b>S</b> I	3 23	8 <b>2</b>	41	S. 1	જ :	7 5	<b>3</b>	102	5	<b>%</b> \$	≨ 2	t S	s <b>\$</b> 8	<b>.</b> %	79	102	\$
Total HDT is System ches	2	4.38	4.26	5.5	2.38	22,9	89.9	6.97	6.25	6.65	<u>5</u>	25.5		4.62	2. 2	5.27	5.81	5.69	5.19	2.30	5.32		4.18	\$6.4 66.4	5.5	5.29	5.88	5.68	4.76	ş. ş	, 65 12 13 13 13 13 13 13 13 13 13 13 13 13 13	5.16	2.3	2. 2.	 	5.6	5.84	5.68	5.34	2. 2.	4.24	3.68	3.82	4.03	4.30	4.73
Hydraulic Detraition Thate is One PM Cell days	1.20	2	9-1	132	1.42	9.1	1.59	99.1	1.49	1.58	1.55	1.27	į	1.10	1.10	1.25	1.38	1.35	27.1	2 2	1.26		0.99	<u> </u>	7 2 1	1.26	<del>5</del>	1.35	1.13	<u> </u>	17	1.23	1.17	1.20	2.1		1.39	1.35	1.27	1.20	3 2	0.87	16.0	0.96	1.02	1.13
Hydraelic Decention Thee is Ons Cell Cell deyr	145	1.26	1.22	1.59	12	1.93	1.92	5.00	1.79	1.91	1.87			1.32	1.33	1.51	1.67	3. 3.	÷ 2	<u> </u>	1.53		1.20	1.32	<u> </u>	1.52	1.69	1.63	1.37	3 5	1.46	1.48	<del>1.</del> ;	3.45	 5	29.	1.68	1.83 5	1.53	5 5 5 5	1.23	8	1.10	1.16	1.23	1.36
Temperaure ) feet Ethant	77	. <del>1</del> 2	2 2	2 92	23	77	<b>8</b>	82	<b>3</b> 6	# :	<b>1</b> 2	3 =	13	15	23	ت	<b>7</b>	<b>13</b> 2	\$ 2	2	12	Ħ	<b>Z</b> ;	4 7	9 2	2	ន	z	<b>X</b> :	2 5	3 7	ជ	<b>=</b> 1	<u>.</u>	3 2	ង	76	92	<b>X</b> \$	J 7.	3 to	2	91	18	œ ;	17
Tempera Influence degree C	7.1	: 5	; ≊	2	র	52	2	36	22	<b>n</b> :	2 2	9 9	17	18	2	: 23	ដ	2 1	a 2	: ::	2	11	11	<b>⇒</b> ≎	9 2	ដ	8	76	<b>X</b> 8	3 8	: =	16	<b>£</b>	2 ;	7 7	: 1:	7.7	11	2 5	3 8	3 €	<u> 2</u>	21	21	<b>7</b> 5	77
lativest ROD mg/L	112	128	155	<del>8</del>	128	8	38	21	92	2 :	5	£ §	<b>%</b>	46	<b>X</b>	8 :	<b>K</b> !	3 5	\$ 5	8	108	121	₹ <b>?</b>	3 <u>c</u>	18/ 18/	179	72.	8	<b>%</b> 8	2 8	2 12	8	ឌ	≽ ≩	<u> </u>	<b>3</b>	157	148	3 ž	<u> </u>	3 53	114	116	108	27 ;	751
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Dete	Jan 91	Feb 91	March 91	April 91	May 91	June 91	July 91	Aug 91	Sept 91	ಷ ಶ :	Nov 51	Jen 92	Feb 92	March 92	April 92	May 92	June 92	July 32	Sent 92	22	Nov 92	Dec 92	Jan 53	Moret of	Anril 93	May 93	June 93	July 93	Sept 93	S S	Dec 33	Jan 24		Mar X	Apr 24	Jun	<b>J</b>	Aug St	s s	z z	Dec 20	Jan 95	Feb 95	Mar 95	Apr 95	May 52

Using the same flow configurations as mentioned above but doubling the flow rate, it can be seen from the results in Table 4-3 that with the proper modifications the lagoon system is capable of producing an excellent effluent. By tripling the design flow rate and with the correct combinations of complete and partial mix cells, the system can be modified to produce an effluent BOD concentration of 30 mg/l or less.

With relatively simple modifications in the existing lagoon system, it is possible to significantly increase the flow rate to the system and produce an acceptable effluent BOD concentration. Modifications of the settling ponds to incorporate multiple drawoff structures and multiple drawoff levels and the utilization of the proper hydraulic residence time should improve the TSS effluent quality.

# Oxygen Requirements

Complete Mix Cell. Determination of the oxygen requirements for the complete mix pond system were verified using Equation 16 shown below.

$$N = \frac{N_a}{\alpha \left[ \frac{C_{SW} - C_L}{C_S} \right] 1.025^{T_w - 20}}$$

(16)

where

- N = equivalent oxygen transfer to tap water at standard conditions, kg/hr
- $N_a = oxygen$  required to treat the wastewater, kg/hr (usually taken as 1.5 x the organic loading entering the cell)
- $\alpha$ = (oxygen transfer in wastewater)/(oxygen transfer in tap water) = 0.9
- $C_L = minimum \ DO \ concentration \ to \ be \ maintained \ in \ the \ wastewater, assume 2 mg/l$
- $C_s = oxygen$  saturation value of tap water at 20°C and one atmosphere pressure = 9.17 mg/l.
- T<sub>w</sub> = wastewater temperature, °C. Assume 23 °C for checking.

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

 $\beta$  = (wastewater saturation value)/tap water oxygen saturation value) = 0.9

Css = tap water oxygen saturation value at temperature  $T_w$ , (see Standard Methods or standard textbooks).

P = ratio of barometric pressure at the pond site to barometric pressure at sea level. (assume 1.0 for an elevation of 100 m)

The design organic load in the influent wastewater is:

$$(C_0)(Q) = (260 \text{ g/m}^3)(65,102 \text{ m}^3/\text{day})(\frac{\text{day}}{24 \text{ hr}})(\frac{\text{kg}}{1000 \text{g}}) = 705 \text{ kg/h}$$

The oxygen demand is assumed to be 1.5 times the organic loading, hence

$$N_{a1} = (1.5)(705 \text{ kg/hr}) = 1,058 \text{ kg/h}$$

Use Eq. 16 to calculate equivalent oxygen transfer.

$$N = \frac{N_a}{\alpha \left[ \frac{C_{SW} - C_L}{C_S} \right] 1.025^{T_w - 20}}$$

$$C_{sw} = (\beta)(C_{ss})(P) = (0.9)(8.68 \text{ mg/I})(1.0) = 7.81 \text{ mg/I}$$

$$N_1 = \frac{1058}{0.9 \left[\frac{7.81 - 2.0}{9.17}\right] (1.025)^{23-20}} = 1723 \ kg/h \ of \ oxygen$$

A value of 1.9 kg  $\rm O_2/kWh$  (1.4kg/hp/h) is recommended for estimating power requirements for surface aerators.

The total power for surface aeration in the complete mix cells:

$$(1723 \text{ kg/h of } O_2)/(1.9 \text{ kg/kWh of } O_2) = 907 \text{ kW } (1,216 \text{ hp})$$

These surface aerator power requirements must be corrected for gearing and blower efficiency. Assuming 90 percent efficiency for both gearing and blowers, the total power for surface aerators in the complete mix cells would be 907 kW/0.9 = 1,008 kW (1,351 hp) or 504 kW (676 hp) per complete mix cell. Eleven 60 hp aerators were installed in each complete mix cell. This is excellent agreement considering that only minor changes in assumptions could result in differences greater than 16 hp.

Partial Mix Cells. An estimate of the oxygen transferred by the two 20 hp aerators in each "partial mix" cell can be made as follows.

Aerators transfer approximately 1.9 kg O<sub>2</sub>/kWh

Total of 40 hp x 0.7457 = 30 kW

Oxygen transfer = 30 kW x 1.9 kg  $O_2/kWh$  = 57 kg Oxygen per hour

BOD loading to "PM" cells:

74 mg/l x (15.75 MGD/2) x 8.34 = 4,860 pounds per day = 2,209 kg/day = 92 kg/h x 1.5 = 151 kg/h

Therefore, the aerators in the "PM" cells are designed to provide approximately 38 % of the design load applied to the first "PM" cell. BOD removal in the "PM" cells was not considered in the design calculations to predict the effluent quality, but aeration was provided to maintain aerobic conditions in the settling ponds. In brief, the PM cells are not true partial mix lagoons.

## PERFORMANCE OF SYSTEM

Influent and effluent data are available for the NIWTP although there are several components to the system. Because of the lack of intermediate performance data, it is impossible to determine the influence of the performance of the lagoon system on the final effluent quality. It is unfortunate that intermediate data were not collected making it is impossible to determine the efficiency of the various operations and processes. For example, it would be beneficial to know how well the automatic backwash filters are removing the algae from the pond effluent. All laboratory and full-scale performance studies have shown that regular granular media filtration without chemical addition can remove about one-third of the algae in pond effluents. The designers expected to achieve far better results. It would be interesting to know the basis for the design of the automatic backwash filters.

Extensive operating data were compiled by the staff and were made available on computer disk for 1991 through May 1995, and this information is shown in monthly summary reports in Appendix A. Influent and effluent flow rates for 1991-1995 are shown in Figure 4-2. As would be expected in a system receiving significant quantities of storm runoff, the flow rate is higher during the rainy season (winter and spring). The influent and effluent BOD concentrations tend to be higher during the low flow periods, and the mean concentration has increased gradually over the past five years (Figure 4-3). Although the mean BOD has increased with time, the mean influent TSS concentration has tended to decrease and has become less sporadic during the past year (Figure 4-4).

In general, the effluent fecal coliform numbers have remained essentially constant but with occasional excursions above 200 organisms per 100 mL (Figure 4-5). The occasional excursions probably are attributable to relatively high TSS concentrations which interfere with the UV disinfection process.

There appeared to be no relationship between the effluent BOD concentration and the effluent water temperature (Figures 4-6 and 4-7). This is not surprising because of the filtration and disinfection (chlorination and ultraviolet light) steps in the process train. Had intermediate performance data been available, it is likely that BOD reduction would be related to temperature.

Figure 4-2. Nogales International Wastewater Treatment Plant Influent and Effluent Flow Rates

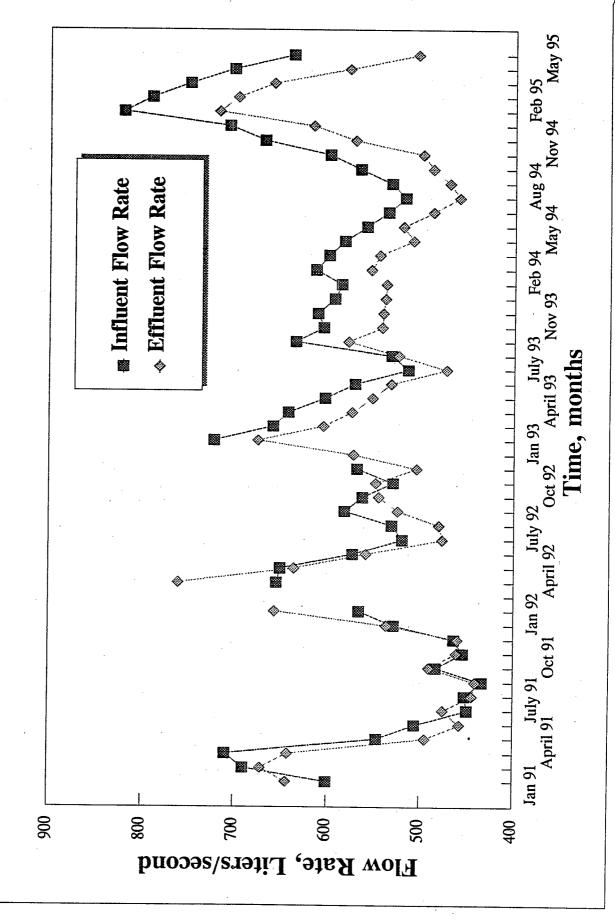
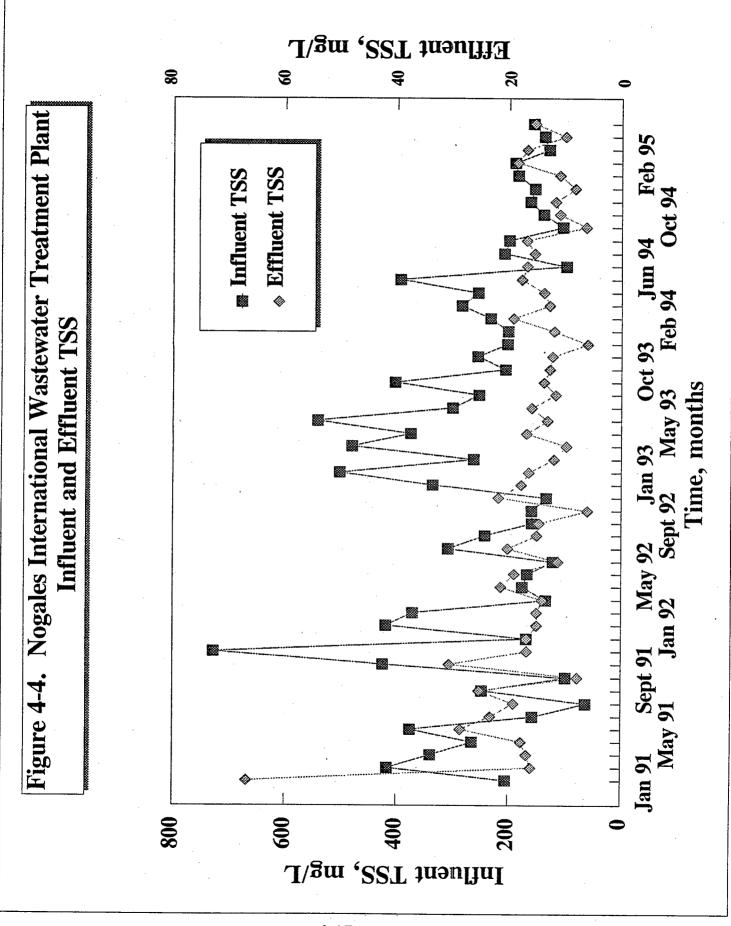
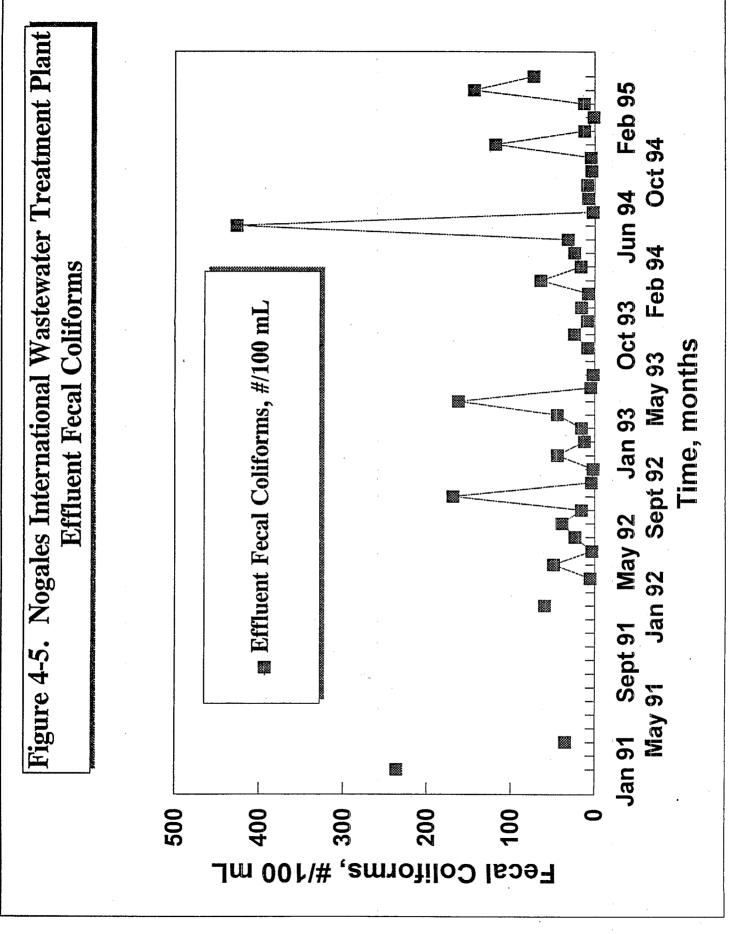
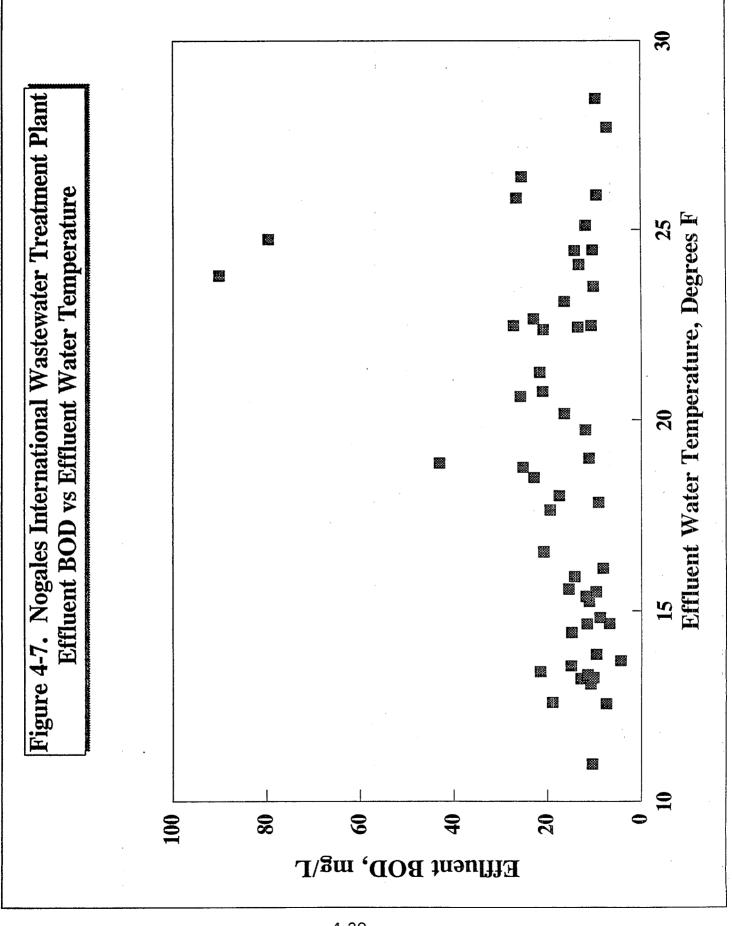


Figure 4-3. Nogales International Wastewater Treatment Plant **Feb 95** Jun 94 Feb 94 Oct 93 May 93 Influent and Effluent BOD ime, months Jan 93 **■** Influent BOD 91 May 92 Jan 92 Sep Sept 91 May 91 Jan 91 **20** 150 100 200 BOD, mg/L





Eff. Water Temp., deg. C 30 25 Feb 95 Figure 4-6. Nogales International Wastewater Treatment Plant Water Temp. Effluent BOD and Temperature Oct 93
May 93 BOD 1 May 92 Jan 93 Jan 92 Sept 92 N Sept 91 May 91 100 8 9 <del>6</del> 20 Effluent BOD, mg/L



# CHAPTER 5

REYNOSA, TAMAULIPAS, MEXICO WASTEWATER TREATMENT SYSTEM

### **CHAPTER 5**

# REYNOSA, TAMAULIPAS, MEXICO WASTEWATER TREATMENT SYSTEM

The Reynosa wastewater treatment plant consists of one primary lagoon (anaerobic) followed by five parallel facultative lagoons with three lagoons in series. A flow diagram for the facility is shown in Figure 5-1. It was originally planned to convert the primary lagoon into an aerated lagoon in 1992; however, there was no aeration at the time of the site visit. Information about the depths and volumes of the facility were not available.

The average flow rate into the system is estimated to be approximately 27 MGD which overloads the facility. The hydraulic detention time is estimated to be less than four days with all lagoons in operation. Primary treatment is the best that the system can deliver. The discharge requirements for the Rio Grande/Rio Bravo of 20 mg/l of BOD and TSS cannot be met by the system.

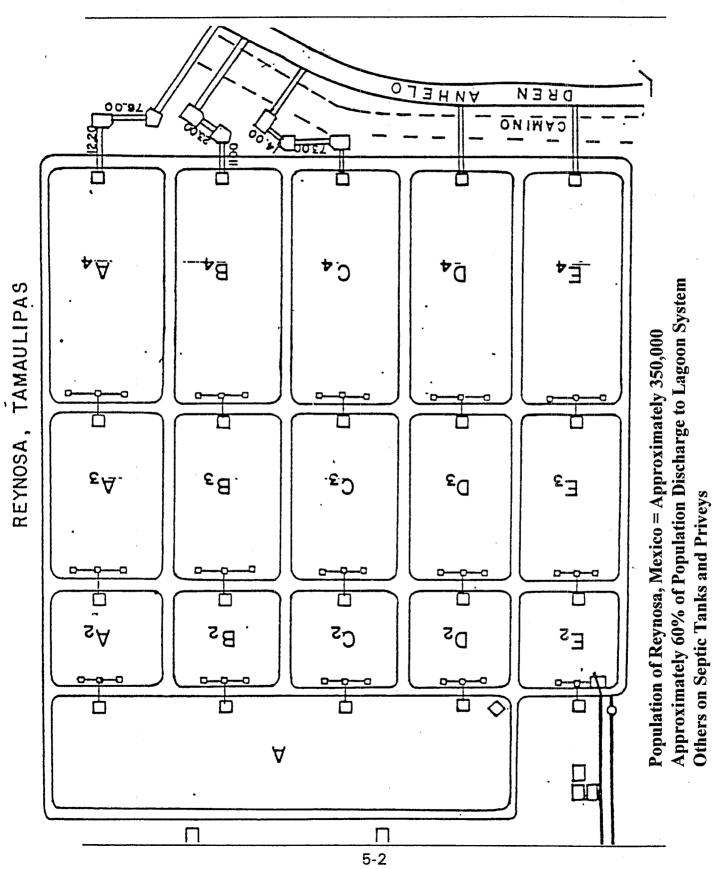


FIGURE 5-1 REYNOSA, MEXICO WASTEWATER STABILIZATION LAGOONS.

# CHAPTER 6 OTHER UPGRADING TECHNOLOGIES

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### **CHAPTER 6**

### OTHER UPGRADING TECHNOLOGIES

In addition to the in-pond approaches discussed in other sections there are several innovative and alternative technologies that appear to have potential for upgrading the effluent quality from these lagoon systems on the US-Mexico border. These include, but are not limited to, constructed wetlands, land application of lagoon effluent, and the use of a floating mat of duckweed plants on some of the lagoon cells. A brief description of each of these technologies is included below. A detailed evaluation of these and other possible technologies is beyond the scope of this effort.

### **CONSTRUCTED WETLANDS**

Constructed wetlands are finding increasing use for treatment and polishing of domestic, municipal and industrial wastewaters. A recently prepared database indicates over 1000 such systems in operation around the world, with the majority in the United States.

There are two basic types of constructed wetlands in general use. One type is similar to a natural marsh with emergent vegetation and with the water surface exposed to the atmosphere; this type of wetland is called a Free Water Surface (FWS) wetland. The other type typically uses a gravel bed up to two feet deep with the water surface maintained below the top of the gravel. This type is called a Subsurface Flow (SF) wetland. The same species of emergent vegetation are used on both types of wetlands. The advantages of the SF gravel bed type include no mosquitoes or other insect vectors, no public exposure to the wastewater, and a higher reaction rate for pollutant removal so a smaller sized system than FWS for the same flow rate. The disadvantage of the SF type is the high cost of procurement and placement of the gravel media. As a result, the SF type is generally used on smaller systems where the advantages are critical, such as on-site home systems, parks and public buildings, etc. The FWS concept tend to be the more economical system at flow rates greater than 100,000 gpd, and would therefore be the likely choice if wetlands were selected to upgrade the ponds along the border.

A FWS wetland system for any of the border sites would typically have multiple cells in at least two parallel trains. The system could be designed for just BOD and TSS or could also include nitrogen, phosphorus and fecal coliform removal. For example, a wetland system designed to produce an effluent of 30 mg/l BOD (wetland influent at 50 mg/l) would require about 100 acres of treatment area for the 37.6 MGD design flow for Mexicali BCN in the year

2015. A wetland system designed for lower levels of BOD and/or significant nitrogen or phosphorus removal would have to be correspondingly larger. A wetland system at any of these border sites can be designed for optimization of treatment or with significant habitat values for birds and other wildlife included.

### LAND APPLICATION OF LAGOON EFFLUENT

There are three basic land application technologies in use today: *Slow Rate* (SR), *Rapid Infiltration* (RI) and *Overland Flow* (OF). All three offer the potential of very effective wastewater treatment and two of the three (SR and OF) have the potential for beneficial crop production.

The slow rate (SR) concept is similar to normal agricultural irrigation except in this case lagoon effluent would be used as the water source. Lagoon effluent would be applied at a controlled rate and in amounts related to the type of crop grown and the season of the year. A continuous year-round application may be possible in the border region and a variety of crops would be possible. The SR system is capable of producing very high quality water with respect to BOD, TSS, N, P, metals, and fecal coliforms. Since crop production is an integral part of the process the wastewater application rates are relatively low so several thousand acres of land might be required for the 37.6 MGD flow from Mexicali BCN in the year 2015.

The overland flow (OF) concept appears to be similar to SR, but in this case the applied water flows over the surface of relatively impermeable soils rather than infiltrating into the soils. The higher application rates and steeper slopes on OF systems limit the feasible crops to forage grasses. Excellent removal of BOD, TSS, N, metals and fecal coliforms is possible. The treated water is collected in ditches at the toe of the slope and can be discharged or reused. An OF system for the 2015 design flow for Mexicali BCN might require up to 1000 acres of treatment area.

The rapid infiltration (RI) concept depends on very high hydraulic loadings on very permeable soils. Hydraulic loadings can range up to 2000 gallons per square foot per year. The system usually consists of several sets of infiltration basins which are flooded and then allowed to drain on a regular schedule. Excellent removal of BOD, TSS, N, P, metals, and fecal coliforms are possible. About 200 acres of RI basins might be required for the 2015 design flow at Mexicali BCN; however, algae which is normally present in lagoon effluents can inhibit the infiltration rate in RI systems and would have to be considered prior to consideration of this concept. The presence of algae would not be a constraint on the other land application or wetland concepts.

### **DUCKWEED COVER**

Duckweed (*Lemna sp.*) is a small floating plant which can be observed on many natural ponds, lakes, and also on wastewater treatment lagoons. The individual plants are very small but under favorable growth conditions the plants can form a thick mat which may then cover the entire pond surface. This mat is very susceptible to wind forces and is typically blown to the windward side of the pond or lagoon. If the mat of duckweed plants could be permanently retained on the water surface then algae growth would be suppressed and lagoon effluent quality for both BOD and TSS would be improved.

A technique for holding the duckweed plants in place is offered commercially by the Lemna Corporation. They manufacture a set of shallow floating baffles which are anchored to the sides of the lagoon. The baffle grid forms cells which retain the duckweed in place on the pond. It is then necessary to periodically harvest some of the duckweed plants, and the same company offers a floating harvester. Both the harvester and the floating grid are patented but the use of duckweed for this purpose is not. Application to the lagoons on the border would probably require a duckweed mat covering most of the existing cells to achieve desired water quality at the present flow rates. It may also be necessary to add lagoon cells to accommodate future design flows with this duckweed concept. The use of duckweed can improve performance for BOD and TSS but will not provide comparable results for N and P without extraordinary harvesting activity and/or supplemental treatment. Because of the duckweed cover the effluent from these lagoons will be devoid of oxygen and supplemental aeration may be required if effluent DO is a requirement.

## SYNOPSIS OF PERFORMANCE EXPECTATIONS

Tables 6-1 through 6-3 contain a synopsis of the performance expectations for the lagoons and upgrading systems discussed in this section of the report. A summary of the performances expected from the more standard lagoon upgrading techniques such as intermittent sand filters, rock filters, etc. are shown in Chapter 2 of the report.

With the selection of the proper treatment method or combination of treatment methods, it is possible to produce a very high quality effluent that would be acceptable for discharge to streams; groundwater recharge; agricultural reuse; or recreational reuse in parks, golf courses, etc.

Design Features and Expected Performance for Aquatic Treatment Units (Bastian and Reed, 1979; Middlebrooks et al., 1981; and USEPA, 1983) Table 6-1

t > 20 days.Little risk of parasitic infection using effluent in agriculture.Some risk when sludge removed.

Fecal Coliform and virus removal adequate when t > 20 days.

Table 6-1 (Continued)
Design Features and Expected Performance for Aquatic Treatment Units (Bastian and Reed, 1979; Middlebrooks et al., 1981; and USEPA, 1983)

	Effluent Characteristics mg/L	BOD 20-40 TSS 20-60 TN 10-15 (Summer) TN no removal in winter Significant heavy metal and toxic organic removal. Helminth removal adequate with separate settling cell. Some risk when sludge removed. Fecal Coliforn and virus removal adequate when t > 20 days.	BOD 10-30 TSS 10-40 Other factors same as above for facultative or aerated pond.	BOD <30 TSS <30 Other factors same as above for facultative or aerated pond.	BOD < 10 TSS < 10 TP < 5 TN < 5 Other factors same as above for facultative or aerated pond.
	Organic Loading kg/ha-day	50-200	1st Cell = Facultative or Aerated Pond	× 30	<50
	Depth meters	2-6	3.5	<1.5	. ▼ 
Typical Criteria	Detention Time days	7-20	100-200	30-50	9 ^
	Climate Needs	None	None	Warm	Warm
•	Treatment Goals	Secondary, Polishing	Secondary, Storage, Polishing	Secondary	AWT, with Secondary Input
	Unit	Partial-Mix Aerated Pond	Storage, and Controlled-Discharge Ponds	Hyacinth Ponds	Hyacinth Ponds

6-5

Table 6-2 Design Features and Expected Performance for Aquatic Treatment Units (Bastian and Reed, 1979; Middlebrooks et al., 1981; Reed et al., 1984; and Reed et al., 1995)

Effluent Characterisics mg/L	BOD 5-10 TSS 5-15 TN 5-10 Significant heavy metal and toxic organic removal. Helminth removal excellent. Essentially no risk of parasitic infection using effluent in agriculture. Fecal coliform and virus removal excellent.	BOD 5-10 TSS 5-15 TN 5-10 Significant heavy metal and toxic organic removal. Helminth removal excellent. Essentially no risk of parasitic infection using effluent in agriculture. Fecal coliform and virus removal excellent.	BOD 5-40 TSS 5-20 TN 5-10 Significant heavy metal and toxic organic removal. Helminth removal excellent. Essentially no risk of parasitic infection using effluent in agriculture. Fecal coliform and virus removal excellent.
Organic Loading kg/ha-day		200	009
Depth	0.2 - 1	0.1-0.6	0.3-0.6
Typical Criteria Detention Time days	0.1	7-15	3-14
Climate Needs	Warm	None	None
Treatment Goals	Polishing, AWT with secondary input	Secondary to AWT	Secondary to AWT
Unit	Natural Marshes	Constructed Wetlands Free Water Surface	Subsurface Flow

Table 6-3 Terrestrial Treatment Units, Design Features, and Performance (USEPA, 1981; USEPA, 1984; Reed et al., 1995)

Concepts	- Treatment Goals	Climate Needs	Typica Vegetation	Typical Criteria Area, ha required	Hydraulic Loading	Effluent Characteristics
Slow Rate	Secondary or AWT	Warm Seasons	Yes	23-280	m/yr 0.5-6	mg/L BOD < 2
						15S < 2 TN < 3 (Varies with type of crop and management) TP < 0.1 Significant heavy metal
	on .					and toxic organic removal.  Helminth removal excellent.  Essentially no risk of parasitic infection.  Fecal coliform and virus removal excellent.
Rapid Infiltration	Secondary, or AWT, or Groundwater recharge	None	<b>%</b>	3-23	6-125	TSS 2 TN 10 TN 10 TP < 1 (In immediate vicinity of basin; increased removal with longer travel distance.) Fecal Coliform 10 Significant heavy metal and toxic organic removal. Helminth removal excellent. Essentially no risk of parasitic infection using effluent in agriculture. Virus removal excellent.
Overland Flow	Secondary, nitrogen removal	Warm Seasons	Yes	6-40	3-20	BOD 10 TSS 10 (Varies with type of wastewater applied) TN < 10

Significant heavy metal and toxic organic removal.
Helminth removal excellent.
Essentially no risk of parasitic infection using effluent in agriculture. Fecal coliform and virus removal excellent.

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# APPENDIX A

NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT MONTHLY REPORTS

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

# MONTH Y REPORT

PREPAR BY-JOSE MANUE ANEZ APROVED AS TO FORM BY WASTEWATER SUPT, LIND VEGA

CML-COMLETE MIX LAGOON PML:PARTIAL MIX LAGOON P.P:PUMPING PLANT

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MONTH: JANUAR 1991

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

# MONTHLY REPORT

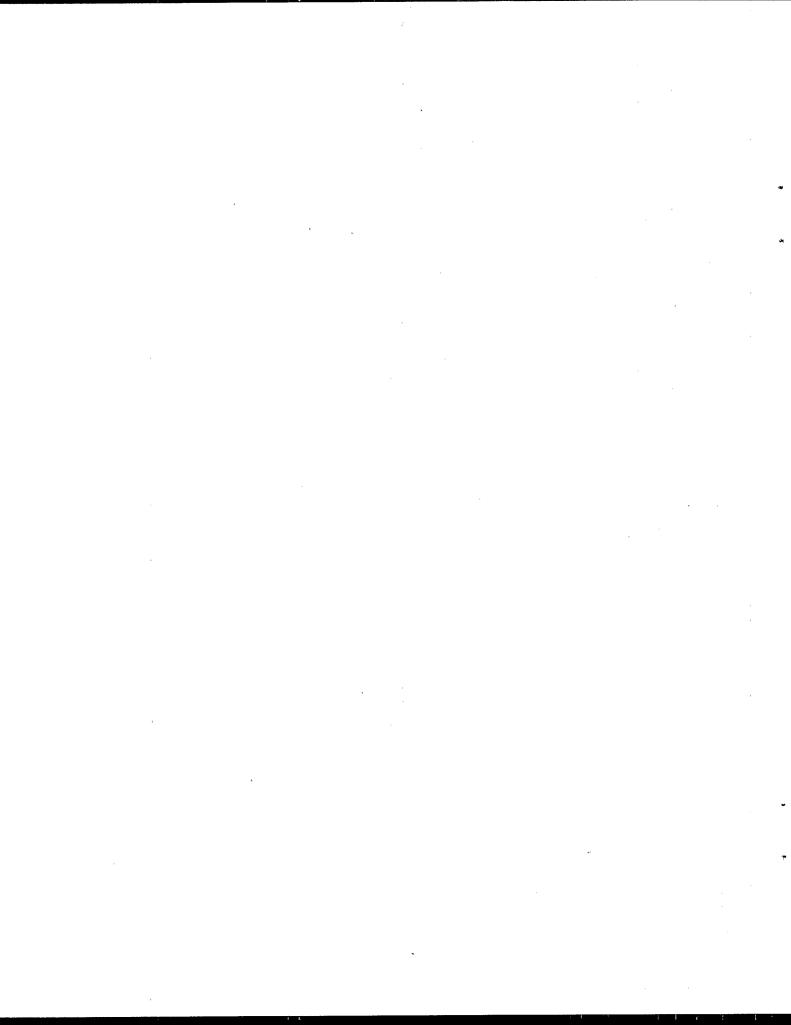
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### MONTHLY REPORT

PREPAR BY: JOSE MAITHT VANEZ APROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CML:COMETE MIX LAGOON PML.PARTM MIX LAGOON P.P.:PUMPING PLANT

Page 1

March 1991

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### MONTHLY REPORT

PREPAR BYJOSE MANUEL ANEZ APROVED AS TO FORM BY WASTEWATER SUPT. LINO VFGA CML.COMLETE MIX LAGOON PAIL.PARTIAL MIX LAGOON P.P.PUMPING PLANT (PUANPING STATION AT NOGALES WASH IS CLOSED DOWN AT THIS TIME)

Page 1

MONTH: April

1991

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ASST.OPERATOR ::	1 : PICKU 1/2 TO	TON		35,484.4	4.4 8		MAIORITOTI			:: 155:191 MG/L :: pH:7.1				
OPERATOR	2 :: GRIT TRUCK	· · · ·	1 :: CHLORINE	RINE BUILDING:		. i.	LECI KICIAN			: DATE:04/11/91		· · · · · · · · · · · · · · · · · · ·		
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April 1991 ::	44.17	71 A.	41 14						:: :: :	: TSS:2 <b>52 MG/L</b> ::pH:7.0				
PML-6 :: PML-7	PML-8	0.0	<u>a</u>	H.	SUSPENDED		SETTLEABLE:: SOLIDS MG/L::		CL2 : RESIDUAL		1			
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COPPER STATE ANALYTICAL LAB INC.

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### MONTHLY REPORT

PREPA :BY JO E MAN LYANEZ APROVED AS TO FORM BY WASTEWATER SUPT, LINO VEGA

CML:COMLETE MIX I AGOON PML:PARTIAL MIX I AGOON P.P:UMPING PLANT

Page

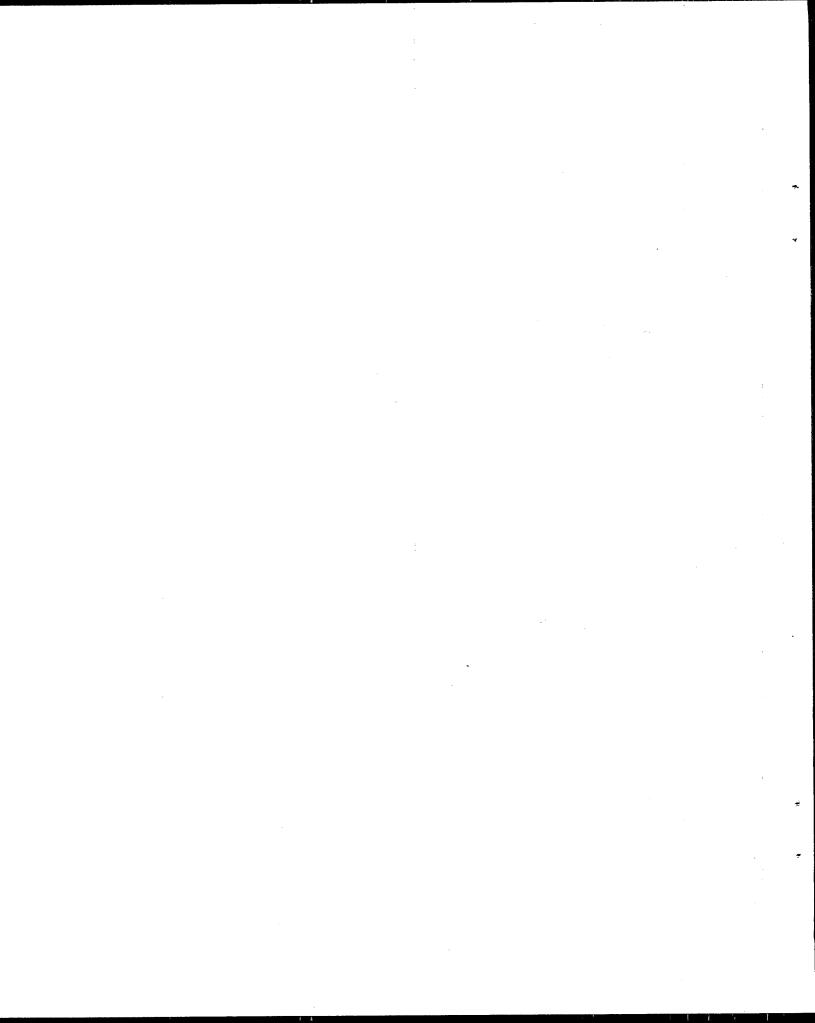
MONTH: May 1991

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LABORER	2 - WATER TRUCK	: .	.i. <u></u>			0	K.W.H.	:: LABORIERS	EKS		, ,	0 :: BOD5:214 MG/L x TSS:110 MG/L	MG/L 3/L			
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### MONTHLY REPORT

PREPAR BY JOSE MANUJE ANFZ APROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA CMI. COMILETE MIX LAGOON PMI. PARTIAL MIX LAGOON P.P. PUMPING PLANT PUMI'NING STATION IS NO LONGER IN SERVICE

Page

Month June 1391

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ULL TIM	IM SERVICES			UTICITIES	UTILITIES AND CHORINE		PERSONNEL SERVICES AND EQUIPMENT	ERVICES AN	DEQUIPMENT				
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CHIEF OPERATOR ::	1:90'D DGE AR S	SKC R	1 : POWE	R CON MPTIO	489600 K.	K.W.H.	CLASSI	ICA I ON	: HRS.THIS :	:: DATE :06/06/91 :: BOD5 :153 MG/L			
ASST.OPERATOR ::	1 PICKU 1/2 TC	10N			47,002.8 4	••••			<i>}.</i>	:: TSS :245 MG/L :: pH :6.9			
OPERATOR	2 GRIT TRUCK	; ;	1 :: CHLO	ORINE BUILDING:			ELECTRICIAL			:: DATE :06/13/91			
LABORER	2 WATER TRUCK		-		0 K	K.W.H.	LABORERS		: 0 ::	:: BOD5 :148 MG/L :: TSS :267 MG/L			
***************************************			:: ::		17.48	•• ••	OVER TIME (OPERATORS)	ERATORS)	39:	:: pH :7.0			
			::::	- INECONSII	5380 I BS	σ.	MOTOR GRADER & OPERATO	R & OPERAT	0	DATE :06/20/91			
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			Here is	ING D E:	05/27/91 TO					:: DATE:06/27/91 :: BOD5:140 MG/L :: TSS:368 MG/L	٠		
June 1991	:/-	¥,	:: <sup>'</sup>	612	6/24/91	••				pH.7.0			
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#### MONTHLY REPORT

PREPAR BY-JOSE MANUE ANEZ APPROVED AS TO FORM BY WASTEWATER SUPT LINO VEGA CML.CCMPLETE MIX LAGOON PML.PARTIAL MIX LAGOON P.P.PUMPING PI ANT FLIMPING STATION IS NO LONGER IN SERVICE

Page 1

lonth July

: DEPT :: AERAT :: :: DEPT :: AERAT :: FEET :: HOURS : DEPT:: AERAT DEPT: AERAT :: DEPT: AERAT PML-2 manana manan  $\begin{picture}(20,0) \put(0,0){$4$} \put(0,0$ :: DEPT :: TEMPE:: AERAT :: FEET :: ATUR :: HOURS OF TOTAL FLOW OF TOTAL FLOW CML-2 NOG. WASH P.P.= AERAT 63% 37% DEPTHETEMPE CML-1 U,S : PLANT : PLANT 9.547 11.037 11.037 11.1037 11.1037 9.823 9.823 9.823 9.739 11.851 10.020 11.140 11.140 11.203 11.203 11.203 11.203 11.203 11.851 M.G.D. 10.867 :: 10.867 :: 10.867 :: 10.787 :: 10.787 :: 10.278 320.091 Z :: STATES : 4,003 4,003 1,172 1,172 1,181 4.088 118.982 FI.OW MEXICO 203.548 9.191 WIND FINITOW, EFFLUENCE DIRECTOR MINIMUM: AVERAG MAXIMU RATURE ·TEMP ..... HER 0.000 : RAIN : FALL : INCHES WEA ..... DATE 

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CLASSI ICA ION HINS, INIS  LECTRICIAN 0  ABORERS 0  ADOTOR GRADER & OPERATO 0  SETLLEABLE B.O.D. RESIDUAL SOLIDS MG/L MG/L 15  EFFLUENT RA : EFFLUEN MINUTES  C.10 103 39 09 9	SUSPENDED   SETTLEABLE   BO.D.   FRESIOUAL   SOLIDS MGIL	DES EQUIPMENT		UTILITIE CONTROL BUILDING	UTILITIES AND CHORINE JILDING	: PERSON: UTILIZE	"PERSONNEL SERVICES AND EQUIPMENT "UTILIZED OT ER THAN FULL TIME	AND EQU		BOUNDARY	λλ.	
ABORERS 0  ABORERS 0  ADTOR GRADER & OPERATO 0  SETLEABLE B.O.D. RESIDUAL SOLIDS MG/L. MG/L  SULDS MG/L. MG/L  15  EFFLUENT :RA : EFFLUEN MINUTES  C.10  C.12  C.12  C.12  C.12  C.13  C.14  10  C.15  C.15  C.15  C.15  C.16  C.16  C.17  C.17  C.18  C.19  C.10  C.19	TECTRICIAN   0   0   0   0   0   0   0   0   0	K C R 1 POWER CON MPTIO	ER CON MPTIO			:: :: ::	ICA		ASTHIS :: DATE:07/03/91 AONTH :: BOD5:45 MG/L :: TSS:102 MG/L			
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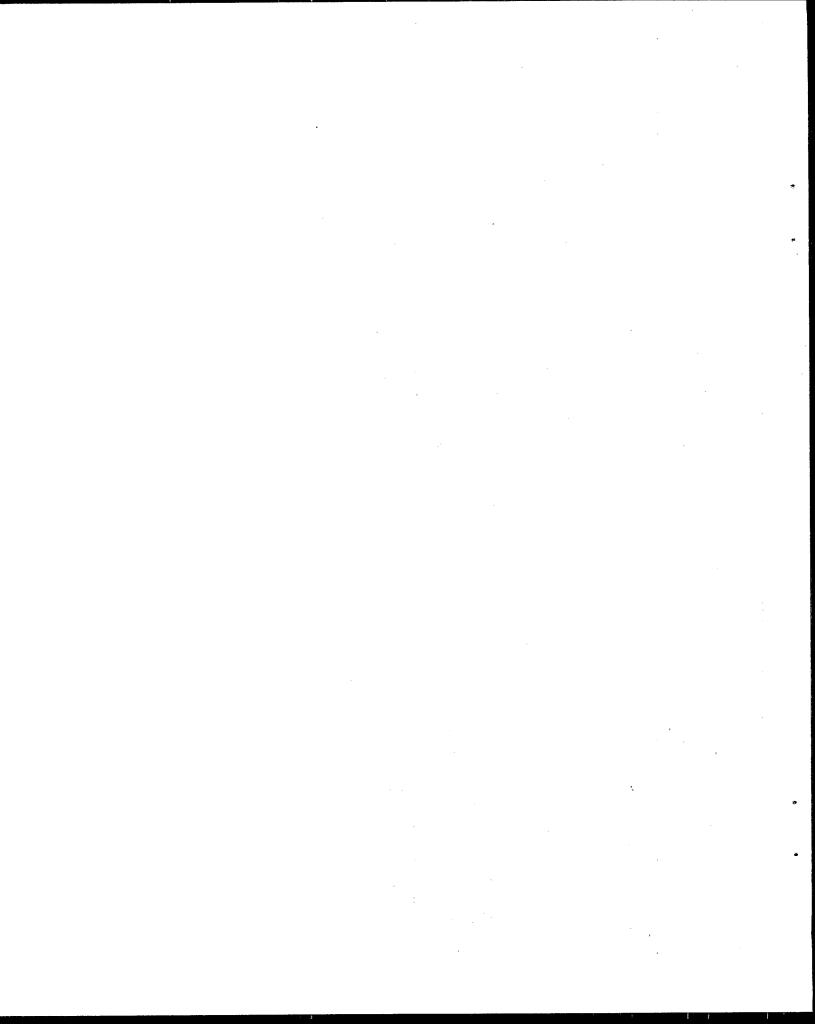
### MONTHLY REPORT

PREPAR RY.JOSE MANUE ANEZ APPROVED AS TO FORM RY WASTEWATER SUPT. LINO VEGA CMI..COMPLETE MIX LAGOON PML.PARTIAL MIX LAGOON P.P.PUMPING PI ANT PUMPING STATION IS NO LONGER IN SERVICE

Page

Month: August 1991

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### MONTHLY REPORT

PREPAR BY:JOSE MANUE ANEZ APPROVED AS TO FORM BY WASTEWATER SUPT. LIND VEGA CML:COMPLETF MIX LAGOCIN PML:PARTIAL MIX I AGOCIN P.P:PUMPING PLANT PUMPING STATION IS NO LONGER IN SERVICE

Page

MONTH: Septembe 1991

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OPERA1OR I ABORFR	2 GRII IRUCK 2 WALLETRUCK		1::CIILO	RINE RUILDING:	0 KWH.		LABORERS OVER TIME (OPERATORS)	ERATORS)	0	:: DATE:09/12/91 :: BOD5:78 MG/L :: TSS:106 MG/L :: pH-7.0				
			CHLO CCHLO	CONSU D XIDE:	11.570 LRS 0 LBS.	W   W	MOTOR GRADER & OPERATO	R & OPERAT	0	DATE:09/19/91 : BOD5:74.3 MG/L : ISS:140 MG/L : pH:7.1				
Page 2 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5 5	en e			NGD E: 08	08/21//91 to 09/25/91					DATE:09/26/91 BODS:174 MG/L TSS 202 MG/L pH:6 8				
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### MONTHLY REPORT

PREPAR BY: RIC ARD 1 ALDWELL APPROVED AS 10 FORM BY WASTEWATER SUPT. LIND VEGA CMI..COMPLETE MIX LAGOON PMI.PARTIAL MIX LAGOON P.P. PUMP ING PLA I PUMPL GSTATIO IS NO LONGER IN SERVICE

Page

MONTH: October 1991

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1.90 B	AR SKC R	R CON MIPTIO	390600 KW.H.	:: CLASSI ICA	NO.	: MONTH ::	:: DATE: 10/03/91 :: ROD5: 90 mg/l			
. <u>=</u>	117 104	2 \$. 31	37,963.4 9				:: TSS: 50 mg/l :: pH: 6.9			
ASST. OPERATOR 1 GRI IRICK		INE &	ULTRY OLET DIS I NFECTION	ELECTRICIAN		:: :::	DATE: 10/11/91			
ODERATORS 2 WALLS TRICK	: :	:: ARFA 1 ·	KWH . 0	::LABORERS		0	:: ROD5: 215 mg/l :: TSS 238 mg/l			
:			9	OVER THAE (OPERATORS)	RATORS)	21 gpH				
LARUKEKS :: 4 . Brusk Hole	****	ŧ		:: MOTOR GRADER & OPERATO	& OPERATO	0	DATE: 10/17/91			
MECHANIC 1		CHI ORINF CONSU	7,300, 0 18S				: BOD5: 240 mg/l			
ELEC I CIAN :: 1.		SULPHUR D XIDE:	0 LBS.				rss. 213 mg/l pH: 6.8			
		UTILI TY CO TS WATER SERVICE	R SERVICE	22.22.7	·		DATE: 10/24/91			
Fage 2		# 535.C	\$ 35.00 11 TO 10/22/91				: 155: 210 mg/l : 155: 210 mg/l : pH: 6.7			
				:		***	DATE: 40/24/04	***** * * * * * * * * * * * * * * * * *		
PML-6 : Pt.11-7 : PML-8		<b>x</b>	SUSPENDED	SETTLEABLE:	B.O.D. MG/L	:4-:: :p-:	DATE: 10/31/91 BOD5; 225 mg/l TSS: 198 mg/l			
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### MONTHLY REPORT

PREPAR BY RIC ARD L. ALDWELL, COMLIANC SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CML;COMPLETF MIX LAGOON PML;PARTIAL MIX LAGOON P.P;PUMPING PLANT PUMPING STATION IS NO LONGER IN SERVICE

Page 1

MONTH: Nov 1991

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## NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

#### MONTHLY REPORT

PREPAR BY RIC ARD L ALDWELL, COMPLIANF SPEC. APPROVED AS 10 FORM BY WASTEWATER SUPT, LIND VEGA CMI.:COMPLETT. MIX LAGOON PMI.:PARTIAL MIX LAGOOH P.P.PUMPING PLANT PUNIPING STATION IS NO LONGER IN SERVICE

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

#### MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LIND VEGA

CML.COMPLETE MIX LAGOON PML.PARTIAL MIX LAGOON P.P.PUMPING PI,ANT PUMPING STATION IS NO LONGER IN SERVICE

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Month January 1992

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## NOGALES INTFRNATIONAL WASTEWATER TREATMFNI PLANT MOGALES INTFRNATIONAL WASTEWATER TREATMFNI PLANT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANF SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

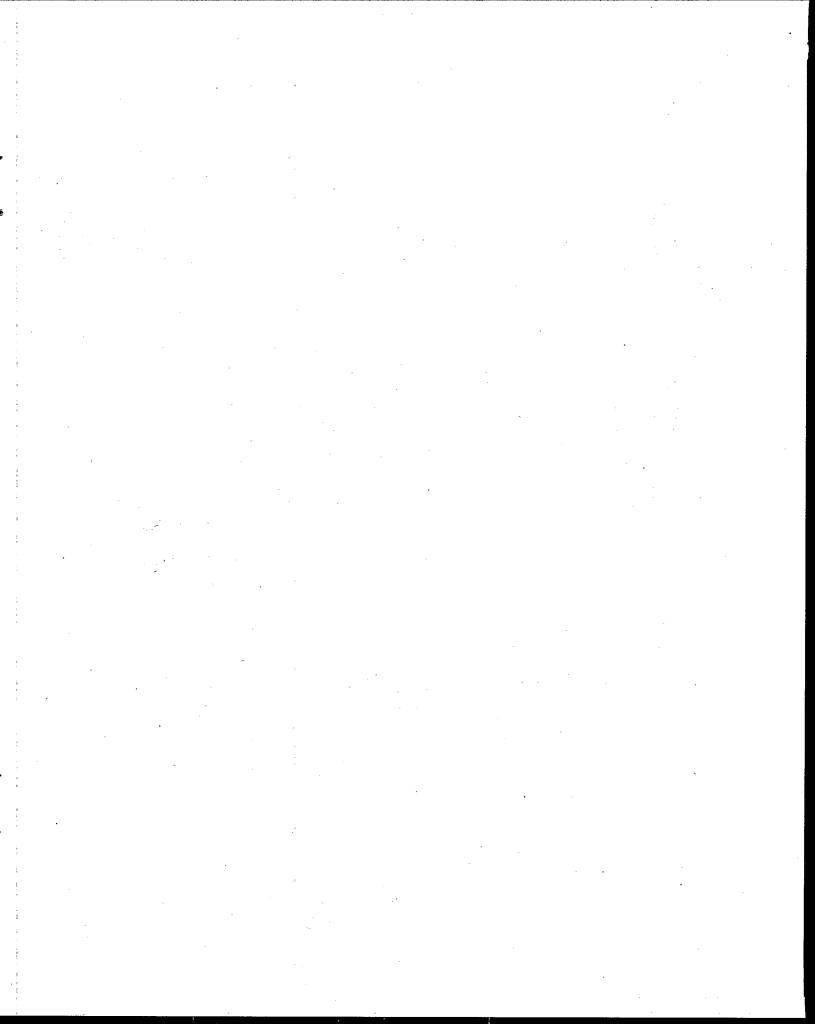
CML:COMPLETF MIX LAGOON PML:PARTIAL MIX LAGOON P.P.PUMPING PLANT PUMPING STATION IS NO LONGER IN SERVICE

Page 1

MONTH: February 1992

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PS.# OF TOTAL FLOW

CML:COMPLETE MIX LAGOON
PML PARTIAL MIX LAGOON
P.S.: NOGALES WAS H PUMPING STATION

MONTHLY REPORT ( ).

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC.
APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

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CY = CLIGHC, YA ARE CHECKED

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CML:COMPLETÉ MIX LAGOON
Phil Partial MIX LAGOON

MONTH: January

PREPAR DY: RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

MONTHLY REPORT

NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

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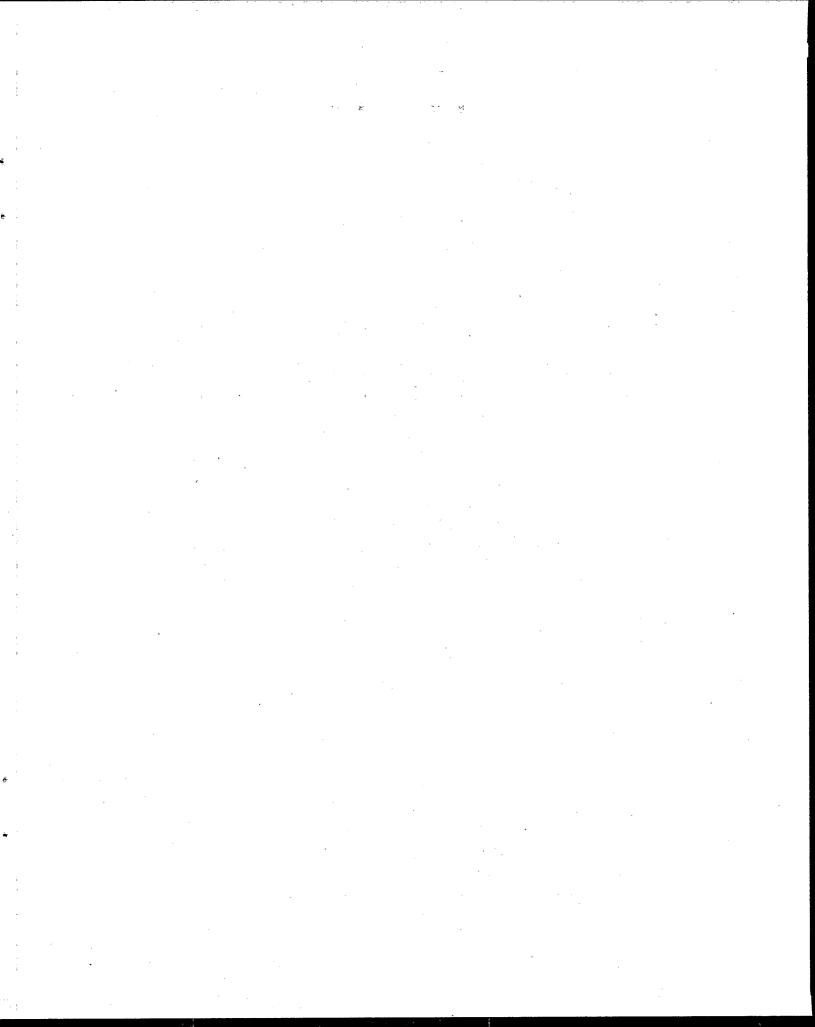
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## NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

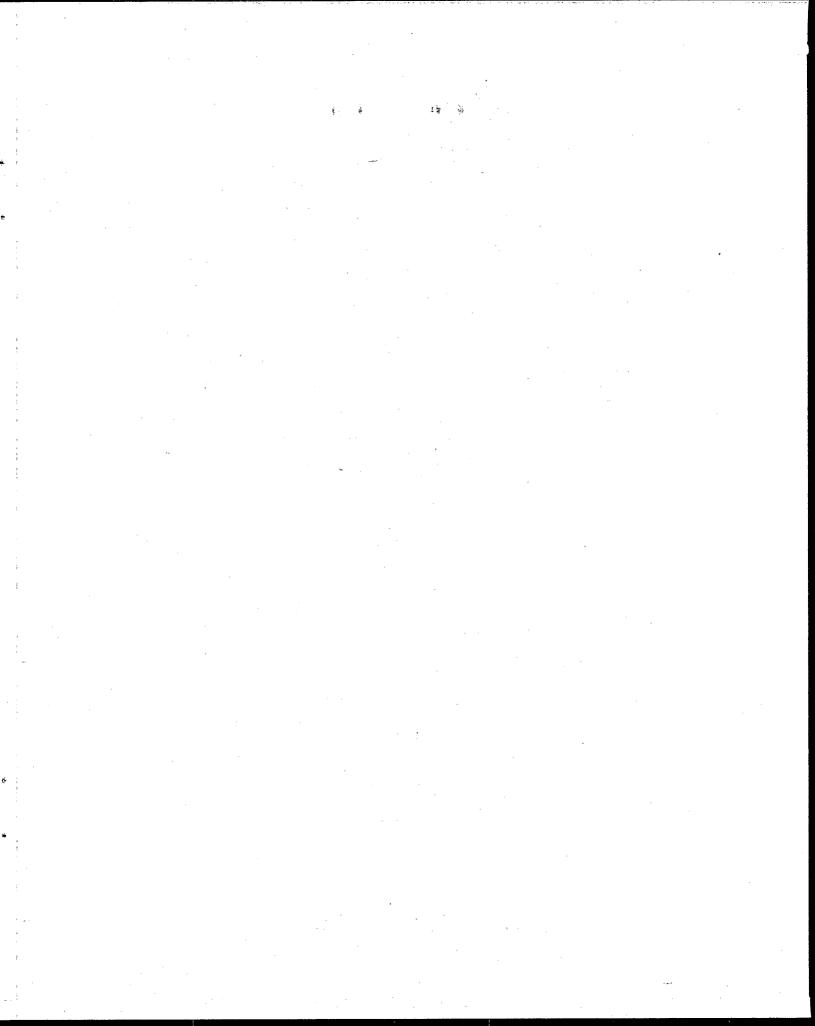
#### MONTHLY REPORT

PREPAR RY RIC ARD L. ALDWELL, COMPLIANE SPEC. AITPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CHI COMPLETE MIX LAGOON
Phil PARTIAL MIX LAGOON

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## NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

MONTHLY REPORT

PRLPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CMI. COMPLETE MIX LAGOON PML PARTIAL MIX LAGOON

MOHTH. Nov. 1992

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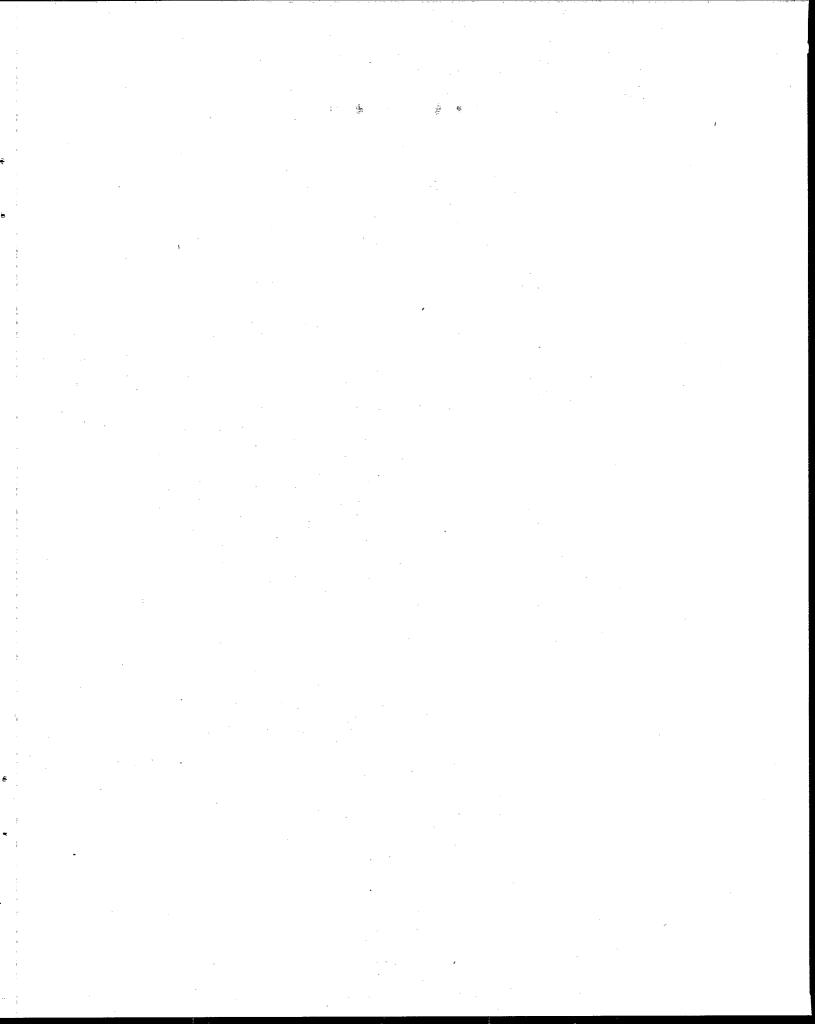
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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT MONTHLY REPORT

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PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CML-COMPLETE MIX LAGOON PML-PARTIAL MIX LAGOON

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# N. N. JALES IN TERNATIONAL WASTEWATER TREATMENT PLANT

#### MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CH, COMPLETE MIX LAGOON
PHI PARTIAL MIX LAGOON

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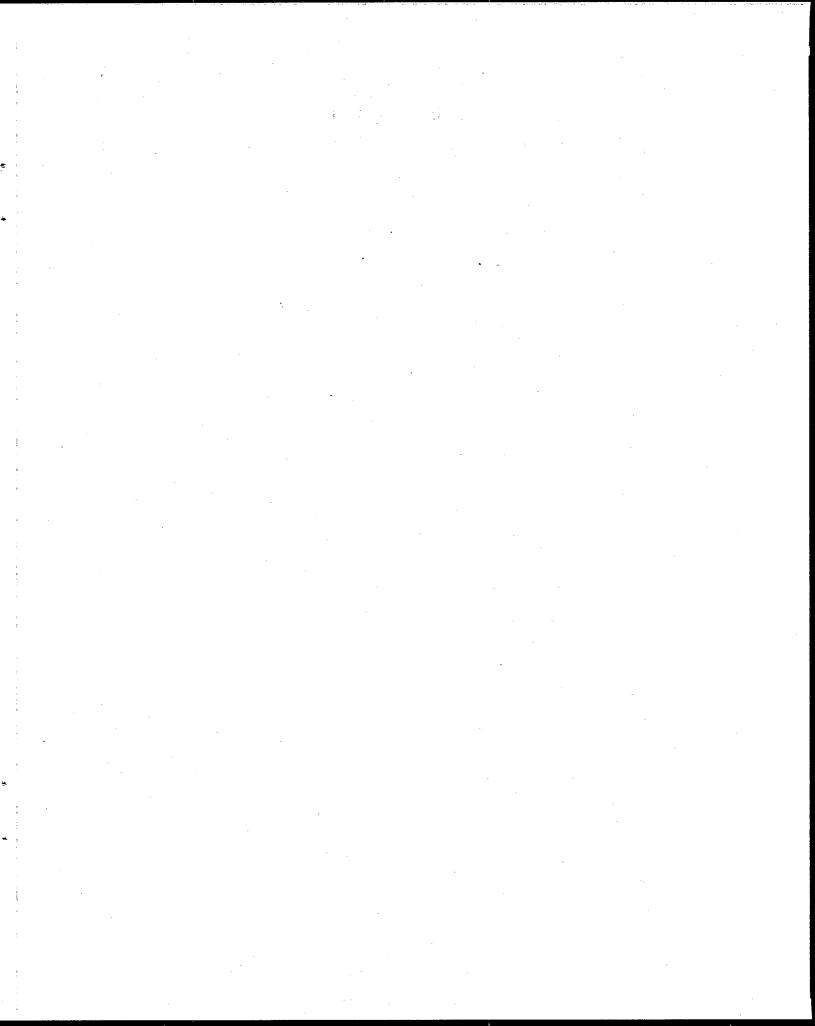
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NOTE PRELIMINARY TREATMENT OFF-LINE FOR CONSTRUCTION

ELECTRICIAN OPERAOR I LABORERS OPERAOR II COMPLIAN: ( SPEC. CHIEF OPERATOR RECHANIC PML-6 " Franklike PERSONNEL . DLPT :: AERATO;: DEPT :: AERATO;: CML- :: CML- :: EFFLUEN:: RA ULL TIM 1 290'D : BACK HOE S GRIT TRUCK ::PICKU 1/2 : WATER TRUCK SERVICES PML-8 DGE AR SKC EQUIPMENT ᅙ D.O. : AREA ::WAT SER I CE: 2 , 423 GALS. ::SER I CE P IOD /28/92 TO ::BILLI NG DA E: /26/92 :: TOT ELEC RIC C :: CHLORINE CONSUM :: CHLO INE & ULTRAVIOLET DISINFECTION :: SULP UR DI XIDE: \* POWER CON MPTIO CONTROL BUILDING #CML- #CML- #EFFLUEN# RAW #EFFLUEN# ъ UTILITIES AND CHIRLORINE STS \$49, . 313 8 2,795 10.560 17.48 49,296.3 . 35 505,80 Ģ :: SUSPENDED 8 FOR \$1,974 21 8/25/ / 92 236 ŝ 196 X X.H BS 88 :: OVER TIME (OPERATORS) ;; SETTLEABLE :: :: MOTOR GRADER & OPERATO :: LABORERS EI ECTRICIAN PERSONNEL SERVICES AND EQUIPMENT IN EFFLUENT : EFF SINFLUEN CLASSI ICATI ON B O D 236 279 262 65 :HRS IHIS MONTH 980 3,400 0 00.55 0 00.05 0 01.5 100 n CPS 0 0% DATE 08/2//92 8OD-5 84 9 mg/l FSS 138 mg/l CEE. RLHOVE 50.7 ; fi.cai ; courora BOUNDARY 5.1 TOURS. . HOURS U:1:2 



# NO ALES INTERNATIONAL WASTEWATER TREATMENT PLANT

### MONTHLY REPORT

PREFAR BY RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT LING VEGA

CLII COMPLETE MIX LÁGOON PAIL PARTIAL MIX LAGOON

Pag: 1

MCHIH August

1992

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# NO MES INTERNATIONAL WASTEWATER TREATMENT PLANT

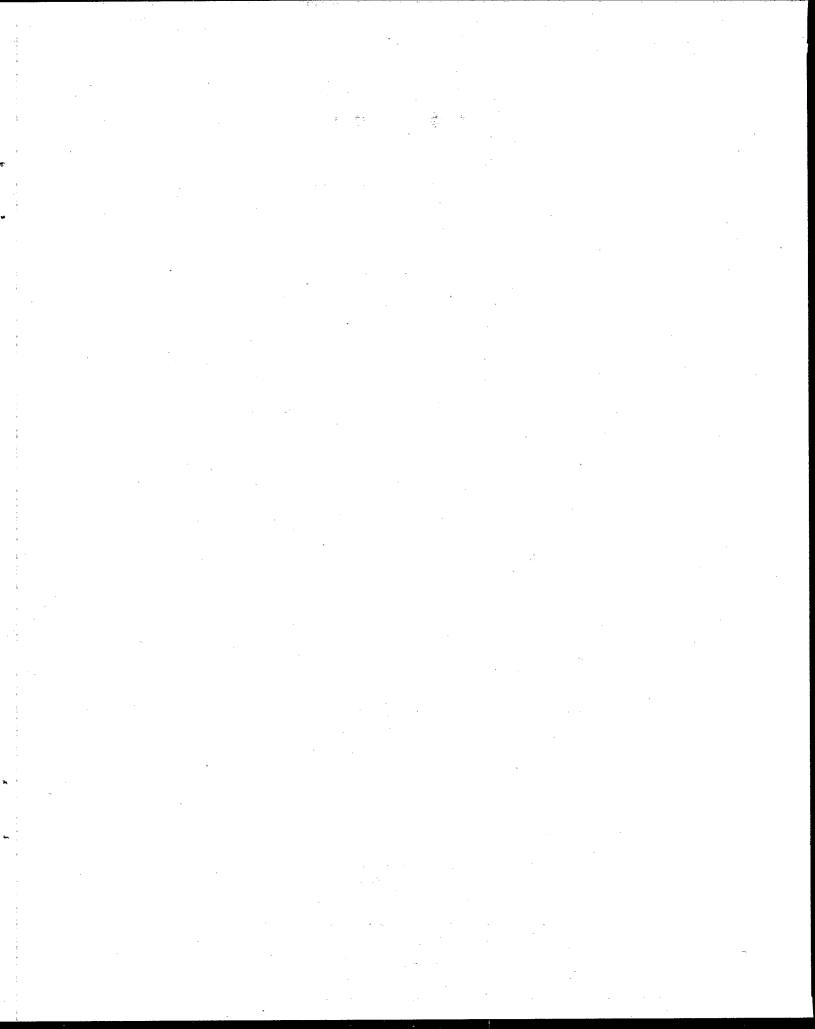
#### MONTHLY REPORT

PREPAR BY RIC ARDL. ALDWELL, COMPLIANE SPEC APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CMI COMPLETE MIX LAGOON PMI PARTIAL MIX LAGOON

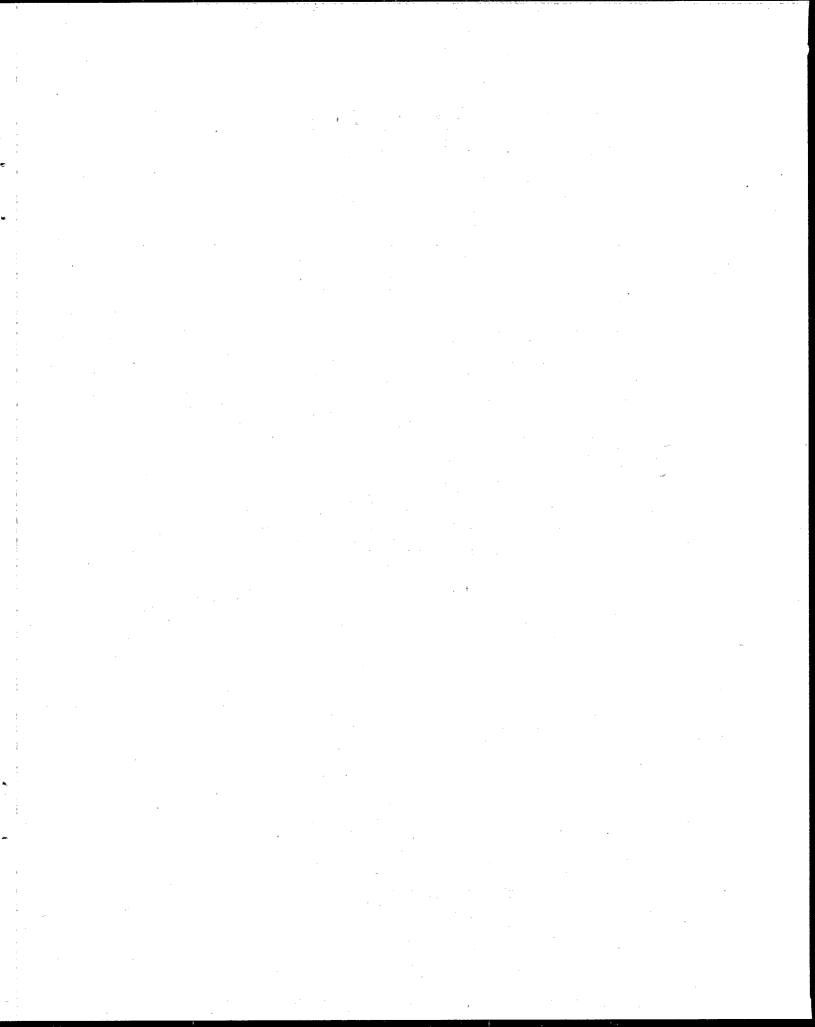
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# NOCALES INTERNATIONAL WASTEWATER TREATMENT PLANT

#### MONTHLY REPORT

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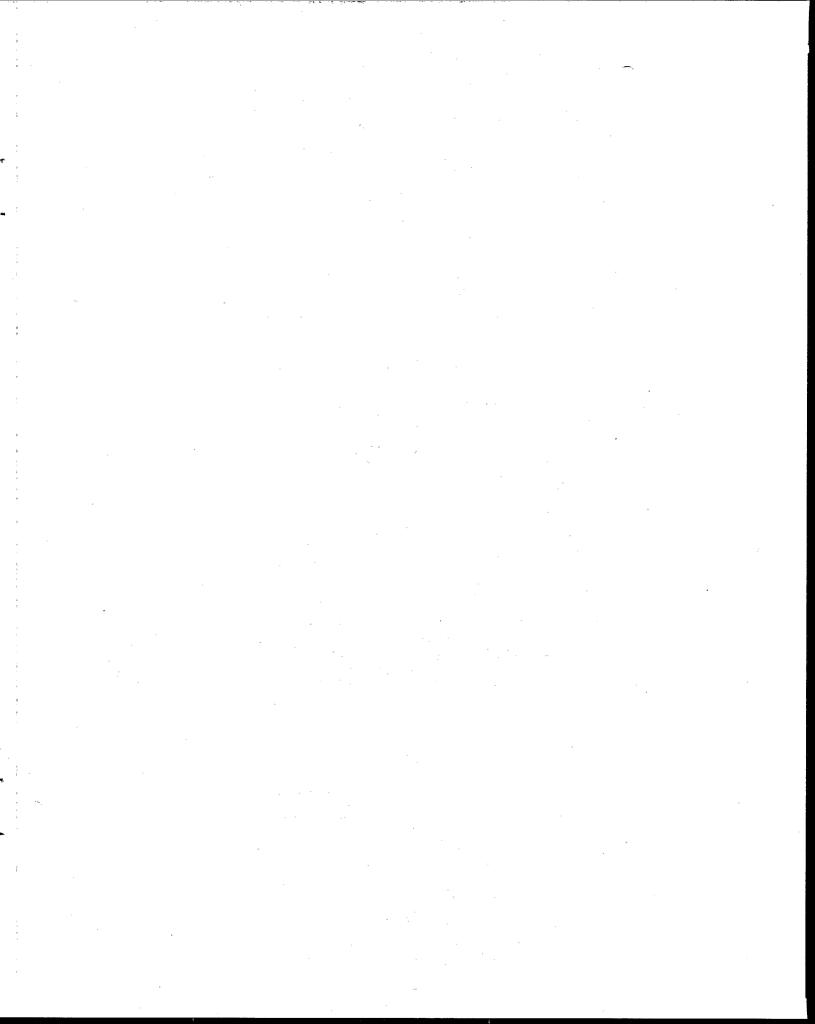
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# NOCALES INTERNATIONAL WASTEWATER TREATMENT PLANT

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#### MONTHLY REPORT

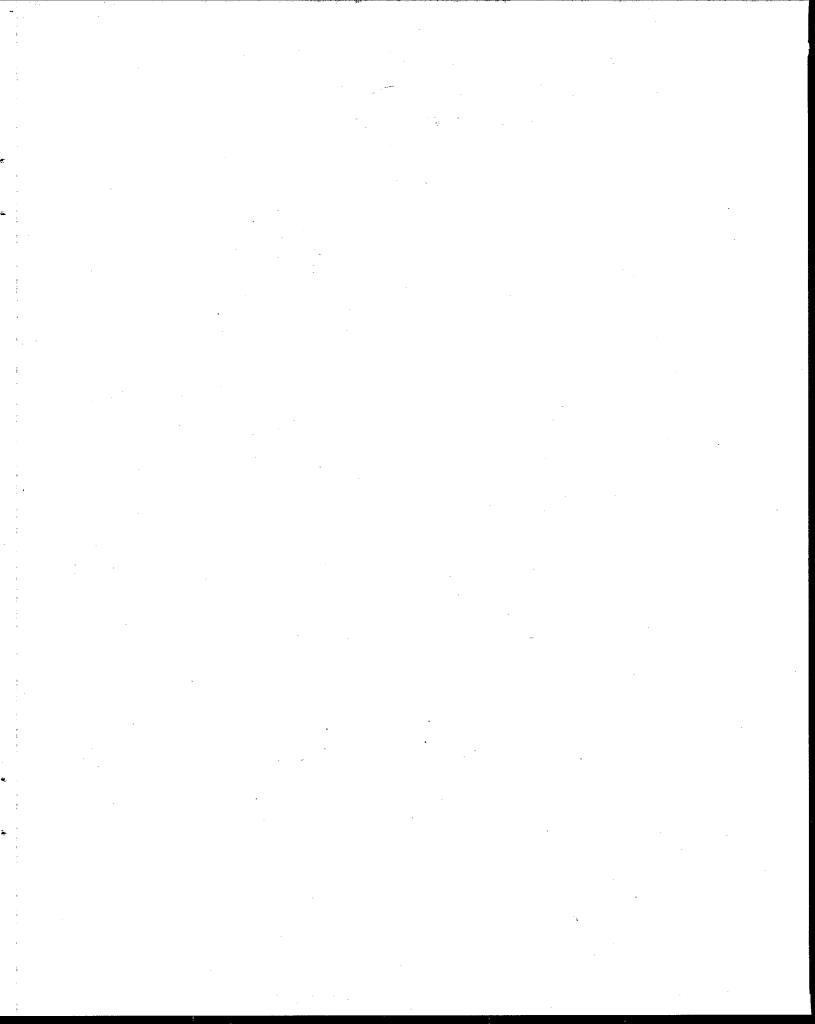
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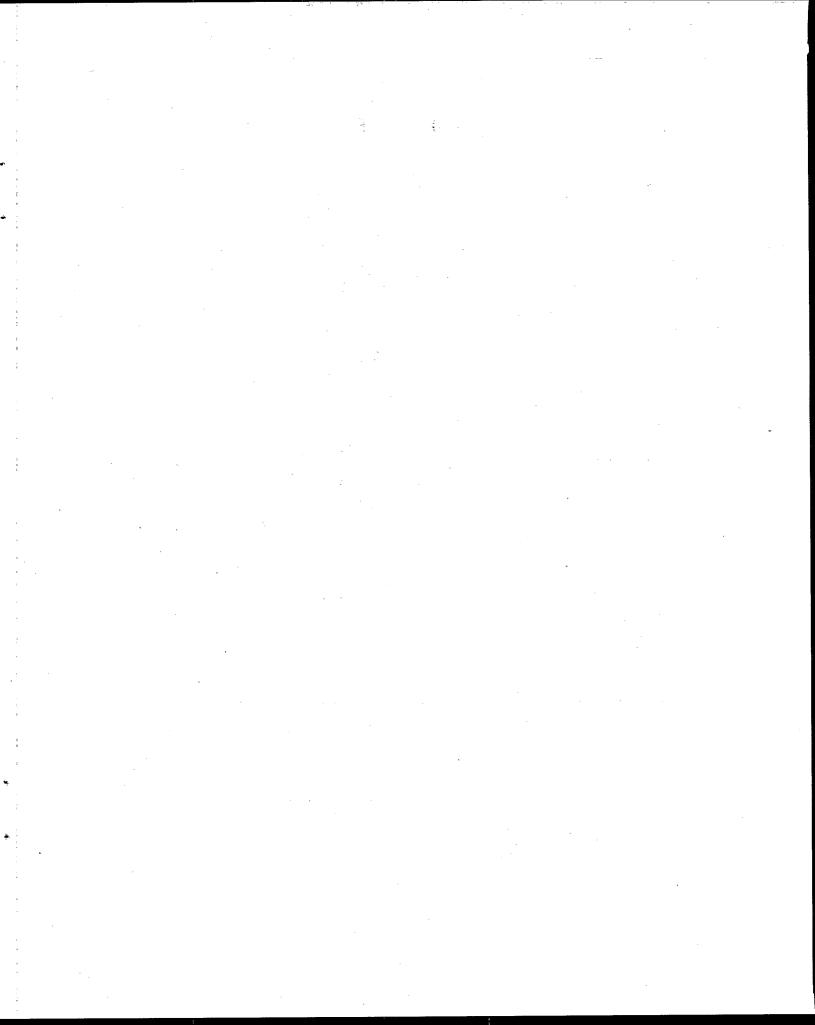
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## NUGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

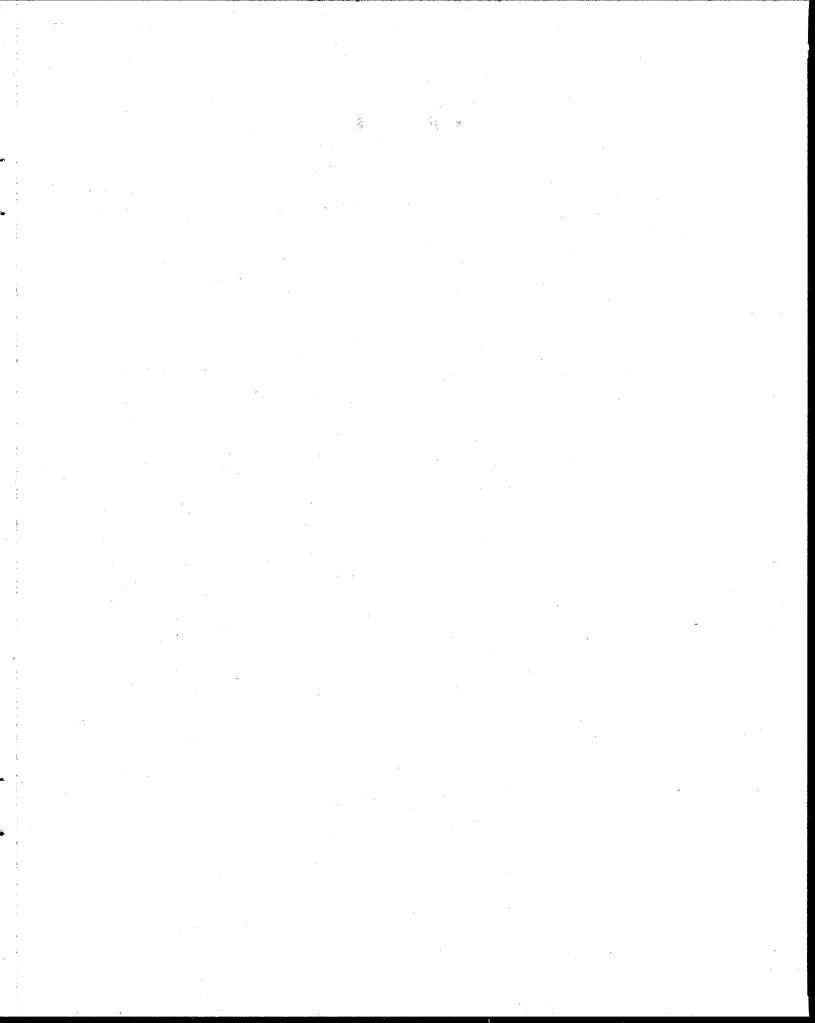
MONTHLY REPORT

PREPARE BY: RIC ARD L. ALDWELL, COMPLIANE SPEC APPROVED AS TO FORM BY WASTEWATER SUPT LING VEGA

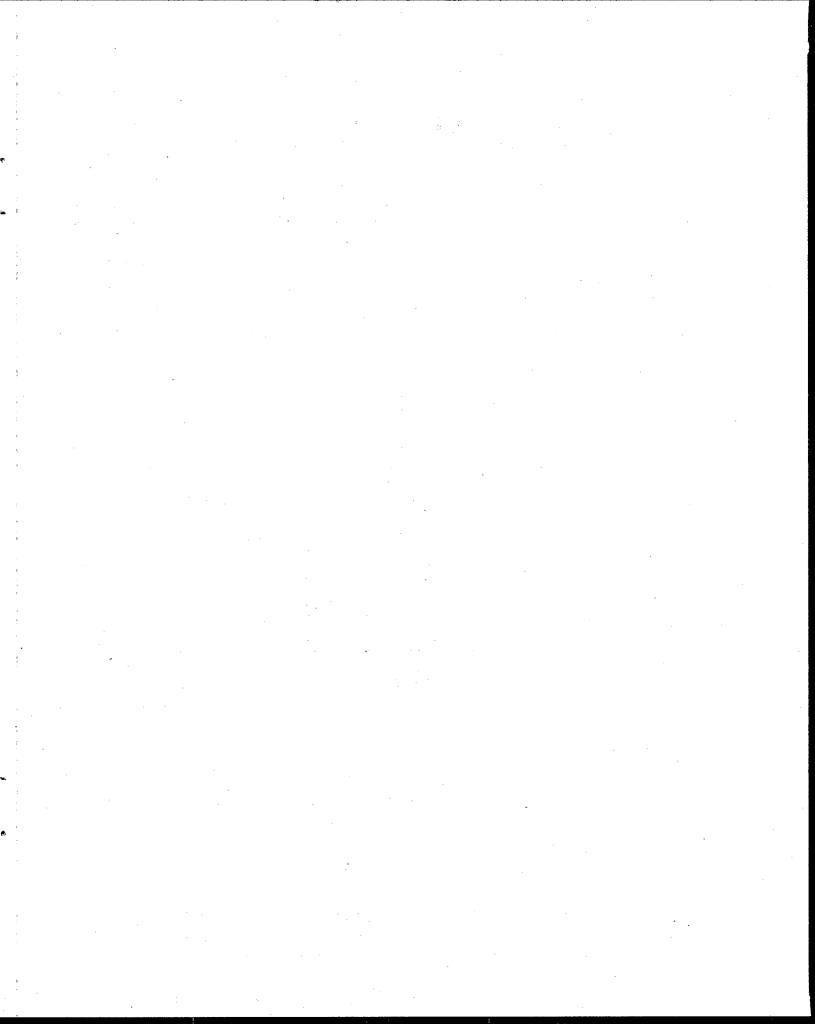
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MOUTH: April 1992

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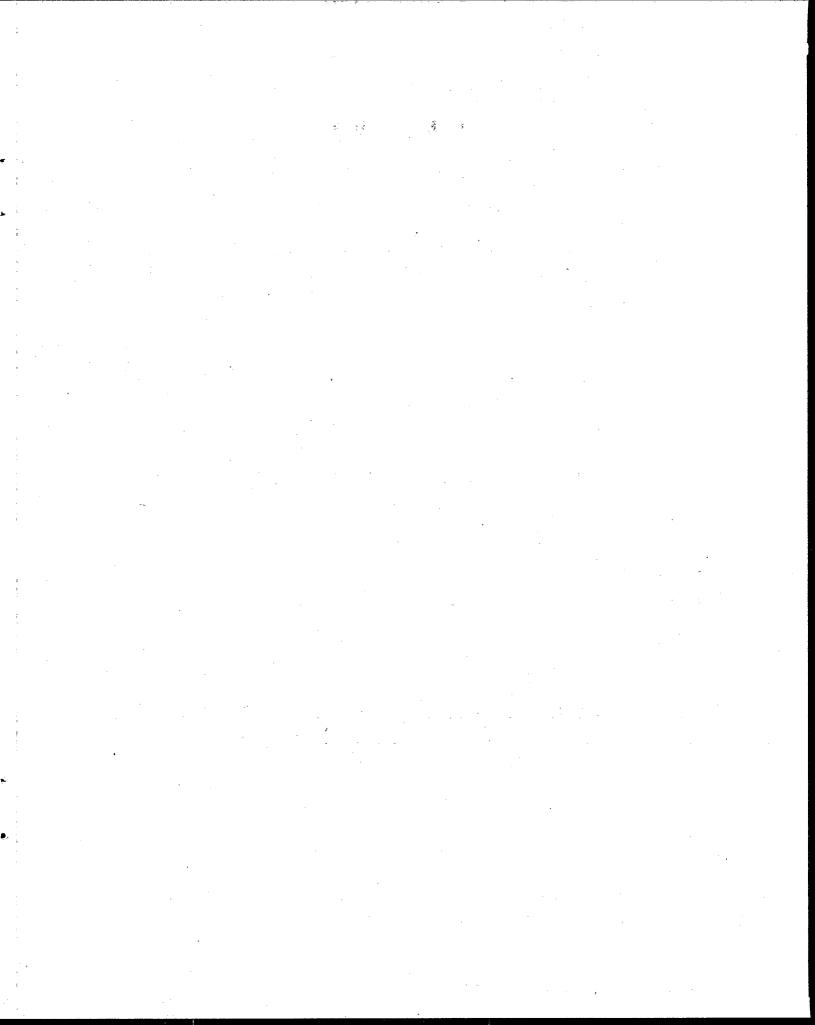
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CML COMPLETE MIX LAGOON
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P.P. PUNNEJING PLANT PUNNEJING STATION IS NO LONGER IN SERVICE

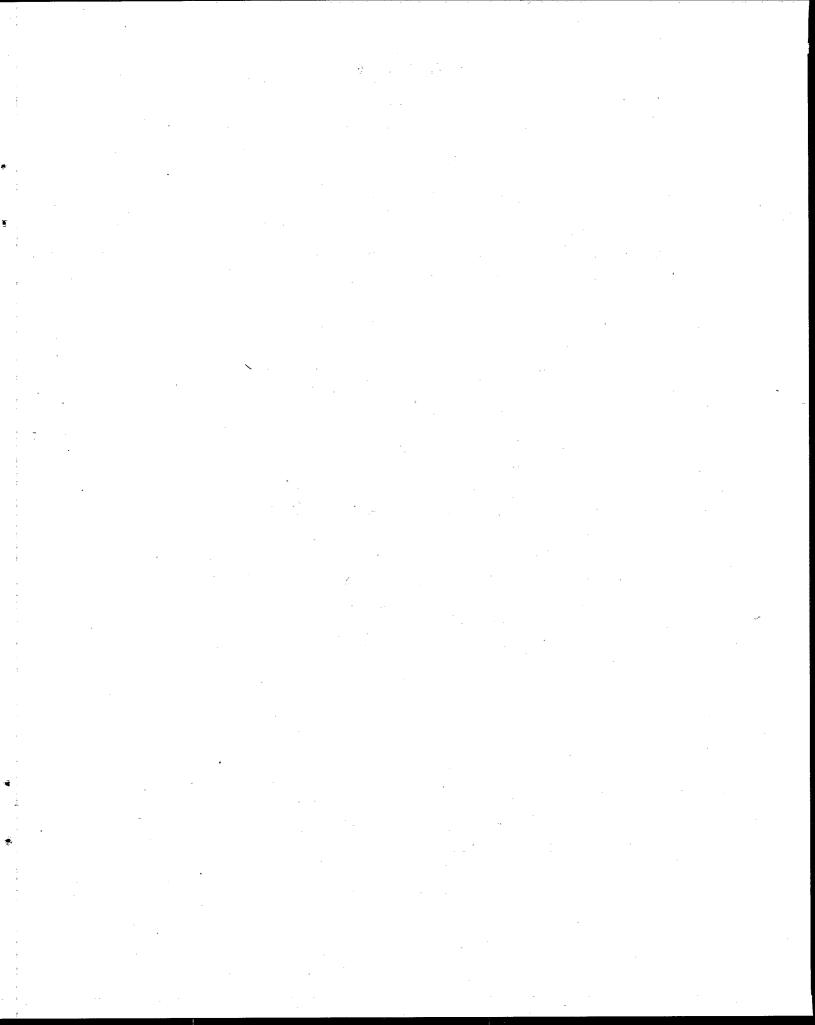
PREPAR BY RIC ARD L. ALDWELL, COMPLIANE SPEC APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

MONTHLY REPORT

NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT



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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

14.

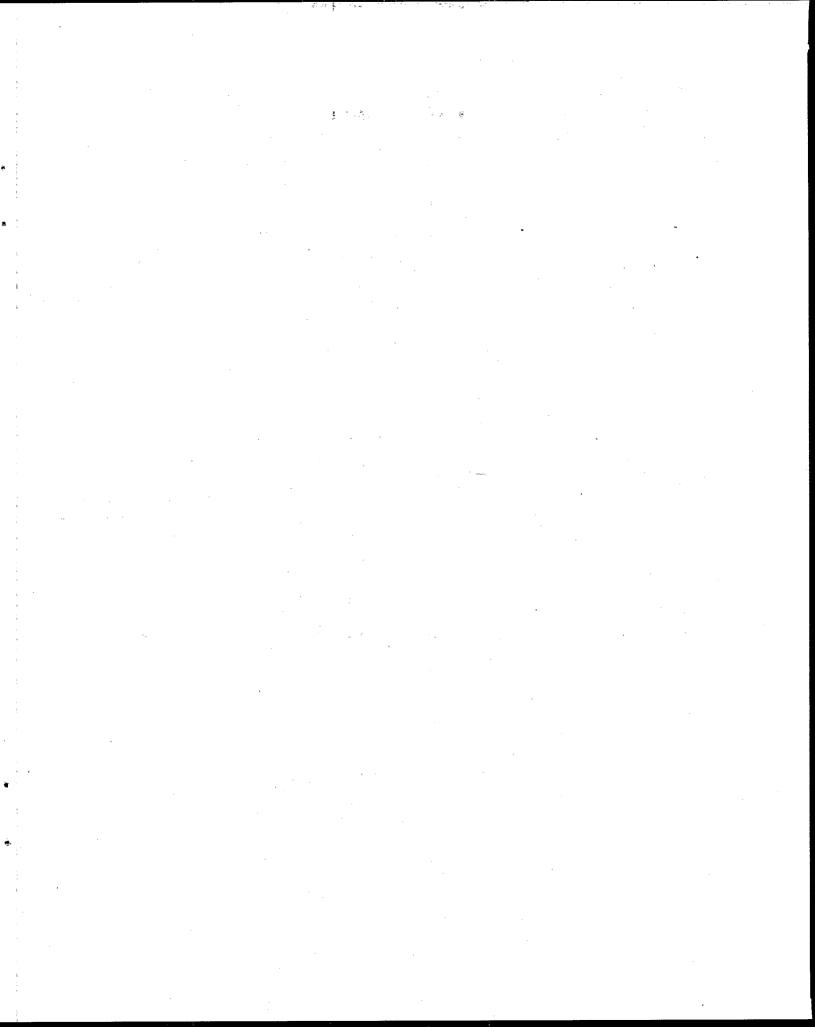
#### MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY; LINO VEGA, PLANT SUPERINTENDENT

CAL COMPLETE MIX LAGOON
PAIL PARTIAL MIX LAGOON

MONTH. January

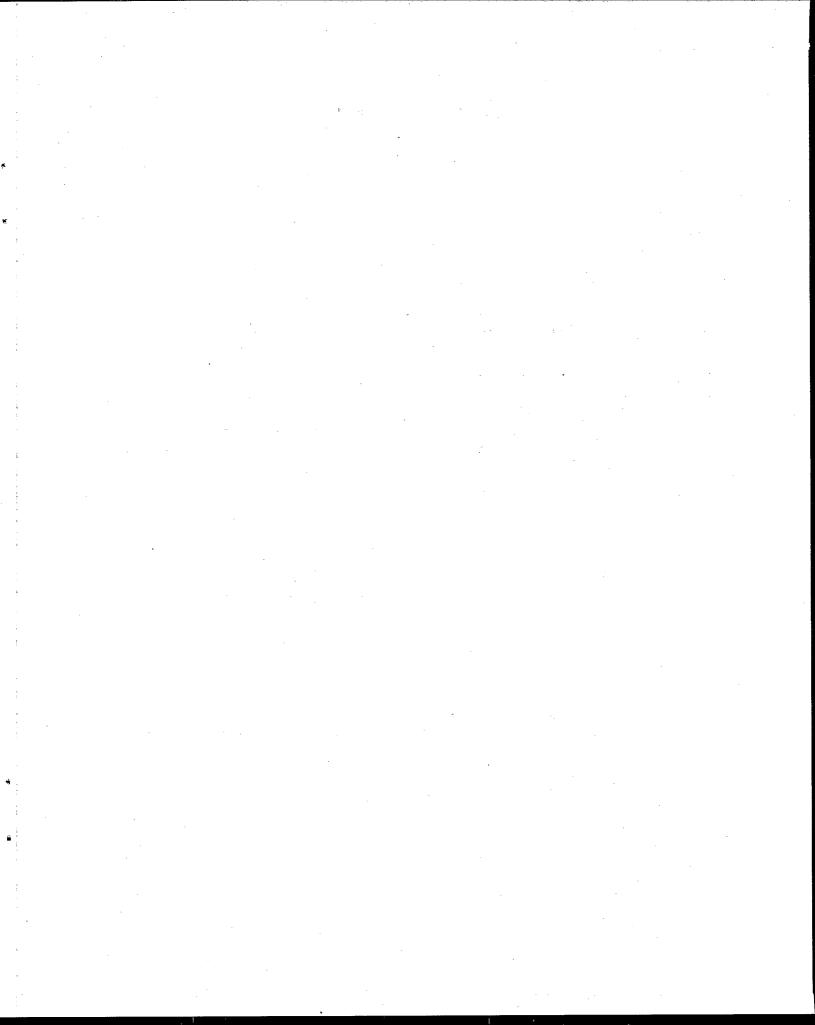
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NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT  MONTHLY REPORT  1,	
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PREPAR BY: RIC ARD L: ALDWELL COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT CML COMPLETE MIX LAGOON
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MONTH Dec.

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CY - CURL YA ARD GRIT COMBIN BASHES ARD GRIT CHAMBE RD - ROWE DE TECH D

ELECTRICIAN MECHANIC OF FRATORS GRADE I OPERAORS GRADE II COMPLIANCE SPEC Разв LABORERS CHIEF OPERATOR : PML-6 PERSONNEL ULL TIM 2 :: GRIT TRUCK PICKU ::90°D BACK HOE WATER TRUCK SERVICES PML-8 DGE AR'SKC z EQUIPMENT ₩ Q MG/L 1::CHLOINE & ULTRAVIOLET DISINFECTION UTILITY COSTS - WATER SERVICE : POWER CON MP 110 :: CHLORINE CONSU BILLING DATE: SULPHUR D XIDE: CONTROL BUILDING :: CML- :: CML- :: EFFLUEN RAW :: EFFLUEN R **22223333**22222222222333323222 UTILITIES AND CH HLORINE E - 4,287 SOLIDS MG/L 87 5 514 70 286 햣 12112 12 22 23 24 24 24 SBJ LBS KWH. PERSONNEL S 11 11.44 14 : MOTOR GRADER & OPERATO E OVER TIME (OPERATORS) ; ELECTRICIAN SOLIDS MI/L :: LABORERS EFFLUENT SERVICES AND EQUIPMENT INT HEFF INFLUEN : ICATI ON B.O.D. TOTAL CL E 1/10 1456 HRS HHS :: (POUIII)S) CHUTCH OPI D £ 2 E 2 2 E E ŧ С C . CHEI RE L'OVE 32 C/Y 23 G/Y 5 C/Y 7.: 181 10 0/ COLLOKY 1.00 SMICH TOTAL TRICKILAN UV-2 HOURS Principle of the service of the serv DATE 

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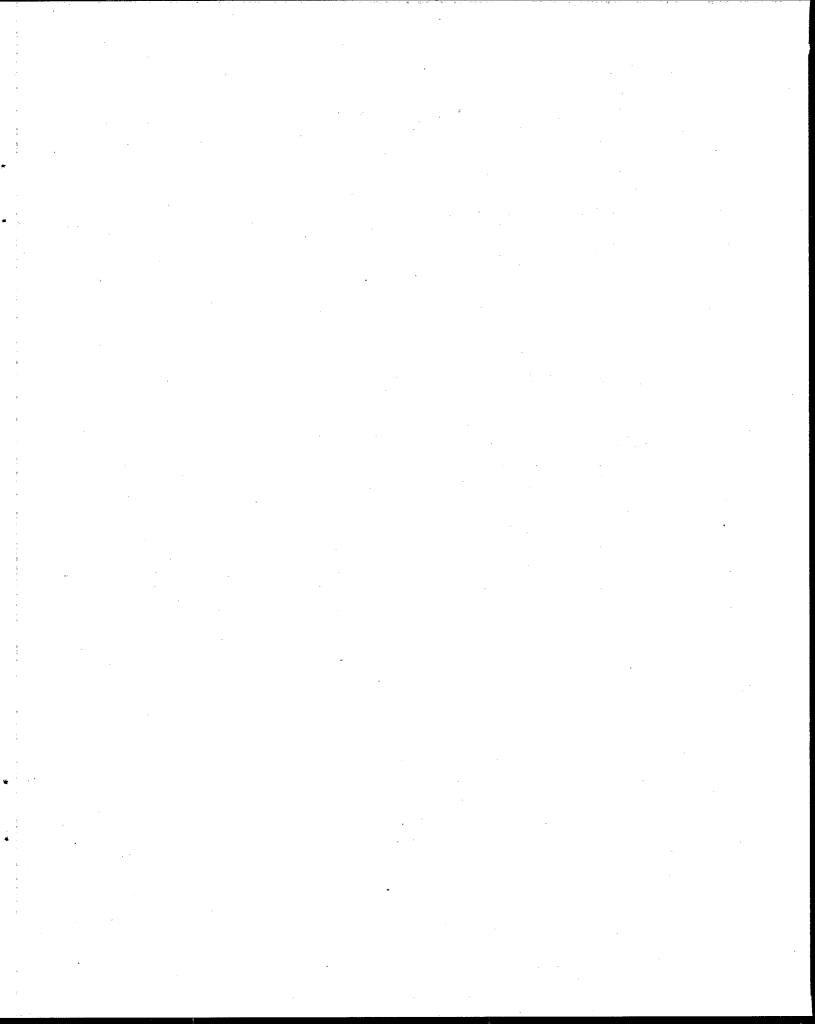
# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

#### MONTHLY REPORT

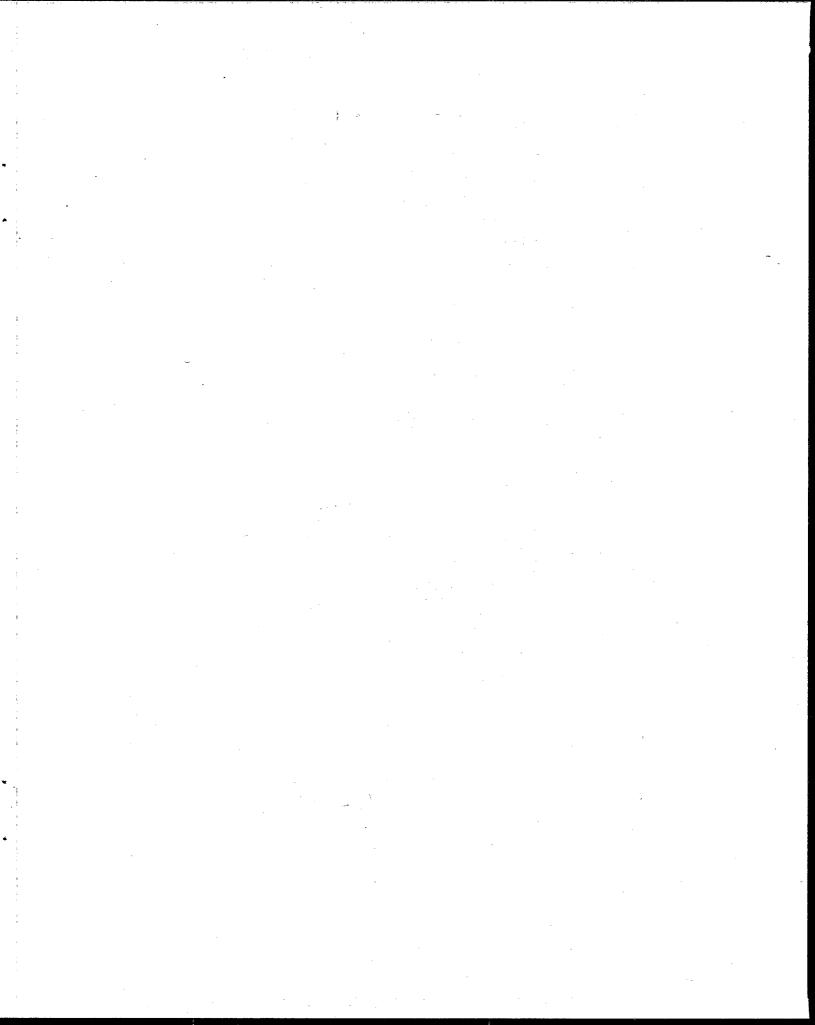
PREPAR BY RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

#### MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY. LINO VEGA, PLANT SUPERINTENDENT

CML COMPLETE MIX LAGOON PML PARTIAL MIX LAGOON

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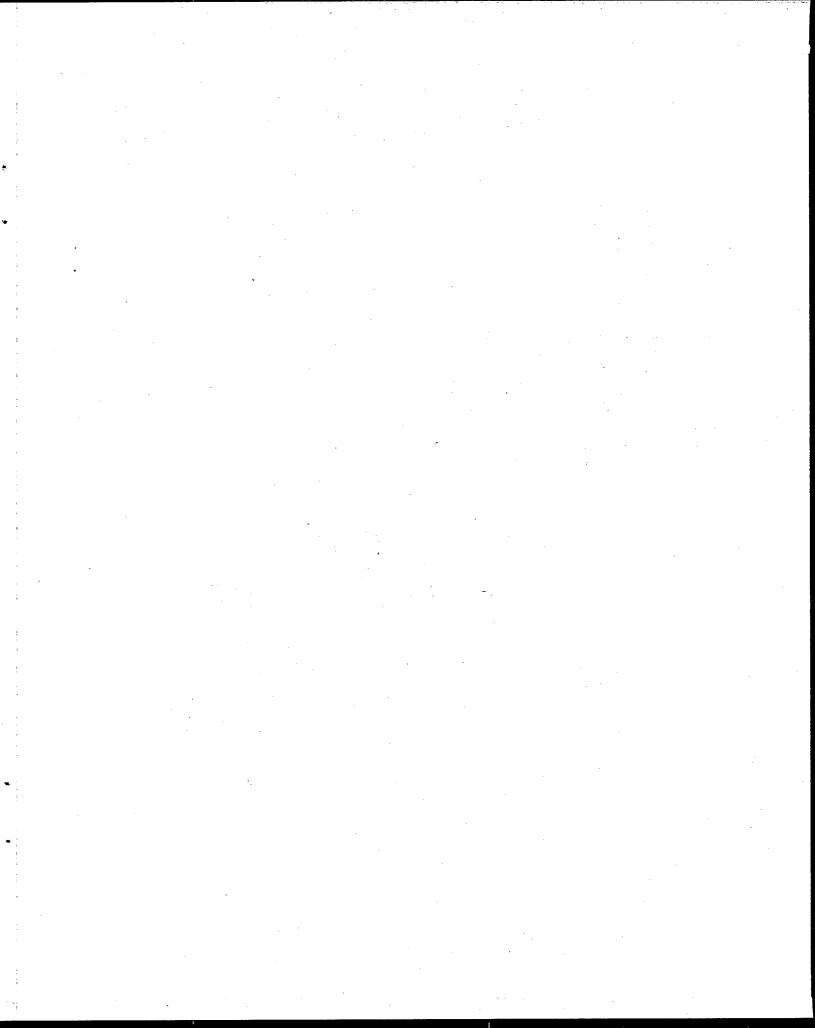
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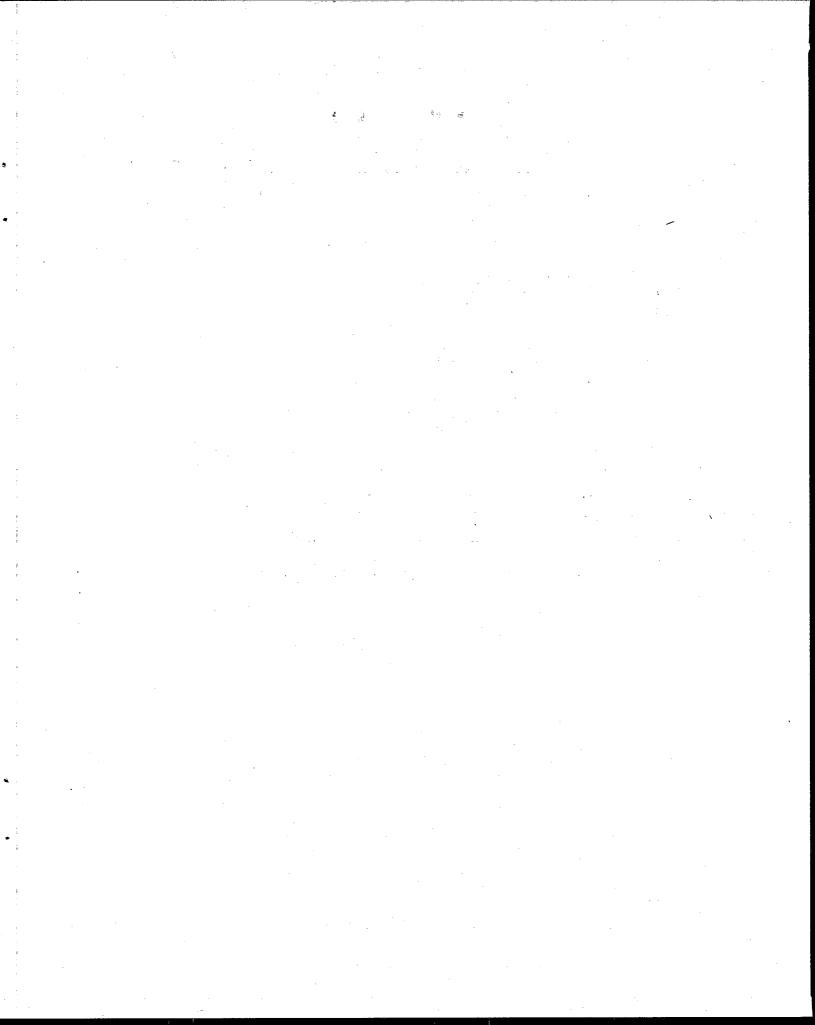
MONTH: Sept.

1993

MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST
AFPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT



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### NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CMI COMPLETE MIX LAGOON
PLIL PARTIAL MIX LAGOON

MONTH. August 1993

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

MONTHLY REPORT

APPREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CMIL COMPLETE MIX LAGOON PML PARTIAL MIX LAGOON

MONTH Page 1 [ JULY 1993

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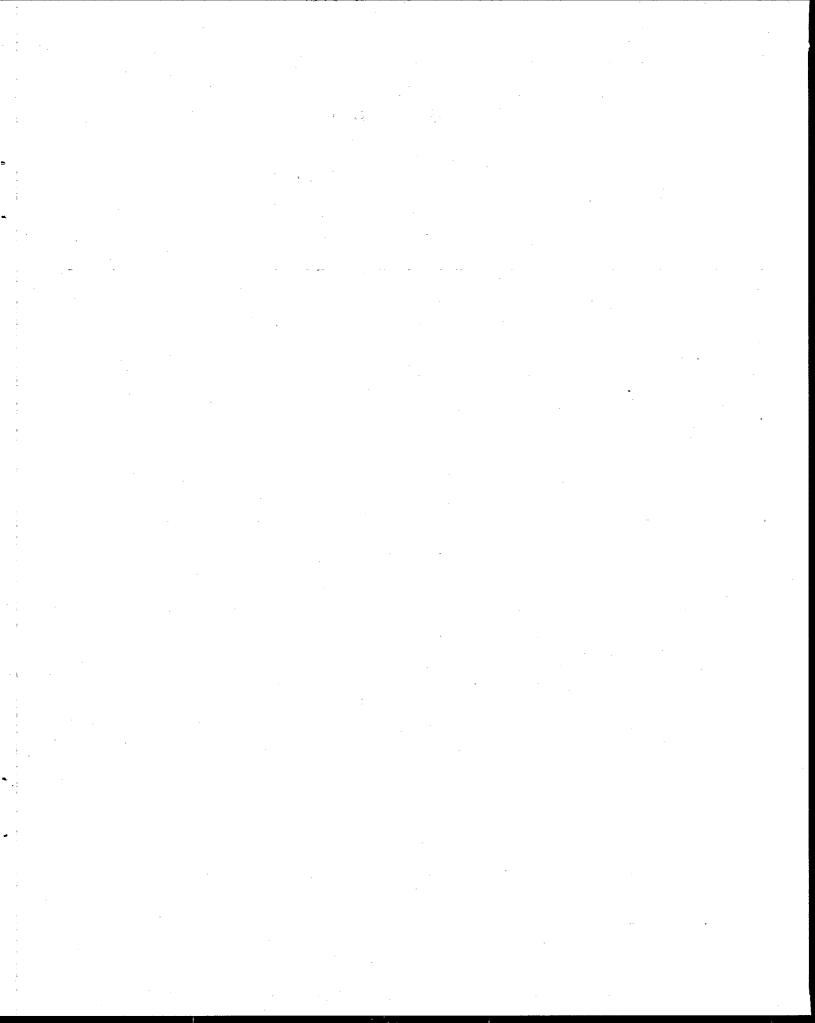
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NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT
MONTHLY REPORT
PREPAR BY: RIC ARD L. ATT
PROVED AS TO F RM. PREPAR BY: RIC ARDL. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT CML COMPLETE MIX LAGOON, PML PARTIAL MIX LAGOON,

MONTH:

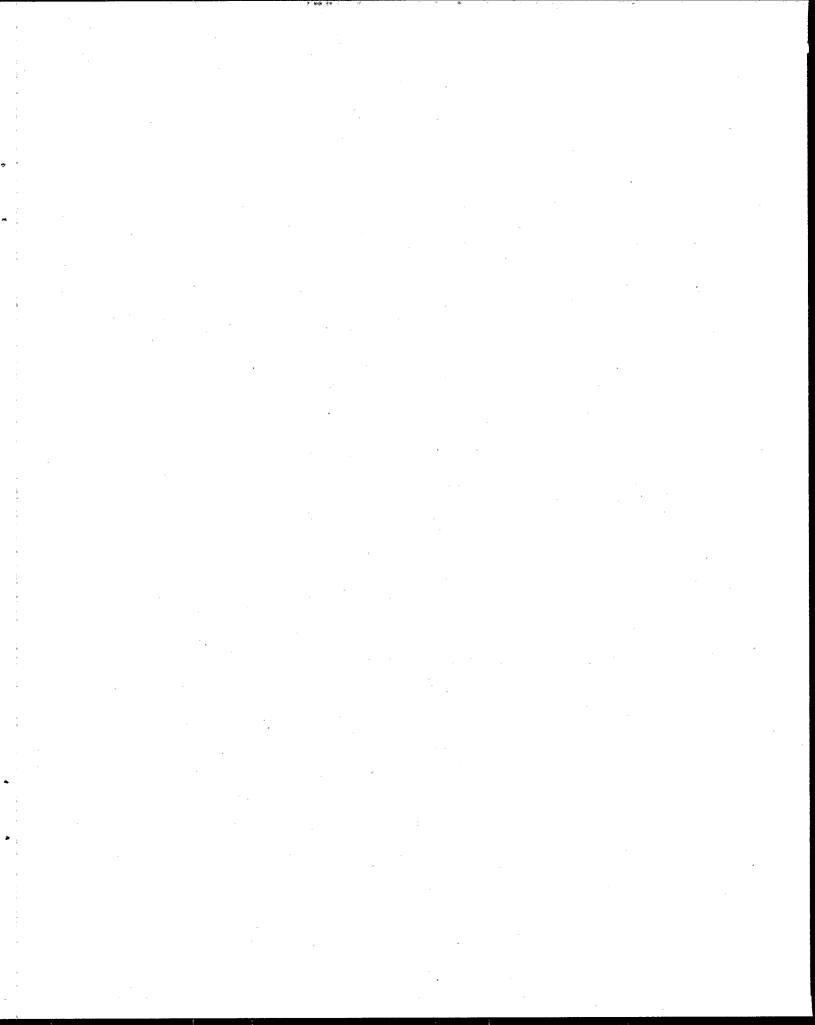
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10 83  EFF & INFLUEN  ICH (POUNDS), RLM-MF (COHFORD)  EFF & 200  101.1 316  200  10.3 200  10.3 200  10.4 800.5 145 mg/l  10.5 162 mg/l  10.6 27 22 192  10.8 3  10.8	T.O. 7.1 7.2 6.9 1204 12. 12. 12. 12. 12. 12. 12. 12. 12. 12.	FEET HOURS FEET HOURS  9	PMI6  PMI6
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### NOGALES INTERNATIONAL WASTEWATER, TREATMENT PLANT MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC. APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CML:COMPLETE MIX LAGOON
PAIL PARTIAL MIX LAGOON

MONTH: May

1993

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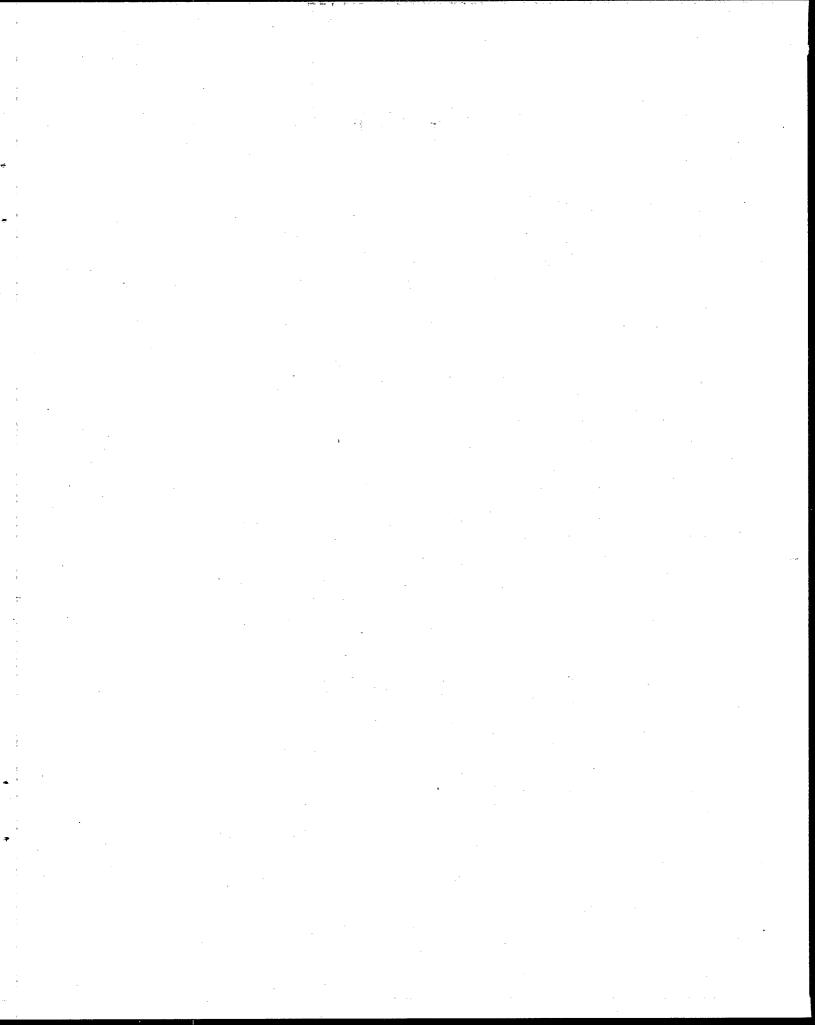
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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

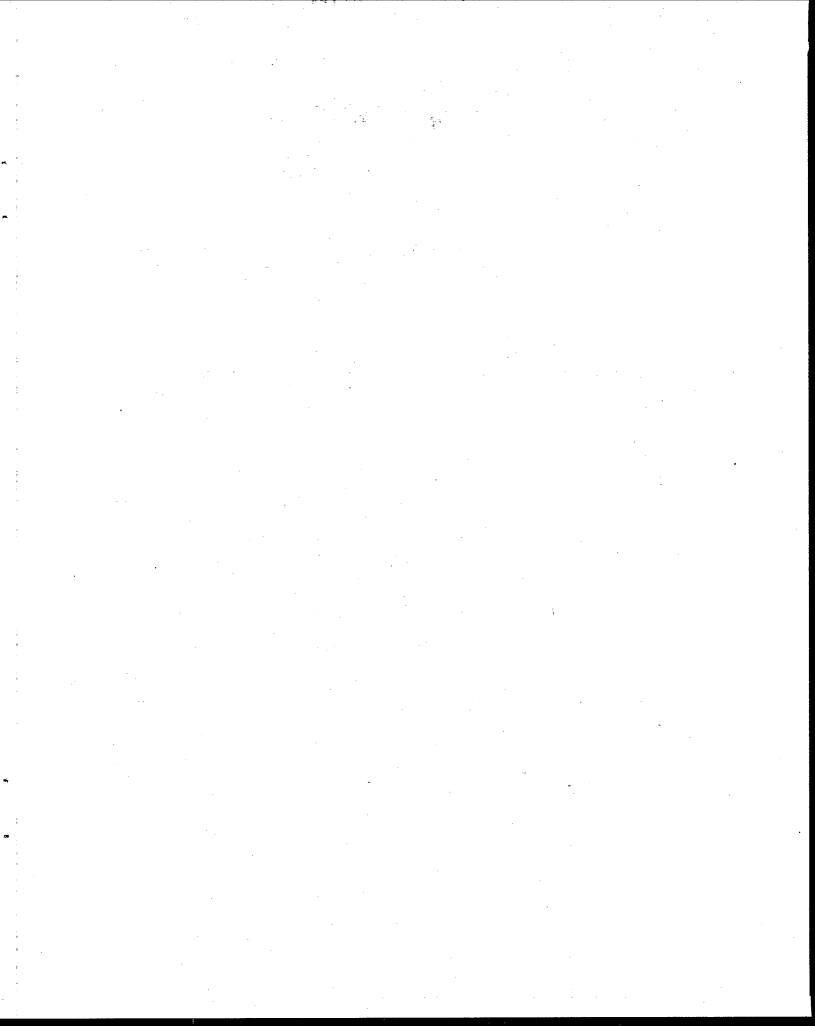
### MONTHLY REPORT

PREPAR BY: RIC ARD L'ALDWELL, COMPLIANE SPEC.
APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

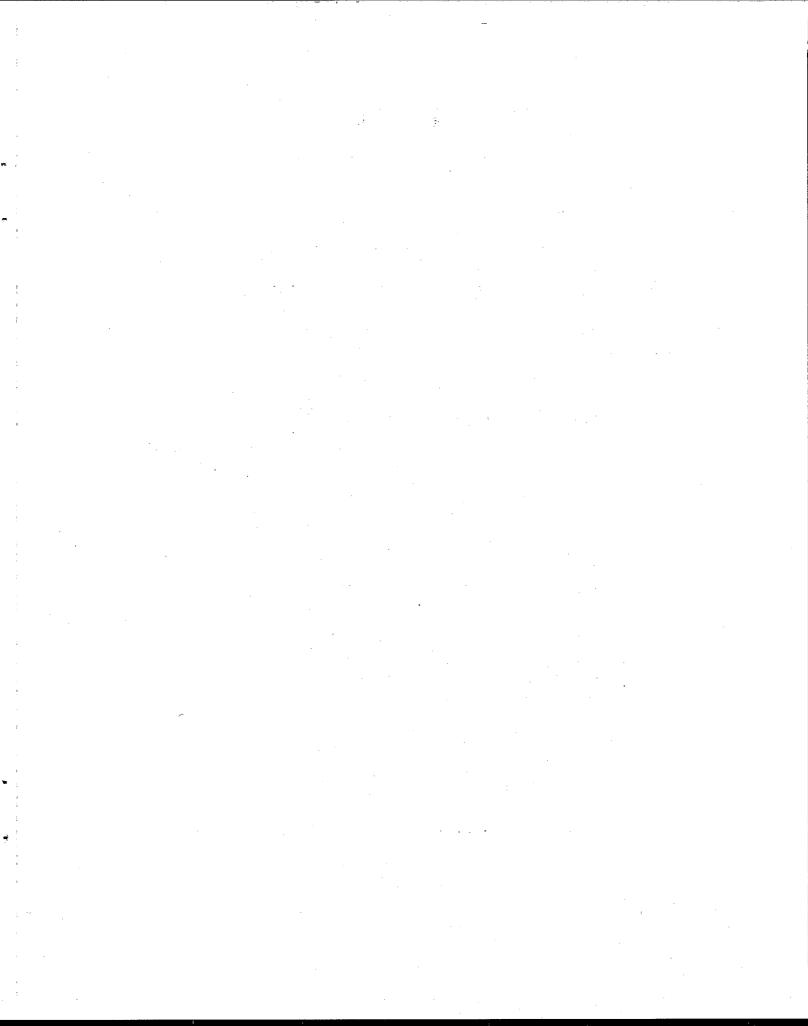
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ELECTRICIAN :: VAC	VACANT:	: SULPHURD XIDE:	4,260 0 LBS. :	MECHANIC : 2	TSS 134 mg/l	
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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPEC.
APPROVED AS TO FORM BY WASTEWATER SUPT. LINO VEGA

CML:COMPLETE MIX LAGOON PML:PARTIAL MIX LAGOON ,

MONTH: March

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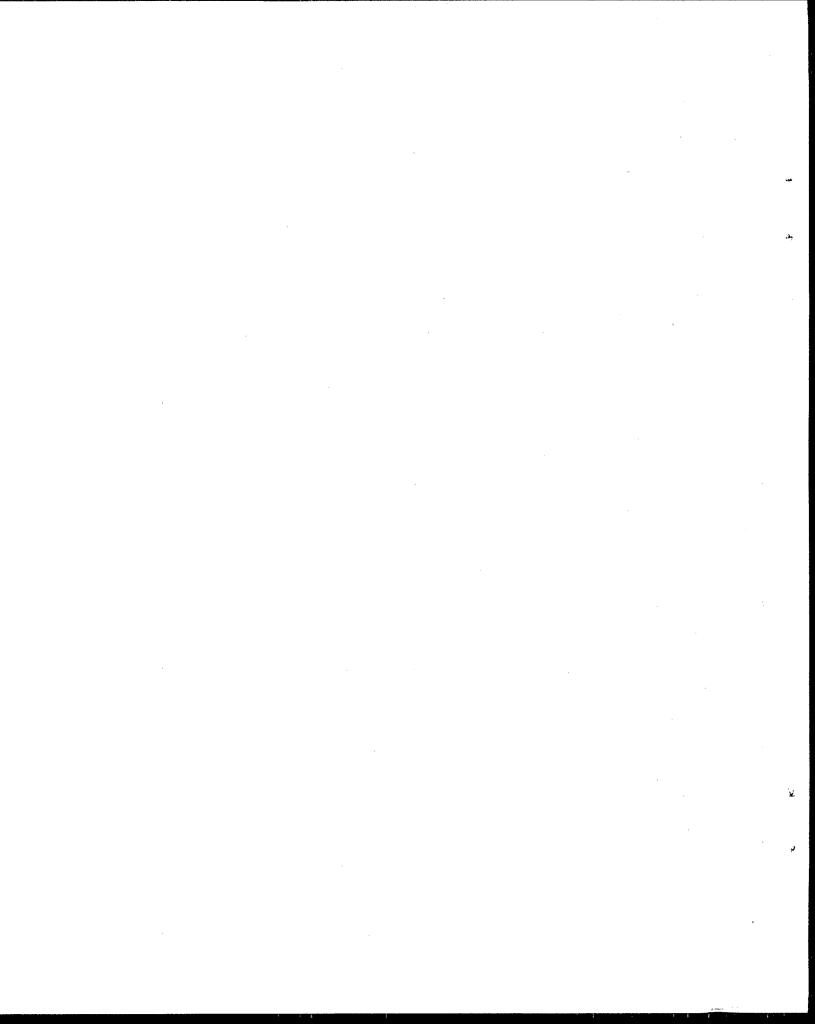
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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

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MONTHLY REPORT

PREPAR BY RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CHL:COMPLETE MIX LAGOON PHL:PARTIAL MIX LAGOON

Page 1

MONTH: February 199

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MONTHLY REPORT

MONTHLY REPORT

PREPAR BY: RIC ARD L: ALDWELL, COMPLIANE SPECIALIST

APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

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# NUGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

### MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PILMT SUPERINTENDENT

CML:COMPLETE MIX LAGOON PHIL PARTIAL MIX LAGOON

1994

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MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST
APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT
CML:COMPLETE MIX LAGOON
PHIL PARTIAL MIX LAGOON

Page 1

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1994

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### REATMENT PLANT NOGALES INTERNATIONAL WASTEWATER'T MOOTHLY REPORT

MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TOF RMBY: LINO VEGA, INT.

CAIL-COMPLETE MIX LAGOON
PML:PARTIAL MIX LAGOON

June Page 1

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, CAMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CMI. COMPLETE MIX LAGOON PMI. PARTIAL MIX LAGOON

Page 1

MONTH: July

1994

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# NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT MONTHLY REPORT

MONTHLY REPORT

PREPAR BY RIC ARD L. 'ALDWELL, COMPLIANE SPECIALIST
APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CML COMPLETE MIX LAGOON
PML PARTIAL MIX LAGOON

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AONTH August

1994

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### NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

### MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT CML. CONPLETE MIX LAGOON
PAIL PARTIAL MIX LAGOON

Page 1

MONTH:

1994

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NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT
MONTHLY REPORT

"REPAR BY: RIC ARD L. APPROVED AS TO F RM PARAMETER TO SAME." PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CALLCOMPLETE MIX LAGOON PAIL PARTIAL MIX LAGOON

Page 1

MONTH: October

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# NOGALES INTERNATIONAL WASTEWATER TREATAIENT PLANT MONTHLY REPORT

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APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT
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## NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

## MONTHLY REPORT

PREPAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CAIL:COMPLETE MIX LAGOON PAIL.PARTIAL MIX LAGOON

Page 1 MONTH Dec

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## NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

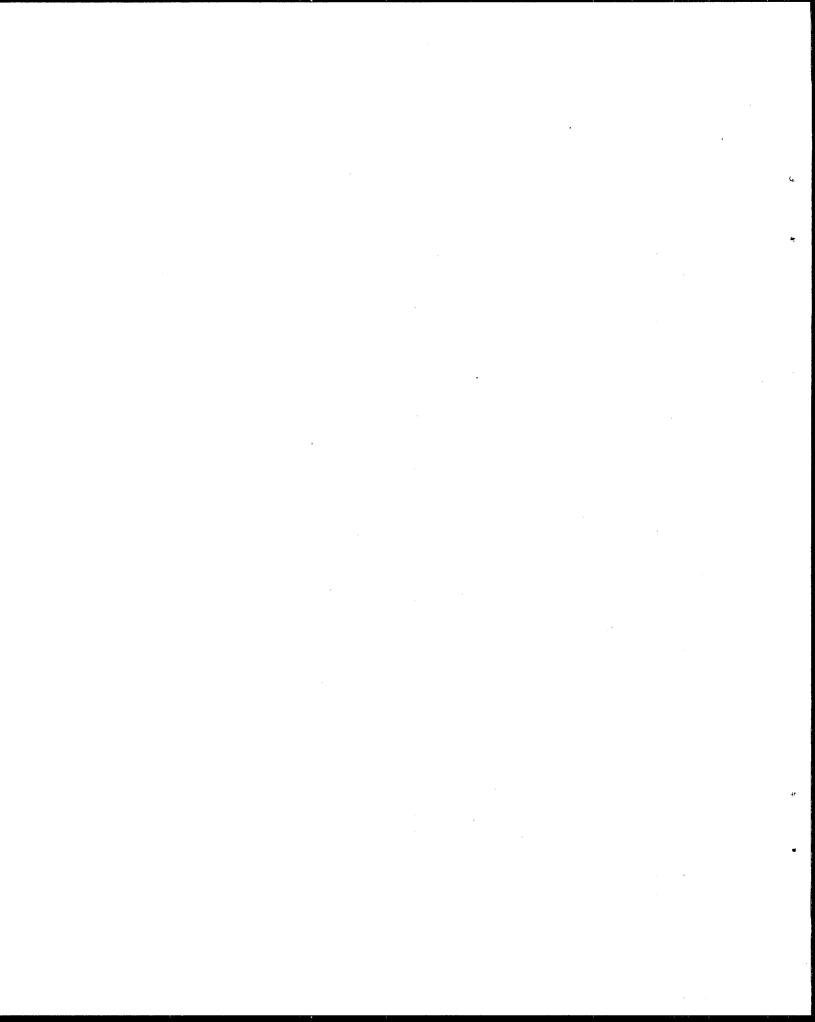
MONTHLY REPORT

PREPAR BY: RIC ARDL. ALDWELL, COMPLIANE SPECIALIST
APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT
CM.: COMPLETE MIX LAGOON
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Page 1

1995. MUNTH. January.

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NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

MONTHLY REPORT

MPLIANE SPECIALIST IT SUPERINTENDENT PREPAR BY: RIC ARD L. ALDWELL, COMPLIA APPROVED AS TO F RM BY: LINO VEGA, PLANT SU

CAIL COMPLETE MIX LAGOON PAIL PARTIAL MIX LAGOON ;

1995;

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PRI PAR BY: RIC ARD L. ALDWELL, COMPLIANE SPECIALIST APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENDENT

CMI COMPLETE MIX LAGOON. PMI PARTIAL MIX LAGOON

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NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT
MONTHLY REPORT

PREPAR BY: RIC AROL: ALDWELL, COMPLIANE SPECIALIST
APPROVED AS TO F RM BY: LINO VEGA, PLANT SUPERINTENIDENT
CALL: COMPLETE MIX LAGOON
PAIL PARTIAL MIX LAGOON

CARL:COMPLETE MIX LAGOON
PART PARTIAL MIX LAGOON

MONTH: April Paye 1

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概 NOGALES INTERNATIONAL WASTEWATER TREATMENT PLANT

MONTHLY REPORT

APLIANE SPECIALIST IT SUPERINTENDENT PREPAR BY: RIC ARD L. ALDWELL, COMP APPROVED AS TO F RM BY: LINO VEGA, PLANT

CAIL COMPLETE MIX LAGOON PAIL PARTIAL MIX LAGOON

1995

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